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Morphological Impact of Large Scale Marine Sand Extraction

(M.Sc. Thesis)

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**CLIENT:**
Ministry of Transport, Public Works and Water Management
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**TITLE:**
Morphological Impact of Large Scale Marine Sand Extraction

**ABSTRACT:**
The investigation is part of the Maasvlakte-2 project and also fits within the ‘Kust 2000’ program of the National Institute for Coastal and Marine Management/RIKZ. The report serves as a Masters Thesis for the author at the Department of Civil Engineering Technology and Management, Faculty of Technology and Management, University of Twente. The purpose is to investigate the impact of the morphological development of the planned marine sand extraction for the construction of Maasvlakte-2. Morphological developments will influence different user functions. The effects on four user functions have been considered: coastline maintenance and risk of flooding, the necessity of future maintenance of the Euro Maasgeul, the stability of the nearby located gas pipeline and finally seabed ecology.

The approach which has been used is to analyse hydrodynamics, depth measurements and dredging data of the Euro Maasgeul area and to model the behaviour of the slopes of the sand excavation, in long shore and cross shore directions. For the morphological modelling, SUTRENCHE and UNIBEST TC have been used. Additionally, the influence of bed slope effects on morphological developments has been investigated, since they are poorly represented in both the utilised models and are thought to be of importance for the quality of the predictions.

Regarding the user functions the following conclusions may be drawn. The gas pipeline northerly of the proposed marine sand extraction only far off shore (> NAP-25 m) might be in danger. Apart from a relatively small area at the borders, sea bed ecology in the extraction pit will be hardly influenced by sedimentation. The widening and deepening of the Maasgeul will reduce the necessity of maintenance of the Maasgeul to zero for at least a hundred years. Large scale marine sand extraction will not lead to a decrease of coastal stability as a result of cross shore transport processes.

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Ton Hoitink
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Summary

Up to the present, the extraction of sand from the North Sea takes place mainly for the purpose of beach nourishment and land filling on shore. Current policy on marine sand extraction regulates these small-scale sand mining projects so that conflicts with other user functions of the North Sea can be prevented. The other user functions can be categorised in six groups, regarding: coastline maintenance, pipelines and communication cables, prevention of flooding, seabed ecology, water quality and navigation. Plans for the construction of a second Maasvlakte concern a land reclamation of 1750 hectares. The estimated quantity of sand to be mined as near to the present Maasvlakte as possible, amounts to 800 million cubic meters. The proposed sand excavation concerns a widening of the Maasgeul from 500 to 2700 m, a deepening from NAP -26 m to NAP -31 m and an additional 2 m excavation of an area of 15000 hectares, north of the Maasgeul. The impact of such a human interference on the enumerated user functions will be substantial and the current regulations for such a large-scale project should be reconsidered.

Within this context, the purpose of the present report is to investigate the impact of the morphological development of the planned marine sand extraction for the construction of Maasvlakte-2. Morphological developments will influence different user functions. In this study, the effects on four user functions have been considered: coastline maintenance and risk of flooding, the necessity of future maintenance of the Euro Maasgeul, the stability of the nearby located gas pipeline and finally seabed ecology.

The approach which has been used is to analyse hydrodynamics, depth measurements and dredging data of the Euro Maasgeul area and to model the behaviour of the slopes of the sand excavation, in long shore and cross shore directions. For the morphological modelling, SUTRENCHE and UNIBEST TC have been used. Both are two dimensional, vertical models. SUTRENCHE has been used for modelling of long shore sediment transport over a simulation period of 50 years and UNIBEST TC has been applied to the cross shore sediment transport for a simulation period of 4 years. The trajectories of the long shore calculations are located across the Maasgeul, between 4.5 and 7.0 km from the coast, which corresponds to depths between 19 and 21 m. The cross shore trajectory is situated at right angles to the coast of Hook of Holland. Finally, the influence of bed slope effects on morphological developments has been investigated, since they are poorly represented in both the utilised models and are thought to be of importance for the quality of the predictions.

From the analysis of data on depth measurements it follows that the northerly slope of the Maasgeul erodes with a rate of 6 cm per year at 21 m water depth to 26 cm per year at 19 m water depth, as a result of a residual current to the North. On the southerly slope, to some extent, deposition of sediment seems to take place, but only marginally. The maintenance dredging of the Maasgeul appears to decrease exponentially with the distance to the coast. The order of magnitude of the maintenance dredging amounts 59 m³ m⁻¹ year⁻¹ at 19 m water depth and 34 m³ m⁻¹ year⁻¹ at 22 m water depth.

The erosion of the northerly slope can be simulated by the SUTRENCHE model with realistic parameter settings. The decrease of morphologic activity between 19 m and 21 m water depth, which was seen in the data, can also be observed in the SUTRENCHE results, but to a smaller extent. Contrary to the data analysis, on the southerly slope in the SUTRENCHE model, permanent deposition takes place. The sediment transport rates coincide well with earlier executed model computations. The reason for the discrepancy between the data and
model results are threefold. First, the current sediment transport formulations lead to an underestimation of local bed slope effects. Second, hydrodynamics have been oversimplified, since density differences and the complex bed topography lead to a water motion that is essentially three dimensional. Finally, the formulations for initiation of motion might under estimate critical bed shear stresses.

The SUTRENCH results show that during a period of 50 years sedimentation and erosion in the proposed sand extraction remains restricted to a strip of 300 m width at the slopes of the extraction pit. Local sedimentation and erosion is 1.4 m per year at most, but is generally far smaller. Flattening of the slopes of the excavation is limited. The migration rates of the slopes is approximately 4 to 8 m per year and is larger for a larger width-depth ratio of the extraction pit.

The UNIBEST TC computations show that transport mechanisms which are dominant in the near shore zone can be applied to the situation outside the NAP-20 m without difficulty. The resulting sedimentation and erosion, either with or without a sand extraction, is in the order of mm per year, which is negligible. The above-mentioned mechanisms predominantly lead to erosion of the cross shore slope.

The effect of a sloping bed on sediment transport is an increase of the transport rate in the direction of the slope as a result of gravity acting upon sediment particles. Both in a situation of a current parallel to a slope and when a current is directed perpendicular to a slope, there will be an increase of the downhill sediment transport component. This is due to the fact that sediment particles easily will roll down a hill and will have difficulty rolling uphill. It can be shown that the occurrence of such mechanism can lead to substantial flattening of slopes and is likely to be the reason for the fact that the southern slope of the Maasgeul in reality does not become steeper like it does in the SUTRENCH simulation.

Regarding the user functions the following conclusions may be drawn. The gas pipeline northerly of the proposed marine sand extraction only far off shore (> NAP-25 m) might be in danger. Apart from a relatively small area at the borders, sea bed ecology in the extraction pit will be hardly influenced by sedimentation. The widening and deepening of the Maasgeul will reduce the necessity of maintenance of the Maasgeul to zero for at least a hundred years. Large scale marine sand extraction will not lead to a decrease of coastal stability as a result of cross shore transport processes.
1 Background of the Study

The present report will deal with the morphological impact of large scale marine sand extraction for land reclamation purposes in the Dutch part of the North Sea. Before specifying the purpose of the investigation, the current practice of sand mining at sea will be confronted with existing plans concerning marine sand extraction within the framework of the Maasvlakte\(^2\) project. Therefore, the current policy on marine sand extraction, the physical effects of large scale sand winning at sea and the plans to build the artificial peninsula Maasvlakte\(^2\) will be described hereafter.

Note: If figures are not included throughout the text, they can be found in the last section of the report.

1.1 Current Policy on Marine Sand Extraction

Policy concerning marine sand extraction started to be developed in the early 1980's, when the need for land fill sand increased while the capacity of conventional sand resources no longer met the requirements. In order to prevent shortages, the use of superficial mineral deposits from the North Sea was advocated in, amongst others, the report 'Harmonization North Sea Policy', the report 'Land fill sand' and the report 'Efficient Extraction' (see references). As a result, marine mining of fill up sand increased to 20 million m\(^3\) per year in the period 1991 to 1994. From 1995 onwards, the yearly extracted amount decreased to 15 million m\(^3\). Simultaneously, an additional extraction of 11 million m\(^3\) per year took place for beach nourishment purpose.

In 1987 an Environmental Impact Assessment (MER) procedure started, taking into consideration all economically extractable minerals in the North Sea: sand, gravel, clay and shells. The aim of the study was to investigate if and how superficial mineral deposits from the North Sea can make a systematic and co-ordinated contribution to the supply of the national requirements, on short, medium and long terms.

The MER resulted in the Regional Extraction Plan for the Dutch Part of the North Sea of 1991. Though carried out to create procedures for private, relatively small-scale projects, many elements of the policy will apply to large-scale marine sand extraction. After a short review of the objectives of the extraction policy the implications for large-scale marine sand extraction will be set out.

The objectives of the excavation policy with regard to the North Sea can be formulated as follows:

1. *Taking responsibility for a socially sound way of extracting superficial mineral deposits from the North Sea*

The objective is to make a balanced evaluation of the interests of the various uses. This can be set out as follows:

**Economic aspect:** The extraction of superficial mineral deposits must take place in an economically efficient way. The economic consequences
of the extraction of superficial mineral deposits must be acceptable with regard to the other uses of the North Sea.

**Spatial aspect:** A balance between the winning of superficial mineral deposits and other functions of the North Sea has to be attained. Possible hindrance to other functions must be limited and if possible prevented.

**Environmental aspect:** The effects of the winning of superficial mineral deposits on the water system have to be zero, or else an acceptable minimum. Methods/techniques of extraction and schedules have to be chosen such that the effects of extraction are either zero or an acceptable minimum. The salt load of ground and surface water as a result of using sea sand at inland locations has to be minimised.

**(Inter)national administrative aspect:** The aim is to provide an administrative framework for industry to extract materials in an economically feasible way. A good and democratic administration/management should be encouraged. Formulation of rules for the extraction policy need to be moderate. There should be cooperation with other North Sea coast countries to reach an international agreement.

2. **Encouragement of efficient use of the superficial mineral deposits of the North Sea Coastal and territorial waters**

The efficient use of superficial mineral deposits implies:
- not using high quality superficial mineral deposits for purposes for which lower quality material would suffice
- combining maintenance dredging (secondary extraction) with primary sand winning: "Making work with work", and recycling superficial mineral deposits.

3. **Taking responsibility for the available superficial mineral deposits from the North Sea Coast and the territorial waters.**

The aim must be that the demand for superficial mineral deposits, on the short, medium and long terms, can be completely satisfied from resources of the North Sea.

The implications for large-scale marine sand extraction are numerous. To begin with, marine superficial minerals appear to be a scarce natural resource, with a clear economical value. Though the total amount of sand in the North Sea might seem to be infinite, dredging an amount of sand however will generally imply higher costs in the future, since one either has to gain minerals from a larger depth or has to navigate over a longer distance. Therefore, an opportunity cost should be taken into consideration when deciding over large-scale sand extraction. That is to say, apart from production costs, scarcity of the marine resource sand should be incorporated within the evaluation of a marine sand extraction plan. The situation can be compared with the governmental policy on natural gas. In order to stimulate optimal use of the natural recourse, production cost and price per m$^3$ strongly differ.
From a more practical point of view, when choosing an extraction area one should try to avoid locations where transport to land is simple, so that these locations can be used for sand extraction for use on land. High-quality minerals such as gravel and coarse sand should not be used to gain land.

Since the economic consequences of marine sand extraction have to be acceptable with regard to other uses of the North Sea, there are zones from which the extraction of superficial mineral deposits is prohibited. In the following table these areas have been summarised.

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<td>Damage/fracture of pipelines</td>
<td>Along the routes of pipelines and 500 meters on either side of the pipelines</td>
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| Tele-communication cables | Damage/fracture of cables | Along the routes of cables and either side of the cables in principal:  
- for cables of which the position is located after laying 1000 m  
- for cables of which the position is not located after laying 2000 m  
Depending on the accuracy of the survey apparatus of the dredger, the distances can be changed. |
| Disposal of refuse   | Contaminated superficial minerals | The designated discharge and dumping areas in the North Sea |
| Protecting the coastline | The safety of the coastline | The excavation of superficial raw materials must take place at such a distance from the coast that safety of seawalls is not placed at risk.  
The limit has been placed at 20 m below NAP i.e. ca. 20 km offshore. This limit applies to both sand and shell winning, with the exception of winning in the channels to Rotterdam and Amsterdam |
| Military activities  | Exercise areas with daily (intensive) use | Exercise areas EHD 41, EHR 4A, HER 4D, HER 8, EHD 42 and HER 15. |
| Offshore-mining      | Safety in relation to collisions | At the location of fixed and mobile mining installations, a zone with a radius of 500 m |
| Fishing industry     | Damage to potential catch | The nursery areas are not necessarily prohibited areas for the extraction of superficial raw materials. However, any extraction within these areas should, in preference, be carried out only outside the growing season (winter/spring) |

Table 1-1 regulation on marine sand extraction [Regional Extraction Plan for the Dutch Part of the North Sea, 1991]

Simultaneously, a fairly complete overview of activity on the North Sea has been obtained. The following two activities can be added:  
- exploration and identification of raw materials  
- shipping, including shipping and access channels to seaports
An important criterion when choosing an extraction area, a technique and a time for the sand extraction, is minimisation of the environmental. That is, negative effects of the mining on the water quality, on the quality and composition of the seabed and on flora and fauna should be avoided. The following general aspects should be borne in mind:

**Extraction technique:** Stationary extraction will change the bottom locally much more than mobile extraction. The bottom fauna will be killed and recovery by the establishment of new animals will proceed very slowly because the composition of the sediment and the current pattern in and around the area will be different from that before extraction.

**Season:** The quality of the water, specifically the suspended matter content, will be most strongly affected during the summer months. Growing fish and fish larvae are, however, vulnerable during the winter half of the year.

**Extraction depth:** Deep extraction has a lasting effect on the morphology and composition of the bottom. Owing to these physical changes in the bottom, the recovery of bottom fauna will take more time then when superficial extraction methods are used.

The policy to satisfy the total demand for superficial mineral deposits in the short, medium and long term from the North Sea might seriously conflict with large-scale projects.

### 1.2 Impacts of Large-scale Marine Sand Extraction

Recently a pilot study has been performed on Effects of Large Scale Marine Sand Extraction, by the National Institute for Coastal and Marine Management/RIKZ, and laid down in a report of the same name [Hoogewoning, 1997]. The goal of the study was to create guidelines for policy on large-scale marine sand excavations. Six ‘user functions’ of the North Sea are distinguished, all of which are concerned with sand excavations. In the next paragraph a summary of effects on these user functions will be given. Thus a general overview of conflicting interests concerning marine sand extraction will be obtained.

#### 1.2.1 Coastal Safety

A sand excavation implies a transformation of hydrodynamic conditions. Water level, wave height and peak period might increase which could lead to a decrease of coastal safety. Kuiper and Philipart [1997] used models to calculate the effect of a 10 m pit at deep water. They concluded that the water level during severe storm conditions was hardly affected by the pit.

#### 1.2.2 Coast Line Maintenance

Changing wave conditions effects the wave field which in turn modifies the mean coast line. These modifications are hard to predict since they are a result of small changes in long shore and cross shore transport. Calculations of [Steijn, 1997] show an increase of longshore transport in the order of \(10^3\) m³/year as a result of an increase in wave intrusion at the coast, which is induced by the sand excavation.
1.2.3 Water Quality

As a result of the tidal motion, the North Sea is highly turbulent. This implies a high ability to mix contaminants from the rivers horizontally as well as vertically. Water quality benefits from this mixing since contaminant concentrations decrease and oxygen will remain available. A large sand excavation, for instance in the neighbourhood of the Rhine outflow, leads to a reduction of flow mixing capacity which might lead to stagnant water. Phenomena like this are not likely to occur for excavations less than 10 m deep.

1.2.4 Ecology

Nearly all fauna that can be found on a sea bed is concentrated in the upper 20 cm. Richness of sea bed fauna can be estimated by measurement of densities, biomass and category of specimen present [Holtmann et al., 1996]. The damage of marine sand extraction to sea bed fauna will depend on duration, span of dredging activity and depth. Increase of either of the three will lead to a less favourable situation. For the former two this is obvious: life at the sea bed will be disturbed over a longer period or over a greater distance. Regarding the third, in a deeper excavation the sedimentation rate will be higher, which will frustrate recovery.

1.2.5 Pipelines and Communication Cables

Pipelines and communication cables usually have a cross-shore orientation. They are covered with sand which prevents them from being exposed to external forcing. Erosion can remove this protective cover which can lead to buckling of the pipeline or cable. An instantaneous effect of a large-scale excavation can be erosion near pipelines or cables. In time, slopes of a sandy excavation at sea might reach pipelines or cables since they probably will migrate and become flatter.

1.2.6 Navigation

Navigation benefits from a sand excavation since the navigable area is enlarged. Apart from this, wave, water level and current conditions near the entrance of a harbour are of vital importance for navigation. Currents and waves can complicate navigation and the average water level influences the keel clearance of a ship. Since marine sand extraction will be far off-shore, effects on near-shore wave and current conditions will be minimal. The local keel clearance where excavation takes place, will initially be influenced in a positive way. In time, there is a chance that the new depth will provoke the generation of sand waves.

1.3 Marine Sand Extraction for the Maasvlakte² Project

In 1994 the Municipality of Rotterdam and the Ministry of Transport, Public Works and Water Management jointly started the project Maasvlakte², within the framework of the Area Planning of the Rijnmond region. The project was induced by predictions stressed in the report ‘Harbour Plan 2010’, in which a future shortage of space in the Rijnmond region in the order of 1250 hectares was estimated. Maasvlakte² refers to a reclamation of land of the North Sea, eastward of the existing Maasvlakte. The aim of the project is an extension of the harbour and industrial area with 1000 hectares and enlargement of the recreational and natural domain of 750 hectares. Maasvlakte² is a large-scale project of national interest,
with major consequences to national economy, public finances, area planning, environment and ecology. This justifies a thorough impact assessment on all aspects and a broad public discussion.

An estimated amount of 800 million m$^3$ of sand has to be acquired in order to obtain sufficient material to build the peninsula, all of which should be gained at locations as close to the Maasvlakte as possible. With respect to extraction areas several plans have been made, all of which suggest to extract the larger part of sand from the Euro-Maasgeul.

Figure 1.1 gives a general overview of the Euro-Maasgeul area. Figure 1.2 is a detail of the same plot, in which the contours of the proposed excavation area have been indicated. In this alternative, starting at the NAP -20 line, the Euro-Maasgeul will be broadened up to 2700 m, and deepened from 26 m to 31 m. Besides, an area of 15 km$^2$, located adjacent to the trench on the northern side, will be excavated by 2 m. The north easterly side of the 2 m excavation area is situated 500 m from a gas pipe line.

In the report 'Influence of the Design of Maasvlakte'2 on the Large Scale Morphodynamics of the Dutch Coast' [Steijn, 1996], four designs are being considered along with four different extraction alternatives. Three of the four designs mainly differ in the extent to which they reach in offshore direction, the remaining design is an island. The above described extraction alternative basically coincides with one of the four alternatives described in the report. The other three suggest to replace the northerly, superficial extraction by a 5 m deep pit on the northern side of the anchoring area, or by a 5 m deep winning south of the anchoring area, or by a superficial winning on the southern side of the Euro Maasgeul. Apparently it is important for decision makers to see the difference between northerly and southerly orientated variants and between deep and superficial extraction. Surprisingly, the distance to the coast has not been varied.

Morphodynamic calculations have been performed, two dimensionally in the horizontal plane, simulating a period of 5 years. Time scales of the morphological response on a large scale intervention like the Maasvlakte will be much larger than 5 years, so that results should be considered initial effects. Focusing on the extraction alternatives, the influence of the variants in five years appears to be relatively small. Sand winning-induced sedimentation as well as erosion appears to be in the order of 5 cm/year. Steijn concludes that the effects of the marine sand extraction are restricted to the borders of the extraction area itself.

A remark can be made referring to the situation of the coast of Delfland, where the beach is permanently being nourished. Steijn's results show that the nearest effects take place in a region which is located 5 to 8 km from the coastline. Indeed this is not alarming, especially not since many of those effects will be a result of a change of silt transport in the vicinity of the silt deposit 'Loswal Noord'. It is highly questionable though, whether the erosion located in a direction north westerly of the end of the Noorderdam, which locally exceeds 20 cm/year, will stay away from the coast on the long term.
2 Purpose and Limitations

A multitude of actors is concerned with large scale marine sand extraction and a variety of effects can be expected from it. Primary aim of this thesis is to predict the morphological effects of a large-scale marine sand extraction such as planned for Maasvlakte². Focusing on morphological effects, four user functions have direct interest in the morphodynamic behaviour of a sand excavation: coastline maintenance, ecology, pipelines and communication cables and navigation. For each of these function a relevant question with regard to the impacts the will be defined.

1. To what extent, how and where will a large-scale marine sand excavation, such as planned for Maasvlakte², affect coastal stability?
2. What rate of deposition can be expected in the excavated areas and how will it change in time?
3. Are pipelines or communication cables endangered as a result of erosion?
4. What can be expected with respect to the future need for maintenance dredging of the Euro-Maasgeul?

Starting point will be the planned marine sand extraction within the Maasvlakte² project. The time horizon will be chosen as large as possible at reasonable computational costs.

Predicting morphological changes is generally attended by high uncertainties. In many cases hydrodynamics and sediment transportation processes are essentially three dimensional, while from a practical point of view it is impossible to model them as such. Physical processes and their interactions, have to be understood completely in order to be able to make valid assumptions. The aim of this investigation also will be to gain generic information on the relevance of various physical processes at deeper water.

Current policy on marine sand extraction as described in the problem definition addresses the present practice of small-scale sand winning for beach nourishment purposes or inland use. Relevant aspects, such as the obligatory minimal distance of an extraction site to a pipeline, will be reconsidered on the basis of the information obtained.
3 Approach

A first step in the approach will be an analysis of the hydrodynamics and bathymetric data of the Maasgeul area. Thus qualitative and quantitative information will be obtained on both the water motion and the morphological changes in the area of interest.

Subsequently the 2DV model SUTRENC, developed by Delft Hydraulics, will be applied to various sections in the tidal direction. Purpose is to predict alongshore effects of the extraction plan. Cross shore developments will be analysed by making use of the 1DV cross-shore model UNIBEST TC, also developed by Delft Hydraulics.

The value of the model results to be obtained will depend on the limitations in the applicability of the models, by the used method of schematising external conditions and by the quality of the models themselves. The first aspect of will be evaluated by investigating the influence of neglected processes. The influence of schematising external conditions will only be assessed in the case of the choice of a single wave that represents the wave climate in case of long-term SUTRENC calculations. No extensive validation study will be performed on the processes implemented in the models. However, one of the shortcomings of both models will be discussed by focusing on slope effects, which might play an important role in predicting morphological developments.
4 Hydrodynamics

This chapter gives a general description of the hydrodynamics in the Euro-Maasgeul area. The main driving forces and their relative importance are described. Subsequently, the possibility of modelling the situation to a two dimensional case is analysed. The chapter is mainly based on the study Marine Sand Extraction [Allersma and Ribberink, 1992], which includes an extensive study on the hydrodynamics in the Euro-Maasgeul area. In the next paragraph an interpretation of this investigation will be given, supplemented by a detailed description of residual currents, based on the report Calibration of the 3D-Coastal Strip-Model for Silt Transport [De Kok, and Salden, 1995]. In order to quantify the phenomenon of tidal current contraction in the surroundings of Maasvlakte, a 2DH calculation based on the Rijnmamo model has been done.

Characteristic for hydrodynamics in the Euro Maasgeul area is the diurnal astronomical tide, with oscillating currents along the coast and in the Maasmond. Besides the tidal motion, other driving forces such as fresh water outflow, density gradients, short waves, wind and Coriolis acceleration strongly influence the water motion. Because of the non-uniformity of the coast and irregular local bed topography, the resulting three dimensional pattern is complex and highly dynamic. The various driving forces will be treated separately. Subsequently, the resulting water motion will be analysed.

4.1 Driving forces

The following driving forces will be considered: tidal motion, fresh water outflow and salt water intrusion, wind, Coriolis acceleration and short waves.

4.1.1 Tidal motion

The tidal wave in the Euro-Maasgeul area propagates in a direction N35°E at flood and N225°E during ebb. It has a component perpendicular to the main direction which approximates 5 to 10% of the main current, causing a rotation of the velocity vector. The northerly flood current lasts a relatively short period and is attended by higher water levels and velocities then the ebb stream. Due to the filling and emptying of Europoort, flow and water level are approximately a quarter of a period out of phase. The vertical tide can be described on the basis of observations at Hoek van Holland, located at the coast, and at the Europlatform, an observation platform at 34 m depth and ca. 40 km offshore, slightly north of the Eurogeul. Table 4.1 gives a review of characteristics.

<table>
<thead>
<tr>
<th></th>
<th>Hoek van Holland</th>
<th>Europlatform</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>high tide</td>
<td>low tide</td>
</tr>
<tr>
<td>spring-tide</td>
<td>+1.21</td>
<td>-0.63</td>
</tr>
<tr>
<td>average tide</td>
<td>+1.05</td>
<td>-0.64</td>
</tr>
<tr>
<td>neap-tide</td>
<td>+0.82</td>
<td>-0.64</td>
</tr>
</tbody>
</table>

Table 4-1 Characteristic Water Levels with respect to NAP, in m [Tidal Tables for the Netherlands, 1991]
An impression of the horizontal tidal motion can be obtained by integration of the tidal velocities. Besides an excursion of 11 to 16 km there is a northerly residual displacement of 2 to 2.5 km per tidal period, which is about 8% of the tidal path. Table 4.2 gives more detailed information.

<table>
<thead>
<tr>
<th></th>
<th>spring tide</th>
<th>neap tide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum flood (m/s)</td>
<td>1.0</td>
<td>0.9</td>
</tr>
<tr>
<td>Maximum ebb (m/s)</td>
<td>0.9</td>
<td>0.75</td>
</tr>
<tr>
<td>Tidal path</td>
<td></td>
<td></td>
</tr>
<tr>
<td>flood (km)</td>
<td>16.0</td>
<td>13.0</td>
</tr>
<tr>
<td>ebb (km)</td>
<td>13.5</td>
<td>11.0</td>
</tr>
<tr>
<td>total (km)</td>
<td>29.5</td>
<td>24.5</td>
</tr>
<tr>
<td>residual (km)</td>
<td>2.5</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 4.2  Tidal Motion at Sea [Allersma and Ribberink, 1992]

4.1.2 Fresh Water Outflow and Tidal Intrusion

Fresh water outflow of the Rhine particularly takes place during ebb, when the water level at sea is relatively low. The low-density fresh water moves in a thin layer along with the ebb stream to the south. During flood, the fresh water plume moves along to the north. As a result of both the increasing water level and the horizontal density gradients, sea water intrudes in the Maasmond, beneath the fresh water. The vertical density gradients have a damping effect on turbulence. Due to the net displacement of the water to the north, the net effect is a fresh water ‘river’ near the surface which moves along the Dutch coast in a northerly direction. Near the bottom the tidally averaged secondary streaming due to density gradients is directed shoreward. Its magnitude is largest near the Rhine outflow.

4.1.3 Wind and Coriolis Acceleration

Wind action on the water surface is the most relevant meteorological factor. The wind force varies in strength over the year with a minimum in spring and a maximum during autumn and winter. Table 4.3 gives an overview of frequencies of wind velocities in two characteristic periods; table 4.4 contains information on yearly averaged frequencies of wind directions. A semi empirical formula states that the wind induced surface current velocity is approximately 3% of the wind speed. Another formula relates the wind to the wind speed via $v \approx 0.02 W^2$.

<table>
<thead>
<tr>
<th>Force (BF)</th>
<th>0 - 1</th>
<th>2 - 3</th>
<th>4</th>
<th>5 - 6</th>
<th>7 - 12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity (m/s)</td>
<td>(\leq 2.5)</td>
<td>2 - 5</td>
<td>5.5 - 8</td>
<td>8.5 - 13.5</td>
<td>(\geq 14)</td>
</tr>
<tr>
<td>Frequency (%)</td>
<td>may</td>
<td>18</td>
<td>52</td>
<td>17</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>nov/dec</td>
<td>6</td>
<td>40</td>
<td>22</td>
<td>23</td>
</tr>
</tbody>
</table>

Table 4.3  Frequencies of wind velocities [Allersma and Ribberink, 1992]

Dominating directions are onshore, from South West to North West. This causes a shoreward current near the surface and a compensating off shore current in the underlying layer. The effect of the Coriolis acceleration on wind driven currents is relatively small due to the limited depth. Nevertheless it does contribute to down welling during flood and upwelling during ebb.
4.1.4 **Short Waves**

The orbital velocities of wind waves cause an oscillating motion near the bottom aligned with the direction of wave propagation. Non linear effects cause a net flow in the wave propagation direction in both a layer near the bottom and in the wave-to-crest layer. The resulting mass flux is compensated by a flow in the intermediate layer, in a direction that depends on the angle of incoming of the wave and other hydrodynamic conditions. An indication of maximal orbital velocities near the bottom as a function of the water depth is given in table 4.4.

<table>
<thead>
<tr>
<th>depth [m]</th>
<th>percentage of exceedance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>90%</td>
</tr>
<tr>
<td>&lt;12.5</td>
<td>0.15</td>
</tr>
<tr>
<td>12.5 - 16</td>
<td>0.08</td>
</tr>
<tr>
<td>16 - 20</td>
<td>0.04</td>
</tr>
<tr>
<td>20 - 25</td>
<td>0.025</td>
</tr>
</tbody>
</table>

Table 4-4  Maximal orbital velocities near the bottom in m/s [Bakker and Bangert, 1969]

The wave climate in the study area can be represented by characteristic wave heights per compass sector and frequencies of occurrence of waves with a specific height and period. Aggregated data on wave heights, periods and directions based on measurements at the platform Licht Eiland Goeree are summarised in the two tables below. The position of the light platform is N51°55'05'', E03°40'02''. The shortest distance to the coast is 18 km and the platform is located 10 km south of the Eurogeul. The local depth is 22.5 m with respect to mean sea level. The direction of the coastline is approximately N40°-220°, so N310° is the direction perpendicular to the coast.

<table>
<thead>
<tr>
<th>( H_{mo} ) (m)</th>
<th>( T_{mo} ) (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.0-3.0</td>
</tr>
<tr>
<td>0 - 0.5</td>
<td>0.471</td>
</tr>
<tr>
<td>0.5 - 1.0</td>
<td>0.136</td>
</tr>
<tr>
<td>1.0 - 1.5</td>
<td>0</td>
</tr>
<tr>
<td>1.5 - 2.0</td>
<td>0</td>
</tr>
<tr>
<td>2.0 - 2.5</td>
<td>0</td>
</tr>
<tr>
<td>2.5 - 3.0</td>
<td>0</td>
</tr>
<tr>
<td>3.0 - 3.5</td>
<td>0</td>
</tr>
<tr>
<td>3.5 - 4.0</td>
<td>0</td>
</tr>
<tr>
<td>4.0 - 4.5</td>
<td>0</td>
</tr>
<tr>
<td>4.5 - 5.0</td>
<td>0</td>
</tr>
<tr>
<td>5.0 - 5.5</td>
<td>0</td>
</tr>
<tr>
<td>&gt; 5.5</td>
<td>0</td>
</tr>
<tr>
<td>( \Sigma )</td>
<td>0.607</td>
</tr>
</tbody>
</table>

Table 4-5  Frequency of wave conditions for all directions [Allersma and Ribberink, 1992]
<table>
<thead>
<tr>
<th>direction N-E</th>
<th>frequency (%)</th>
<th>wave height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wind</td>
<td>Waves</td>
</tr>
<tr>
<td>30 - 60</td>
<td>8.3</td>
<td>2.8</td>
</tr>
<tr>
<td>60 - 90</td>
<td>6.6</td>
<td>2.4</td>
</tr>
<tr>
<td>90 - 120</td>
<td>5.2</td>
<td>2.8</td>
</tr>
<tr>
<td>120 - 150</td>
<td>5.0</td>
<td>2.2</td>
</tr>
<tr>
<td>150 - 180</td>
<td>5.4</td>
<td>2.6</td>
</tr>
<tr>
<td>180 - 210</td>
<td>11.7</td>
<td>2.8</td>
</tr>
<tr>
<td>210 - 240</td>
<td>18.1</td>
<td>16.4</td>
</tr>
<tr>
<td>240 - 270</td>
<td>11.3</td>
<td>20.1</td>
</tr>
<tr>
<td>270 - 300</td>
<td>8.6</td>
<td>11.9</td>
</tr>
<tr>
<td>300 - 330</td>
<td>7.1</td>
<td>10.2</td>
</tr>
<tr>
<td>330 - 360</td>
<td>6.3</td>
<td>12.2</td>
</tr>
<tr>
<td>0 - 30</td>
<td>6.5</td>
<td>13.5</td>
</tr>
</tbody>
</table>

Table 4-6  Frequencies of wind and waves from a specified direction; characteristic wave heights per direction [Allersma and Ribberink, 1992]

### 4.2 Resulting Water Motion

The water motion due to the above described driving forces is strongly influenced by the topography of the sea bed and non-uniformity of the coast. Both the bathymetry and the shape of the coastline have been influenced by human interference. The following list gives an impression of human activity in the past [Allersma and Ribberink, 1992].

- **XVIII - XIX**: Construction of the piers of Delfland.
- **1866 - 1872**: Excavation of the Nieuwe Waterweg through the Hook of Holland.
- **1904 - 1905**: Construction of the harbour of Scheveningen with breakwaters.
- **1950**: Closure of the Brielse Maas.
- **1964 - 1971**: Construction of the Maasvlakte and the Europoort and deepening of the entrance. Beginning of the dumping of dredged material at the dumping site Loswal Noord.
- **1966**: Closure of the Brielse Gat.
- **1968 - 1970**: Extension of the breakwaters of Scheveningen.
- **1969 - 1985**: Beach nourishment at the head of Goeree (total of $6.8 \times 10^6 \text{ m}^2$).
- **1971**: Closure of the Brouwersdam.
- **1971 - 1972**: Extension of the coast at Hook of Holland (ca. $1 \text{ km}^2$, $19 \times 10^6 \text{ m}^2$).
- **1974 - 1989**: Beach nourishment at the Voorne Kust.
- **1976 - present**: Beach nourishment at Hook of Holland, Terheide and Scheveningen (total of $6.610^6 \text{ m}^2$).
- **1983**: Deepening of the Euro-Maasgeul to allow for 70' ships.
- **1985**: Deepening of the Euro-Maasgeul to allow for 72' ships.
- **1986**: Construction of the Slufterdam with a dumping site for polluted sediment.
- **1987**: Deepening of the Euro-Maasgeul to allow for 74' ships.

The construction of the Maasvlakte probably has the largest impact of these engineering works. It extends approximately five km into the sea and causes tidal current contraction. Therefore streamlines are strongly curved in the vicinity of the Maasvlakte, and high velocities occur. The flood current entering the Europoort is not only suppressed by the outflow of fresh water, but it might also be rotating, since streamlines are strongly curved. The pathlines of water particles then have the shape of a spiral. In a layer near the bottom something similar may occur in the Maasgeul. This is demonstrated in the picture below.
The scale of the longitudinal direction in this figure is much larger than that of the transverse direction. A overview of the Maasgeul area can be found in figure 4.2.

![Off shore direction](image)

Figure 4-1 Curved current over a trench

By looking at the figure 4.3 and 4.4, each of which is a detail of figure 4.2, an impression can be obtained of the offshore extension of the tidal contraction in the Maasgeul area. It shows the pathline of a water particle adjacent to the current Maasvlakte and the resulting tidal contraction. The simulation period runs from neap tide to spring tide, hence a quarter of a cycle, starting near the Maasmond, based on a 2DH simulation with wind nor waves and no Rhine outflow. The curvature of the pathline ends at approximately 8 km from the coast.

Residual currents are of importance because they may influence the residual transport. Because sediment transport is largest near the bottom, the residual currents in the lower layers are of special interest. In the report Three Dimensional Modelling of Suspended Sediment Transport in the Dutch Coastal Waters [De Kok and Salden, 1995] a four layer model is being used for the coast of Holland and Zeeland. The upper and the lowermost layer are of a constant thickness of 3.0 m. The intermediate layers both have a thickness equalling 50% of the remaining part of the water column. To take into account the effect of wind a wind, climate of 7 categories has been used. Average values have been used to take into account fresh water inflow. Residual current velocities at the surface are of the order of magnitude of 7.5 cm/s. They are mainly directed to the north and up to 20 km an offshore component is present. Near the bottom the order of magnitude of residual currents is roughly 4 cm/s. Figures 4.5 show the residual currents in the lowermost layer, in the situation with an extended Maasvlakte. Its longshore component also is directed to the north and is not as large as the cross shore component which is directed to the coast. Figure 4.5 shows a plot of the lowermost layer in the Euro Maasgeul Area. Figure 4.6 is a detail of the same plot at the coast of Delfland. Its shows the residual current in the Maasgeul to be approximately 12 cm/s in a direction along the trench, which is much larger then else where. Northerly of the Maasgeul ebb appears to dominate flood near the bottom since the residual current is directed to the south.
5 Data Analysis

This chapter is devoted to a description and interpretation of dredging data and depth measurements with respect to the Maasgeul. The purpose of the analysis is to obtain quantitative information on the morphological behaviour of the trench, which later on can be used for calibration and validation of the SUTRENCH model in chapter 6. First the general analysis of the available dredging data and depth measurements will be given. After this a specific area with a clear morphological behaviour, which is at the same time of interest for sand extraction, will be considered in more detail. The time-scale to be considered is chosen as large as possible, since our main concern is the long-term coastal behaviour.

5.1 Dredging of the Maasgeul Area

In order to ensure the required depth of the Maasgeul the trench is continuously being dredged. If the need for maintenance dredging at a certain location equals the local sedimentation, one could gain information on morphology by investigating the dredging amounts. However, not all dredging activities in the Maasgeul primarily take place for maintenance purposes. Since ships become larger and larger there is also a need for increasing the depth of both the trench and the anchoring area. Moreover, some extra dredging occasionally takes place in order to gain sand for beach nourishment or to use onshore.

The dredging data available go back to 1970 and concern the volumes that have been extracted by the dredging vessels. In appendix A, a summary of the information on dredging is given. Until 1991 it is clear what part of the total dredging was undertaken primarily for maintenance reasons. After 1991, the dredging activity has increased strongly and from the available information one can no longer distinguish between capital and maintenance dredging, or the surplus extraction that took place for commercial- or beach nourishment purposes. For that reason we focus on the period 1970 - 1991. With respect to the deepening during this period, literature and field data seem to be in contradiction. Formally, the Maasgeul should have been adjusted to ships with a draught of 70, 72 and 74 feet in the year 1983, 1985 and 1987. In effect all deepening took place in the period 1981 - 1984, respectively. Regulations on keel clearance required for 74' ships were adjusted during that period, which reduced the amount of dredging required.

Dividing the total volume dredged for deepening by the total area of the Maasgeul provides an indication of how much draught is gained on average during the early nineteen eighties. The total of dredged mass of $8.88 \times 10^6$ m$^3$ divided by the total area of the 600 m wide Maasgeul plus the 800 m radius anchoring area produces an average gain in draught of 1 m (3'3" or 3.25 ft.). No data are available on dredging during the early seventies. The graph below shows "dredged volumes for maintenance" as a function of time.
Figure 5-1  Annual dredging in the Maasgeul; red stripes indicate the periods in which draught raising took place.

Considering the extreme 'maintenance dredging' in 1972 most of the deepening to adjust the trench to 68' probably took place in 1972 and 1973. As sedimentation in a trench is a function of its depth, we distinguish between a 68 and a 74 feet period, respectively corresponding to 1972 - 1980 and 1981 - 1990. Within the two periods a rather high variability can be observed, as well as a decreasing trend. This observation might be the result of over dredging, but this is not very probable, since the dredging companies do not benefit from dredging more than necessary to attain the required depth. They do not get paid for the surplus they extract, whence it would go at the expense of future work. The larger part of the observed variability must therefore be a result of varying sedimentation in the trench.

Over the period 1982 to 1987 it has been registered how the extracted amounts were distributed over the total length (i.e. per km). In order to obtain an indication of the distribution of maintenance dredging we need to know when and where draught-raising and other non-maintenance sand extraction took place. Comparing data on total dredging with the sum of required maintenance and draught-raising dredging it occurs that for the years '82, '83, '86 and '87 total dredging is slightly less than the sum of maintenance and draught-raising, which can not be the case. The difference however is relatively small, and might have an administrative reason. When looking at the dredging data given in appendix A it is not likely that the extraction for draught raising was equally distributed over the length of the Maasgeul, for instance because there has not been any deepening from km 11.0 to 11.3. Therefore we do not use the years in which deepening took place, to calculate the distribution of maintenance dredging in the Maasgeul. The remaining years are 1985, 1986 and 1987. Over these years we calculate the average maintenance dredging volumes in the Maasgeul, per km of its length. Results are graphically demonstrated on a normal and a log-scale to investigate the variability over km 5-11.3 in figures 9.2 and 9.3.
Figure 5-2  Distribution of maintenance dredging in the Maasgeul

\[ y = 0.0067x^6 - 0.2971x^5 + 5.4076x^4 - 52.177x^3 + 285.74x^2 - 864.32x + 1202.7 \]
\[ R^2 = 0.996 \]

Figure 5-3  Distribution of maintenance dredging in the Maasgeul (log plot)

Because of the lack of data to verify the total extraction in 1986 there might have been supplementary dredging. However, this is not very likely since this did not occur in 1985 nor in 1987. The best-fit trendline turns out to be a sixth order polynomial, as indicated in figure 5.2, with regression factor of 0.996.

Rijkswaterstaat (Directorate-General for Public Works and Water Management) pays dredging companies for their services on the basis of tons of dry matter. Therefore they need to convert the measured dredged volumes to tons of dry matter. The conversion factor used is based on samples and varies for different locations as both porosity and density of the sediment vary. This conversion factor is of interest for a morphological analysis of the Maasgeul area since it is an indicator for the type of sediment at a specific location. The conversion factors used are tabulated below.

<table>
<thead>
<tr>
<th>Location</th>
<th>Conversion Factor ([10^3 \text{ kg/m}^3])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maasmond</td>
<td>0.567</td>
</tr>
<tr>
<td>Maasgeul km 0-5</td>
<td>Loswal North[1.277]</td>
</tr>
<tr>
<td></td>
<td>Beach Nourishment Scheveningen[1.572]</td>
</tr>
<tr>
<td>Maasgeul km 5-11.3</td>
<td>1.621</td>
</tr>
</tbody>
</table>

Table 5-1 Conversion of dredged volumes to tons dry matter [source: Rijkswaterstaat, directie Noordzee]
Hence the density of a cubic meter of dredged material increases as the distance from the coast increases. The differences can be explained by the presence of silt near the coast, which is of lower density than sand. Dredged bulk in the 'Maasmond' is extremely silty whereas after some 5000 m it will have gradually turned into almost pure sand. These 5000 m represent the transitional zone between the Maasmond, where the dredged material consists of only silt and the area where the presence of silt can be neglected. An amount of quartz sand with a usual porosity of 0.4 and a specific density of 2650 kg/m³ would have a conversion factor of 1.590. If we consider a conversion factor exceeding 1.590 as an indication for a region to be free of silt, we can estimate to what extent silt is present in the Maasgeul. Based on averages over the period 1990 - 1995, 4% of all dredged bulk from km 0 to km 5 is suitable for beach nourishment at Scheveningen [source: Rijkswaterstaat, directie Noordzee]. With that, the average conversion factor in this sector is 0.04 * 1.572 + 0.96 * 1.277 = 1.289. We model the conversion factor as a function of the distance from the Noorderdam as has been visualised in the following graph.

Conversion of m³ of Dredged Material to Tons of Dry Sediment

Figure 5-4 Volume to weight conversion of dredged material in the Maasgeul

The point at which the conversion factor exceeds 1.590 10³ kg/m³ corresponds to km 4.56. Offshore this point very little silt will be present. There, one can presume the porosity of dredged material to be equal to that in the bed of the Maasgeul.

5.2 Analysis of Depth Measurements

Within the Directorate-General for Public Works and Water Management, which is part of the Ministry of Transport, Public Works and Water Management, there is a department dedicated to the Dutch part of the North Sea, named 'Directie Noordzee'. As to the depth measurements in the area of concern, information older than six years is removed from the database. During the period 1990-1996 there were 70 surveys; 35 for each of the two parts in which the trench has been divided for administrative purposes. The dates of the surveys are listed below.
Morphological Impact of Large Scale Marine Sand Extraction

Maasgeul km 0 - 5
1990: 27/3 4/5 24/7 6/11
1991: 15/1 5/3 2/7 29/8 12/11 29/11
1992: 14/1 1/4 3/7 6/12
1993: 5/2 5/3 12/5 13/9 17/11
1994: 25/2 7/4 1/6 22/9 11/10
1995: 6/1 31/3 27/6 28/7 5/10 7/12 29/12
1996: 12/3 6/5 31/7 10/10

Maasgeul km 5 - 11.3
1990: 19/3 11/5 11/7 26/10
1991: 21/1 12/3 21/6 2/9 30/10 6/12
1992: 27/1 26/3 10/7 15/12
1993: 2/2 5/3 19/5 16/9 24/11
1994: 17/2 8/4 15/6 26/9 20/10
1995: 25/1 6/4 30/6 21/7 17/10 21/12
1996: 12/1 21/3 15/5 18/7 24/9

*multibeam measurement; all others are single beam measurements

Table 5-2 Dates of the used surveys [source: Rijkswaterstaat, Directie Noordzee]

During each survey, ships cover the Maasgeul area with tracks with an intermediate distance of 50 m. These tracks have an east-west direction. The intensity of the measurements between surveys varies. Using a multi-beam-system, depths over a width of four to five times the water depth are being measured. In case of the Maasgeul surveys, the average resolution acquired is one measurement per five by five meter. The single beam system measures vertically beneath the ship only. For this method, the frequency of measurement varies. However, on the larger part of a track the spatial frequency can be approximated by a measurement per five meter. Hence, compared to single-beam measurements, the multi-beam system has a higher resolution.

To be able to make an accuracy estimation, it is of interest to know how well depth measurements can be compared. In literature, the relative accuracy of depth measurements is referred to as the repeatability of measurement. [Hemelrijk et al., 1978] calculate this repeatability as the sum of a stochastic and a systematic fault. The former depends on equipment, the type of ship and navigation speed. For single beam and multi-beam measurements a simple calculation based on practical experience results in a reliability of 0.44 dm and 0.32 dm, respectively. Systematic faults are a result of the error in positioning and apply for the whole survey. For both single beam and multi-beam measurements, reliability has been calculated to equal 0.71 dm.

Apart from the anchoring zone the area of measurement has a width of 1100 m. The width of the trench is approximately 500 m, which leaves 340 and 260 m for respectively the southern and the northern slope. At first sight it seems that the surveys cover at least the larger part of both the northern and the southern slope and always the same area. In order to find out how this frequently measured area fits into its surroundings the samples used for the bottom file of the 'Rijmammo model' have been analysed. This model covers the whole Dutch part of the North-Sea and is based on a large number of surveys. Those surveys took place around 1990, in a period of approximately two years. The two pictures below demonstrate the extent of the Maasgeul at a distance of 4.65 and 6.50 km from the end of the Noorderdams, respectively.
The survey of the Maasgeul that fitted best to the Rijmamo samples happened to be the ones taken in May and March 1990 for km 0 to 5 and 5 to 11.3, respectively. Apparently the surveys include all of the southern and most of the northern slope. A cross-section of a survey in 1995 has been plotted in the pictures to stress that the missing part of the northern slope is of interest from a morphological point of view.
5.2.1 Depth of the Maasgeul

On the basis of the depth measurements a check can be made to see whether the target depth is achieved. Simultaneously the deepening of the trench can be investigated. Figure 5.7 shows the depth of the dredged part of the Maasgeul, that is without slopes, during the period 1990 - 1996.

![Depth of the Maasgeul in dm](image)

Figure 5-7   Depth of the Maasgeul in dm

Until 1994 no severe deepening takes place. Halfway the Maasgeul the bed level decreases from 25 to 26 m, which is in conformity with the target depth. A trough, or some troughs, on the northerly side near the coast disturb this picture. In the figure 5.7, they are only partly visible. Locally their depth reaches up to 30 m, which is 5 m below their environment. Apart from this the toe of the southern slope on the coast near half of the trench seems to have a higher level then its environment. The excavation must have begun somewhere in 1994 and globally extends from km 6 up to km 11.

5.2.2 General Description

In figure 4.2 the bathymetry of the coastal area of the two southermost coastal provinces of The Netherlands is presented (Zeeland and Zuid Holland). It shows the Maasgeul to be situated in a complex area which can not be considered uniform. The cross shore slope in the surroundings of the Maasgeul becomes smaller in seaward direction, whereas an existing long shore slope increases as far as the Maasgeul reaches. Because of the irregularity of the coastline it is difficult to determine the coast parallel direction at deeper water. Nevertheless it is clear that the depth contours both south and north of the Maasgeul slightly deviate from the longshore direction in a clockwise direction.
Navigation channels in a tidal area mostly have slopes with the shape of an S-curve. However, examining the slopes of the Maasgeul, one can notice that all slopes have a fairly rectilinear segment, similar to the slopes as depicted in the figures 5.5 and 5.6. The change of gradient of this segment is of interest because, apart from the dredging that sporadically takes place on the slopes, it is the result of natural processes. Therefore average gradients of both the northern and the southern slope have been calculated for 1990, 1993 and 1995 (beginning, middle and end of the available data period), per 25 m in the Maasgeul. The software used was RGF Grid for the creation of grids and Quick Inn visualisation of bathymetries, both from Delft Hydraulics, and the spreadsheet program MS Excel. A description of the former two programs is included in the appendix B.

Calculations were done according to the following procedure.
1. Two grids were created with a resolution of 25 by 25 m and an orientation along the Maasgeul, the first one covering the complete southern slope and the second the northern slope up to the anchoring area.
2. Averaging per grid cell was computed from the samples.
3. To determine the bed level gradient of the northern and the southern slope:
   - The straight part of the northern and the southern slope could be covered with four and three grid cells, respectively (in trench-transverse direction). The gradient of each part for these four or three grid cells was calculated using a 'linear least squares' method.
   - These resulting transverse gradients are depicted against the distance in the Maasgeul in figures 5.8 and 5.10. A moving average filter was applied to obtain a clearer picture.
4. To determine the erosion and sedimentation at the northern and southern slope:
   - The differences between the three snapshots (1990, 1993 and 1996) of both the northern and the southern slope were used to determine erosion and sedimentation.
   - Averaging in transverse direction was done over a width of 125 m and 225 m for the northern and the southern slope, respectively.

5.2.3 Dynamic Behaviour of the Southern Slope

The moving averages of the gradients and the sedimentation/erosion of the southern slope are depicted in figure 5.7 and 5.8, respectively. The gradients of the rectilinear segments vary from 8% near to the coast to 2% at the end of the Maasgeul and decreases fairly monotonically in between. From figure 5.9 it can be observed that a remarkable increase in gradient occurs from kilometre 7 to 10 between 1990 and 1996. This can be either the natural response to the sea sand extraction in that segment during early 1990's, or the result of dredging on the slope. The sedimentation at km 7 to 8, which is in the order of 4 dm, makes the first explanation more plausible since it cannot be explained from dredging activity. When looking at the behaviour of the gradients of km 0 through 6 of the southern slope, there seems to be an equilibrium, or at least a situation where no clear increase or decrease can be noticed. Simultaneously there is a slight tendency of erosion, especially in the first five kilometres.
5.2.4 Dynamic Behaviour of the Northern Slope

The first 3200 metres of the northern slope are very irregular due to the presence of deep troughs. Therefore the calculated gradients and sedimentation/erosion there, are not very accurate, and we will focus on the section between the troughs and the anchoring area. The summary plots, analogous to those of the southern slope, are demonstrated below. Gradients do not exceed 0.5% and with that are very small compared to those of the southern slope. Erosion is much more apparent and reaches one metre in six years at km 4.4. Offshore of this point the erosion and the decrease of the slope gradually decreases. Considering that the sand extraction was negligible up to km 6, the observed development will be tide-induced. There is little change in gradients and relatively low erosion between km 3.3 and 3.9. This could be a region with relatively low velocities between the area where the tidal current runs practically perpendicular to the Maasgeul and the strongly curved tidal current that warps around the Maasvlakte.
5.3 Tracks to be Modelled

Combining results of the analysis of hydrodynamic pathlines with the above described behaviour of the Maasgeul, it seems possible to some extent to schematise the water motion to a 2DV situation seaward of km 4.4 onwards. Four tracks will be defined, at km 4.65, km 5.25, km 5.88 and km 6.50, all in a direction parallel to the streamlines and with a length of 4000 m.

The depth along the tracks is illustrated in figure 5.11 and stem from the RIJMAMO bathymetry. The direction of the tracks corresponds with the approximate mean tidal direction, derived from the pathlines from figure 4.3. The measured mean tidal direction deviates 17° counterclockwise from an axis perpendicular to the trench.
Figure 5-12  Definition of trajectories which will be the starting point of the SUTRENCH analysis

In order to obtain detailed information about the morphological development of the local slopes of the trench, for each of the four tracks a 1125 × 200 m grid was used, with its longer axis perpendicular to the trench. The resolution of the grids once again is 25 by 25 meters. The width of 200 m has been chosen with the intention to filter the data from local disturbances which either results from transverse processes or from errors in the depth measurements, while keeping it as small as possible. After grid cell averaging per measurement, per track, all areas have been reduced to a single line by averaging in transverse direction.

Three segments have been defined, located on these cross sections of 1125 m length. The southern bank, southern slope and northern bank will be represented in the table below. For each segment the data were averaged in longitudinal direction, resulting in a single value per segment per track.

<table>
<thead>
<tr>
<th>Track</th>
<th>Southern Bank</th>
<th>Southern Slope</th>
<th>Northern Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Track 1</td>
<td>m 1447 - m 1651</td>
<td>m 1682 - m 1886</td>
<td>m 2645 - m 2749</td>
</tr>
<tr>
<td>Track 2</td>
<td>m 1472 - m 1676</td>
<td>m 1707 - m 1911</td>
<td>m 2670 - m 2774</td>
</tr>
<tr>
<td>Track 3</td>
<td>m 1487 - m 1691</td>
<td>m 1722 - m 1926</td>
<td>m 2685 - m 2789</td>
</tr>
<tr>
<td>Track 4</td>
<td>m 1506 - m 1710</td>
<td>m 1741 - m 1945</td>
<td>m 2704 - m 2808</td>
</tr>
</tbody>
</table>

Table 5-3  Geographical definition of segments of the trajectories in figure 5.12

In figure 5.13 cross sections of the trench have been plotted with a distance to the coast that correspond to the four tracks. Figures 5.14 depicts the time evolution of the southern bank and the southern and northern slope. Apart from a slight erosive tendency of 1 and 2 dm in six years for km 4.65 and km 5.25, respectively, the southern bank appears to be reasonably stable. On the southern slope deposition takes place until the beginning of 1994. Afterwards a strong erosion is found which becomes larger in off-shore direction. The latter development must have been induced by the deepening, which is larger offshore, whereas the former tendency must correspond to the situation with a constant depth of 26 m. We will focus on the situation before 1994. The northern slope systematically erodes in all
cases. The effect of the deepening on the northern slope is probably negligible, since there is no significant change in deposition rate in 1994. The rates of deposition and erosion are tabulated below.

<table>
<thead>
<tr>
<th></th>
<th>Northern slope</th>
<th>Southern slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>km 4.65</td>
<td>-13.50 cm/year</td>
<td>25.92 cm/year</td>
</tr>
<tr>
<td>km 5.25</td>
<td>-7.77 cm/year</td>
<td>10.95 cm/year</td>
</tr>
<tr>
<td>km 5.88</td>
<td>-5.48 cm/year</td>
<td>9.13 cm/year</td>
</tr>
<tr>
<td>km 6.50</td>
<td>-2.19 cm/year</td>
<td>5.84 cm/year</td>
</tr>
</tbody>
</table>

Table 5-4 Average erosion (-) and deposition (+) on the slopes of the Maasgeul within the period 1990 - 1996

The observations are consistent with the general description. Nevertheless it is surprising that there is deposition of material on southern slope, while the bed level gradient does not change. This is an indication that its behaviour is analogous to that of a shock wave, forced at a fixed position by dredging. Probably the most important conclusion which can be drawn is that the northern slope systematically erodes at the above mentioned rates, and behaves like a migrating expansion wave. Together with the dredging data, the information will be used to calibrate the SUTRENCH model.
6 Modelling Morphological Alongshore Developments with SUTRENCH

This chapter will be devoted to a prediction of long-term alongshore development, as a result of deepening and/or widening of the Maasgeul at deep water, offshore the NAP -20 m line. To this purpose, the non-equilibrium 2DV sand transport model SUTRENCH will be applied to a number of alongshore tracks. SUTRENCH solves the basic sediment equation, calculating flow velocities on the basis of an empirical flow model named Profile, adapted to a situation with waves. A short description of the SUTRENCH model can be found in appendix C.

First the general set up of the model will be given, including the input data (tidal conditions, waves, physical parameters, etc.). Then, the numerical aspects will be treated. Preparatory to long term simulations, adequate values for all parameters and variables in the model will be determined by applying it to the track at km 4.65 offshore (MG4.65), as described in the data analysis. For this trajectory, the field data, described in chapter 5, can be used to calibrate the model. After small modifications, the resulting model can be applied to adjacent longshore tracks. The performance of the model with the calibrated parameter values was investigated, by applying it to three adjacent tracks. With this, a calibrated model has been obtained of which it is known how to what extent it can be applied to nearby tracks.

Subsequently, the model will be applied to three 15 km tracks. In order to be able to perform long-term computations at reasonable computer cost, it is necessary to schematise boundary conditions further. The purpose of the long term simulations is to investigate the alongshore effect of the sand extraction scenario on its surroundings, specifically on oil and gas pipelines, future maintenance of the Maasgeul and erosion and sedimentation rates in relation to ecology.

6.1 General Set Up and Input Data

The SUTRENCH model requires discrete information on all physical input. The input will be grouped in three categories. Firstly there is a part, to be referred to as physical parameters, which can be chosen in a straightforward manner and is held constant during all computations. Subsequently there is wave and tide related input that needs to be formulated in a limited number of representative conditions. For a given percentage of occurrence of these conditions, a single wave and tide scenario can be built. The third part of the input consists of variables which cannot easily be identified, either because of the ever changing conditions, or by a lack of information. Therefore they are used as calibration parameters.

The trajectories, as described in the data analysis, have the orientation of the predominant tidal direction. Since the orientation of the trench deviates approximately 17° from a direction perpendicular to the tide, streamline refraction on the upstream and downstream side slopes will occur and the direction of pathlines in the intermediate area will differ. In paragraph 6.1.4 a description of the phenomenon is given and of how it can be dealt with.
6.1.1 Physical Parameters

The following input data are directly taken from literature [Van Rijn, 1987]:

- sediment density, $\rho_s = 2650$ [kg/m$^3$]
- fluid density, $\rho = 1025$ [kg/m$^3$]
- porosity of bed material, $\alpha = 0.4$ [-]
- constant of Von Karman, $\kappa = 0.4$ [-]
- angle of natural repose, $\phi = 35$ [$^\circ$]
- acceleration of gravity, $g = 9.81$ [m$^2$/s]
- kinematic viscosity, $\nu = 1 \times 10^{-6}$ [m$^2$/s]

Soil samples have led to the following data on the grain diameter of sediments in the area of the Maasgeul [Van der Gouwe, 1990].

<table>
<thead>
<tr>
<th>Location</th>
<th>Variation [μm]</th>
<th>Average [μm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maasgeul 0-5 km.</td>
<td>155-250</td>
<td>175</td>
</tr>
<tr>
<td>Maasgeul 5-11 km.</td>
<td>142-345</td>
<td>210</td>
</tr>
</tbody>
</table>

Table 6-1: Particle diameters of bed material [Van der Gouwe, 1990]

The survey area corresponds for the larger part with Maasgeul 5-11 km. $D_5$, $D_{50}$ and $D_{95}$ correspond respectively with 142, 210 and 345 μm. In the SUTRENCHE-MODEL the particle dimensions are represented by $D_{50}$ and $D_{90}$. We calculate $D_{90}$ by assuming the logarithm of the grain size to be normally distributed. The value of $D_{95}$ then can be fitted to the error function by iteration:

$$0.95 = \frac{1}{2\sqrt{\pi}} \int_{-\infty}^{x} e^{-(ax)^2} \, dx \Bigg|_{x=\log D_{95} \rightarrow \log 345} \Rightarrow a = 5.373$$

Consequently $D_{90}$ can by calculated, once again by iterating:

$$0.90 = \frac{1}{2\sqrt{\pi}} \int_{-\infty}^{x} e^{-(5.373x)^2} \, dx \Bigg|_{x=\log D_{90}} \Rightarrow D_{90} = 309\mu m$$

6.1.2 Representation of Tidal Conditions

The sea surface is a random collection of waves of various shapes and sizes. A formal mathematical representation of this surface uses the harmonic progressive wave train as a building block and expresses the sea surface as a combination of a finite number of these component waves, each with its own amplitude, frequency and direction. Hydrodynamic models are being calibrated by making use of harmonic analyses of water level measurements at numerous measuring stations. For the Dutch part of the North Sea, the Rhine Maas Model (RIJAMAMO) has been developed. The 2DH, hydrodynamic model will be used to generate representative tidal conditions at the beginning of the trajectory MG4.65.
The model was run for a time series of 13 days, from neap tide on February 20th, 1994, to spring tide on March 3rd, 1994. This yielded the water levels, velocities and corresponding directions in the grid points nearest to the origin of the km 4.65 track. Neither wind nor river outflow conditions were implemented in the model: only the astronomical tide was calculated. Fourteen of the approximately 57 tides within a spring- neap cycle suffice for an adequate representation. This series of fourteen tides begins with the tide with the smallest amplitude and ends with the largest amplitude tide, and starts at slack tide.

One could argue that it would be better to implement an average river outflow. However, in the 2DH situation, incorporating river output will mainly contribute to the cross-shore component of velocities, which would not lead to an improvement of the longshore current. The effect of wind will be taken into account by adding a variable residual velocity during the SUTRENC calculations.

The depths in the grid point, nearest to the origin of the track were respectively 19.35 m and 18.68 m. The difference can be taken into account by calculating flow velocities as the discharge resulting from the RJJMAMO calculation divided by the water depth at the origin of the track. The water level during the simulated period to be utilised has been plotted below.

![Graph showing water levels from neap tide to spring tide at the origin of track MG4.65](image)

**Figure 6-1** Water levels from neap tide to spring tide at the origin of track MG4.65

The mean tidal direction will be estimated on the basis of the float tracks, or pathlines, that have been used to determine the scope of influence of tidal flow contraction at the Maasvlakte (figure 4.4). The resulting direction is N38.5°E. Flow velocities appear not to be entirely unidirectional. Figure 6.2 shows velocities relative to the local grid, which has the orientation N23.8°W.
The dashed line indicates the mean direction. On the basis of flow velocities, the mean tidal direction equals N39.1°E. A co-ordinate transformation is applied in order to determine the velocity component in the mean flow direction. An illustration has been given in figure 6.3. The dashed line in figure 6.2 corresponds to the \( u' \) axis in figure 6.3. The equation of the applied transformation reads:

\[
u' = v \sin \alpha + u \cos \alpha
\]  

(6.1)

The resulting velocity amplitudes during flood increases from 0.54 m/s at neap tide to 1.02 m/s at spring tide. They are plotted against time in figure 6.4.
In the SUTRENC model, the tide has to be represented by 20 time steps at maximum. The duration of a condition can be variable. Two tides will be chosen and each of them will be discretised in 10 time steps of equal length. Subsequently, a percentage of occurrence of each of the two tides will be derived from the condition that the net transport has to be reproduced.

In order to find out which two tides would be most representative, the tidal amplitude during a spring tide - neap tide cycle is plotted against time in figure 6.5. The amplitude of tides at 1/16 period and at 5/16 period are plotted in the same figure. Assuming an equal percentage of occurrence, these two tides will best represent the spring tide - neap tide cycle, since the average deviation from the exact amplitude will be the least.

The two tides are schematized further by means of linear interpolation, enlarging the time step from 40 minutes to 100 minutes. It is important that the net transport during an individual tide, as well as during a neap tide - spring tide cycle remains unchanged. Therefore, the two tides will be scaled up to satisfy the former constraint and the percentage of occurrence of the two tides will differ, to satisfy the latter. In this stage, the sediment
transport is calculated using the Bijker transport formula. It would be more consistent to use a method analogous to that of SUTRENCH. However, such formulae are far more complicated and cannot be programmed easily.

The Bijker sediment transportation formula is explained in appendix D. As to its application, a few remarks should be made. In order to calculate the square of the near-bed orbital velocity amplitude, linear wave theory will be used. The following formula has been applied to the 33 wave conditions from the Light Island Goeree:

$$\hat{u}_0^2 = \sum_{i=0}^{33} \alpha_i \left( \frac{\omega_i H_i}{2 \sinh(k_i h)} \right)^2$$

(6.2)

in which:

- $\hat{u}_0$ = maximum horizontal velocity component just outside the boundary layer [m/s]
- $\omega_i$ = wind wave frequency (=$2\pi/T$) [s$^{-1}$]
- $\alpha_i$ = percentage of occurrence during condition i [-]
- $H_i$ = wave height during condition i [m]
- $k_i$ = wave number during condition i, calculated on the basis of the dispersion relationship:

$$\frac{\omega_i}{k_i} = \sqrt{\frac{g}{k_i}} \tanh(k_i h)$$

(6.3)

Water level variation influences the magnitude of $\hat{u}_0$ both directly and indirectly via the wave number $k$. This influence will not be taken into account though, because a formula is needed which can be applied without having to carry out an iteration procedure.

For the bed roughness, $r$, the value 0.05 m is chosen. The Einstein integrals are tabulated in various handbooks. Once again, water level variation will not be taken into account. Then, the following approximation can be derived for the Einstein integral value $Q$ (from formula A7 in appendix C), by fitting the tabulated data to an exponential curve:

$$Q_{h=19.35 m, r=0.05} = 1 + 183[I_1 \ln \left( \frac{33h}{r} \right) + I_2]_{h=19.35 m, r=0.05} = ((z* + 1)^{-7.1796}) * 952.74$$

(6.4)

in which:

- $Q$ = Einstein integral factor [-]
- $I_1, I_2$ = Einstein integrals [-]
- $z*$ = Rouse number [-]

Other input parameters are chosen according to paragraph 6.1.1.

The net transport during neap tide appears to be -396 m$^3$/year, hence, surprisingly, in a southerly direction. The net transport during the representative spring tide equals 392 m$^3$/year. Velocities after reducing the number of time steps are being scaled up (by
multiplication with a constant factor) until the net transports coincide with the situation before schematization.

Not only does the net transport during a tide have to remain unchanged, the net transport during a spring tide neap tide cycle should not change, either. Once again the Bijker transport formula has been applied for each of the 40 minute time steps. The net long shore transport than can be calculated by averaging, and appeared to equal 224 m$^3$/m year. The percentage of occurrence now can be calculated as follows:

\[
\begin{align*}
\alpha_s 392 - \alpha_n 394 &= 224 \\
\alpha_s + \alpha_n &= 1
\end{align*}
\]

\[
\alpha_s = \frac{224 + 394}{392 + 394} = 0.786; \quad \alpha_n = 0.214
\]

$\alpha_n$ = fraction of occurrence of the representative neap tide
$\alpha_s$ = fraction of occurrence of the representative spring tide

The tidal boundary condition at the origin of track MG4.65 is known now. The resulting tide-induced residual velocity amounts 1.7 cm/s to the south. Instead of performing an analogous operation for nearby tracks, new tidal boundary conditions are calculated by adapting the calculated velocities. The surface elevation will not substantially vary in place. Therefore, the surface elevation boundary condition is held the same for all tracks. In the longshore direction, velocities can be calculated by presuming bed level changes to be small and local. Then, the discharge along the longshore pathlines will be constant. In the cross shore direction this would not be correct, since the water depth globally increases off shore. Assuming a constant longshore gradient in surface elevation, quasi-steady flow and a homogeneous bed topography, the velocity increase between point 1 and 2 on the same coastal transect, can be approximated using Chézy's law:

\[
\frac{v_1}{v_2} = \frac{C\sqrt{a_1 I}}{C\sqrt{a_2 I}} = \frac{a_1}{a_2}
\]

From the origin of track 1 to the origin of track 4, the depth increases from 18.68 to 20.13 m. Since this difference is small, this simplistic approach suffices.

### 6.1.3 Representation of Wave Conditions

The wave climate as described in the analysis of hydrodynamic conditions will be used (table 4.5). Therefore, the values for $H_{\text{mo}}$ and $T_{\text{mo}}$ have to be translated into significant wave height, $H_s$, and peak period, $T_p$, respectively. The next empirical relationship will be used, valid for conditions at the Northsea [Bosboom et al, 1996].

\[
T_p = 1.36T_{\text{mo}}^2 - 0.45
\]

A relationship between $H_{\text{mo}}$ and $H_s$ can be found the lecture notes on wind waves by Battjes [Battjes, 1992]:

\[
\begin{align*}
H_{\text{mo}}^{1/3 \text{empirical}} &= H_s = 3.8\sqrt{m_0} \\
H_{\text{mo}} &= 4\sqrt{m_0}
\end{align*}
\]

\[
H_s = 0.95H_{\text{mo}}
\]
in which \( m_0 \) is the surface of the wave energy spectrum. In the SUTRENCH model, the wave direction can only be varied within a tide and not between tides. A test calculation for the trajectory at km 4.65 of the Maasgeul shows that the influence of the wave direction in SUTRENCH can be neglected at deep water. Therefore, the wave direction is held constantly equal to the mean tidal direction during flood. Within a modelling period of four months, all wave conditions occur once, during a time interval that is calculated on the basis of its percentage of occurrence.

### 6.1.4 Steering parameters

Apart from physical constants and tidal and wave conditions, the model needs input on a number of parameters which can not be easily measured in nature. Three of these parameters will be discussed briefly and an estimate of their order of magnitude will be given. They are summarised and their final value is given in table 6.3.

The hydraulic roughness of a bed varies not only with sediment characteristics but also with the shape of the bed. In river engineering, flow conditions are being classified in a lower, a transitional and an upper flow regime, each of the categories with its own class of bedforms. Conditions at sea, with continuously varying flow velocities and the wave induced orbital motion near the bottom, are entirely different, and the expertise from river engineering cannot be used easily. In the SUTRENCH model, the hydraulic bed roughness is being represented by the coefficients \( r_{\text{wave}} \) and \( r_{\text{current}} \).

Both the wave-related as the current-related bed-roughness influence the effective bed shear stress. The velocity profile as well as the near-bed concentrations are enlarged. Hence, a higher roughness implies higher shear stresses and therefore a higher transport rate. The order of magnitude of both parameters is \( 10^{-2} \) m, with a minimum of \( 3 D_{50} \) in case of a flat mobile bed.

The reference level up to which sediment transport is considered to be bed-load, \( z_{\text{ref}} \), influences solely the suspended sediment transport in case of the SUTRENCH model. The equilibrium bed concentration is inversely proportional to this reference level. It is desirable to apply a bed-boundary level close to the bed, because then the approximation of an instantaneous adjustment to equilibrium conditions close to the bed is more valid. Since \( z_{\text{ref}} \) represents no physical quantity, solely a modelling one, it is a steering parameter par excellence.

For sediment diameters in the range from 50 \( \mu \)m to 300 \( \mu \)m, in water of 18\(^{\circ}\)C, the fall velocity, \( w_f \), can be approximated by the following empirical formula [Van der Velden, 1995]:

\[
\log \left( \frac{1}{w_f} \right) = 0.4758 (\log D_{50})^2 + 2.1795 (\log D_{50}) + 3.1915
\]  

(6.9)

Hence, it exclusively depends on \( D_{50} \). Since in the Maasgeul \( D_{50} \) varies between 155 and 210 \( \mu \)m, the particle fall velocity will vary between 1.61 and 2.45 cm/s. Instead of varying only the fall velocity, \( D_{90}, D_{50} \) and \( w_f \) will be varied simultaneously, satisfying equation 6.9.

The sediment mixing coefficient is related to the fluid mixing coefficient as follows:
\[ \varepsilon_s = \beta \phi \varepsilon_f \]  \hspace{1cm} (6.10)

in which:

\[ \varepsilon_s = \text{sediment mixing coefficient} \quad [\text{m}^2/\text{s}] \]
\[ \varepsilon_f = \text{fluid mixing coefficient} \quad [\text{m}^2/\text{s}] \]
\[ \beta = \text{ratio sediment mass mixing and fluid momentum mixing coefficients} \quad [-] \]
\[ \phi = \text{turbulence damping factor} \quad [-] \]

The \( \beta \)-factor represents the difference in the mixing of a fluid particle and a discrete sediment particle and is assumed to be constant over the water depth. The \( \phi \) factor expresses the damping of the turbulence by the sediment particles resulting in a reduction of the fluid mixing coefficient. Both parameters will be kept at a constant value of 1 throughout the computations.

The cross-section integrated sediment continuity equation is discretised using the following numerical scheme (LAX-scheme):

\[ z_{b,x}^{t+\Delta t} = z_{b,x}^{t} - \frac{N \Delta t}{2(1-p) \rho_s b \Delta x} \left( S_{s,x}^{t} - S_{s,x-\Delta x}^{t} \right) + \frac{1}{2} \gamma_s \left( z_{b,x+\Delta x}^{t} - 2z_{b,x}^{t} + z_{b,x-\Delta x}^{t} \right) \]  \hspace{1cm} (6.11)

The \( \gamma_s \)-factor in equation 6.11 determines to what extent the bed levels of the surrounding points of \( z_{a,x} \) are taken into account for the computation of the new bed level \( z_{a,x} \) at time \( t+\Delta t \). This causes numerical smoothing at sharp transitions of the bed level profile, which can be justified physically by the occurrence of slope effects in reality. This is a rather crude way of incorporating slope effects though, and it is questionable if calibration by making use of the \( \gamma_s \)-factor leads to better predictions. Therefore, the \( \gamma_s \)-factor is kept as small as possible.

### 6.1.5 Streamline Refraction

Since the Maasgeul is orientated approximately 17° oblique to the flow direction, streamline refraction on the upstream and downstream sides of the trench will occur. The effect of streamline refraction is threefold:

1. velocities in the trench will increase since the distance between streamlines will decrease
2. the tidal path in the trench will be enlarged as the obliqueness of the flow relative to the trench increases
3. a pathline on the northern side of the Maasgeul will run approximately parallel to the same pathline on the southern side, but will be shifted in a coastward direction

The effect of an increase of velocities can be neglected for an angle of approach smaller than 20° [Van Rijn and Tan, 1985]. In order to find out to what extent the tidal path in the trench will be enlarged, it is necessary to know the orientation of the flow in the trench. Since flow velocities constantly change, the orientation also will vary. Predicting stream line refraction is difficult and different approaches to approximate current conditions lead to different relationships between the propagation direction inside and outside the trench. In this report, the tidal current is presumed to be dominated by inertia (which is debatable). Then, the average streamline refraction, can be calculated using Snell’s law, and applying the shallow water approximation for the tidal wave celerity:
\[
\frac{\sin \phi}{\sin \phi_0} = \frac{c_0}{c_1} = \frac{\sqrt{gh_0}}{\sqrt{gh_1}} = \sqrt{\frac{h_0}{h_1}}
\]

in which:

- \(\phi\) = angle between the wave propagation direction and an axis perpendicular to the trench [°]
- \(c\) = wave celerity outside the trench [m/s]
- \(g\) = acceleration of gravity [m/s²]
- \(h\) = water depth [m]
- \(0\) = index for the situation inside the trench
- \(1\) = index for the situation outside the trench

For track MG4.65, the water depth increases from 19 m to 26 m where the flow enters the trench. Using the above stated approximation, the angle between the wave propagation direction and an axis perpendicular to the trench, \(\phi\), increases from 17.0° to 20.0°. The Maasgeul, as it formally defined by Rijkswaterstaat, has a width of 500 m, in which the slope is not included. The width of the Maasgeul relative to the flow, in a situation without refraction, equals 500/cos(17.0°) = 523 m. As a result of refraction this width increases to 500/cos(20°) = 532 m. A specific pathline northerly of the Maasgeul is situated 523*sin(20.0°-17.0°) = 27 m closer to the shore than the same pathline south of the trench. The cross shore bed level gradient in the area north of the Maasgeul equals 0.21%. Hence, the decrease in depth along a pathline before and after the trench will be in the order of magnitude of 6 cm. This will not severely affect the transport capacity.

For the other tracks, the Maasgeul is shallower relative to its surroundings. Therefore, streamline refraction will be even less important. When the Maasgeul will be broadened and deepened though, the influence will increase. Assuming a new depth and width of 31 m and 2700 m, respectively, the angle between the wave propagation direction and an axis perpendicular to the trench increases to 21.9°. Performing a similar calculation as has been done for the present situation, the pathline before and after the broadened and deepened trench at km 4.65 will be shifted \(\sin(21.9°-17.0°)\times2700/cos(21.9°) = 254\) m. The depth decrease of the northern part of the pathline relative to the southern part then will be 0.53 m, which cannot be neglected.

The effect of streamline contraction on pathlines will be neglected for the case of the present situation. In case of the marine sand extraction scenario, the trajectories will be adapted on the basis of calculations analogous to those as performed in the previous paragraph.

### 6.2 Discretisation and Calibration of the Model

This section is devoted to aspects concerning discretisation and to the calibration of the model. Firstly the discretisation in time and space will be discussed. Subsequently, the sensitivity of the model to parameters which yield high uncertainty will be investigated, using a situation with best estimate values as a reference. The model will be calibrated by modification of these parameters, using the volume balance of the northernly slope of the Maasgeul from the data analysis.
6.2.1 Discretisation

A period of one year will be simulated, hence all 33 representative wave conditions occur three times within this period. The \((x, z)\) plane is discretised in a number of grid cells. The number of grid cells in the vertical direction is constant along the entire trajectory. During the calibration phase, the water depth is divided into fifteen equidistant spatial steps. Along the trajectory, the spatial step is taken 10 m.

Sediment concentrations are continuously adapting to the changing flow conditions. This adaptation can be seen as a relaxation process, with its own relaxation time. This adjustment time can be approximated by the local water depth, divided by the particle fall velocity [Ribberink and De Vriend, 1989]. The particle fall velocity varies between 1.61 and 2.45 cm/s, whereas the water depth during all calculations varies between 18 and 31 m. This implies the adjustment time varies between 12 and 31 minutes.

Since the adaptation time is relatively small compared to the tidal period, the duration of a tide in the model can be scaled up. By doing this, total computation time is reduced and the effect of numerical smoothing decreases. Initially, the duration of a tide is scaled up in such a manner that 20 tides occur within a month (more specific: one twelfth of a year), instead of approximately 58. Test runs showed that then the smoothing parameter can be set at \(10^{-4}\) without causing instability.

6.2.2 Sensitivity and Calibration

Having implemented all initial and boundary conditions and having defined the computational grid, the model is calibrated using the sediment balance of the northern slope and, in a qualitative manner, the sediment balance of the southern bank and slope. The numerical information on sedimentation and erosion on the southern slope and bank is of limited use because it will be strongly influenced by the dredging of the trench. However, the order of magnitude should be right. The same goes for the dredging data.

Initially a reference run is executed. Subsequently the sensitivity of the model to the steering parameters and the added residual current is determined.

By mistake, the tidal representation for the sensitivity runs is incorrect. Nevertheless it is possible to use the runs to assess the sensitivity of the model to the steering parameters. After the sensitivity analysis the adequate tide has been implemented.

Table 6.2 provides in an overview of the applied values of steering variables during the calculations.
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reference situation</th>
<th>Sensitivity Simulations</th>
</tr>
</thead>
<tbody>
<tr>
<td>reference level $z_e$ [m]</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td>wave roughness $r_w$ [m]</td>
<td>0.01</td>
<td>0.05</td>
</tr>
<tr>
<td>current roughness $r_c$ [m]</td>
<td>0.01</td>
<td>0.001</td>
</tr>
<tr>
<td>residual velocity [m/s]</td>
<td>0.01</td>
<td>0.05</td>
</tr>
<tr>
<td>fall velocity [m/s]</td>
<td>0.0245</td>
<td>0.0191</td>
</tr>
<tr>
<td>$D_{50}$ [$\mu$m]</td>
<td>210</td>
<td>175</td>
</tr>
<tr>
<td>$D_{90}$ [$\mu$m]</td>
<td>309</td>
<td>231</td>
</tr>
</tbody>
</table>

Table 6-2  Steering parameters in the sensitivity analysis

Figure 6.6 and 6.7 show the erosion and sedimentation after one year. In the reference situation, the erosion pattern along the trench seem to be qualitatively correct. The sedimentation on the southern slope and in the trench is compensated by the erosion on the northern slope. Deposition is highest at the beginning of the southern slope. It gradually decreases to a minimum at the toe of the northern slope. Erosion is largest at the upper end of the northern slope, which will lead to a decrease of the slope, in line with observations.

The model is the most sensitive to the wave related bottom roughness and the residual current velocity. The rates of erosion and sedimentation along the entire trajectory increase when applying a larger wave related bottom roughness. A smaller particle fall velocity, thus a smaller particle diameter, produces the same effect but to a lesser extent. Applying a larger residual velocity or current-related bottom roughness leads to an increased deposition, especially on the southern slope. The erosion on the northern slope will also increase, yet on the southern bank erosion will decrease. Unrealistic results are obtained for a reference level $z_e$ of 0.001.

The erosion on the northern slope, as calculated in the data analysis included in table 5.4 of chapter 5, is the average over the segment between 2645 m and 2749 m on the x axis in the figures 6.6 and 6.7. The yearly erosion on the northern slope appears to be an order of magnitude larger than according to the field data. The strategy to adapt the steering parameters is to put the bed roughness parameters at 0.001 m, and to reduce the residual current to zero. With the largest grain sizes $D_{50}$ and $D_{90}$ and corresponding fall velocity that occur in the Maasgeul, the model calculations still result in an overestimation of the erosion on the northern slope. Ultimately, the reference level is increased to 0.024 m, such that the erosion on the northern slope exactly coincides with the field data. The final values for the steering variables have been summarised in table 6.3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>After calibration</th>
</tr>
</thead>
<tbody>
<tr>
<td>reference level $z_e$ [m]</td>
<td>0.024</td>
</tr>
<tr>
<td>wave roughness $r_w$ [m]</td>
<td>0.001</td>
</tr>
<tr>
<td>current roughness $r_c$ [m]</td>
<td>0.001</td>
</tr>
<tr>
<td>residual velocity [m/s]</td>
<td>0</td>
</tr>
<tr>
<td>fall velocity [m/s]</td>
<td>0.0245</td>
</tr>
<tr>
<td>$D_{50}$ [$\mu$m]</td>
<td>210</td>
</tr>
<tr>
<td>$D_{90}$ [$\mu$m]</td>
<td>309</td>
</tr>
</tbody>
</table>

Table 6-3  Values of steering variables after calibration
6.2.3 Schematising Tidal and Wave Conditions and Transport along the Profile

During the calibration, the duration of a tide was scaled up in such a way that 96 spring tides and 96 neap tides occur within a year. For the long term simulations, the duration of a tide is scaled up to a single tide per month. It is not possible to scale up the duration of wave conditions as well, since the number of intervals for wave conditions is limited to 100. By iteration it is found that applying a wave with height 2.25 m and period 5 s results in approximately the same sedimentation/erosion pattern along the profile MG4.65, which is demonstrated in figure 6.8.

For the situation with a single wave, the net transport along the profile can be calculated. Figure 6.9 shows suspended load and bed load transport along the profile MG4.65 during flood and during ebb, averaged over neap and spring tide. The net transport is in the order of magnitude of 35 m³/year outside the trench, and reduces almost to zero in the northern part of the Maasgeul. The obtained transports with the Bijker formula where in the order of 225 m³/year, so an order of magnitude larger. The difference can partly be explained by the bed roughness which appeared to be smaller then estimated. The relative results of the Bijker formula rather then the absolute values, influence the quality of the tidal representation. Therefore, it is decided not to recalculate the tidal boundary condition, although a slight improvement could be obtained.

The net transport complies well with values obtained form the project Kustgenese [Van Rijn et al., 1995]. In this study, the net longshore sand transport rate at a depth of 20 m was estimated to be in the range of 15 to 45 m³/m/year in northward direction. The variation ranges were 10 to 20 m³/m/year.

6.3 Validation

A validation of the model will be performed on the basis of the following three tests.
1. comparison of simulated developments of SUTRENCCH applied at the trajectory MG4.65 with the observed behaviour of the trench
2. comparison of sediment deposit in the trench with measured dredging amounts
3. comparison of observed morphological activity at adjacent trajectories with model results for these tracks

First, the model is run for five years, simulating the dredging activities artificially once a year by adapting the profile locally over the segment that is defined to be the Maasgeul. Results are plotted in figure 6.10.
Figure 6-10 5 years dynamic modelling with yearly dredging at MG4.65

At the northern slope, the simulation result for t = 5 years perfectly coincides with the measured bed level after five years. This is not surprising since the model has been calibrated based on a sand balance of the northern slope. The southern slope, however, seems to grow continuously, which is not in accordance with observation. An attempt to obtain a more or less stable southern slope by decreasing the angle of natural slope did not lead to better results. Essential to the behaviour of a slope with a tendency to become steeper are bed slope effects in longitudinal as well as transverse direction. In chapter eight both effects will be discussed in detail.

During the five year simulation, the amount of sediment that had to be removed from the trench monotonically increased from 33 m$^3$ in the first year to 52 m$^3$ in the last year. The total amount of sediment that settles in the trench is larger, since deposition takes place at the southern slope. This can be explained by the shift of the southern slope to the north, capturing a smaller percentage of the sediment that settles in the trench. The measured dredging amount at this location equalled 58 m$^3$/m during the period 1985 - 1987, according to figure 5.3 from the data analysis. Simulating a long period would ultimately lead to much larger dredging amounts, which is not realistic.

A result of the data analysis was the observation of a gradual decrease of morphologic activity when in the offshore direction. The erosion of the northern slope decreases from 13.5 cm/year at MG4.65 to 2.19 cm/year at MG6.50. The model was applied to the trajectories MG5.25, MG5.88 and MG6.50. Velocities are adapted using the Chezy formula, according to formula 6.6. The yearly deposition and erosion is shown in figure 6.8.

From the data analysis it follows that there is an apparent decrease in morphological activity between, say 19 m depth and 21 m depth. Such a decrease hardly can be noticed from the SUTRENC calculations, although sedimentation and erosion at MG4.65 exceed deposition and erosion values of the other three trajectories. Two plausible explanations can be found. The first one concerns an oversimplification of the hydrodynamics. Instead of the tidal motion along the trajectories as defined in the data analysis, the real water motion could be much more complicated, as a result of the complex bathymetry and density differences. The depth increase along the trajectories supports this explanation; it is more likely that pathlines will run parallel to depth contours, which is not entirely the case in the model.
(see, for instance, figure 5.11). A second explanation is that formulations for the initiation of motion in the SUTRENCH model do not adequately represent reality. If the critical shear stress is being underestimated, for calm tidal conditions sediment particles in the model are in motion when they should not. This leads to an overestimation of sediment transport, and in addition probably to an overestimation of sedimentation and erosion, which is relatively largest where morphological activity is smallest.

In conclusion, the model performance at MG4.65 is hard to evaluate, but is nevertheless satisfactory since the behaviour of the northern slope is simulated well and the order of magnitude of measured dredging amounts corresponds with the deposition in the trench according to the model. Applying the model to trajectories offshore of MG4.65 leads to an overprediction of morphological activity.

6.4 Effects on Pipelines and Rate of Deposition

As can be noticed in figure 1.2, the distance between a gas pipeline and the northerly border of the planned sand excavation is limited. In the relatively shallow part of the sand extraction area, the area between the gas pipeline and the sand extraction pit is still several kilometres wide. At deep water, the distance between the extraction area and the pipeline equals 500 m, which the minimum allowed distance. In the following section the SUTRENCH model will be applied to both the shallow and the deep part of the extraction area. Apart from the extraction variant as proposed in the Maasvlakte study, a second variant will be simulated in which the deep extraction in the Maasgul is being replaced by a broader one which is half as deep. Simultaneously, insight in the rate of deposition will be obtained. First, the relatively shallow area will be described, next the deeper water.

Since the net transport is directed to the north, the largest impact can be expected on the northern side of the Maasgul. The trajectories MG4.65 and MG6.50 have therefore been extended to 15 km, in a direction to the north, and will be referred to as trajectories L10 and L30, respectively. A third trajectory, L20, lies exactly in the middle, between L10 and L30. Since alternative extraction variants imply different pathlines due to refraction, the segment of the trajectories north of the Maasgul will be adapted. The resulting cross sections along the tracks L11, L12, L21, L22, L31 and L32 are plotted in figure 6.11A-C. Figure 6.12 depicts the geographical location of the tracks. The effect of stream line refraction on pathlines has been taken into account for the deep part of the excavation only. Hence, the trajectories consist of three linear pathlines, of which the two outer ones are parallel.

Erosion and deposition during a year has also been plotted in figure 6.11A-C. Clearly, very little sediment is available to settle in the sand extraction area. For both variants, the sedimentation and erosion remains restricted to the slopes, where it locally can reach considerable values, up to 1.4 m/year on the southern slope. The width of the area at the slope where erosion and deposition are large is approximately 300 m. The amount of sediment deposited will be larger in case of the deep variant, since the difference between sand transport capacity in- and outside the trench will be larger. This can only be noticed at the trajectories L1* (= L10, L11 and L12) though, for the others the difference is negligible.

Figure 6.13A-C depicts the situation after 50 years of morphodynamic simulation for the reference situation and the two sand extraction variants. Morphologically, the migration of the slopes seems to take place with limited levelling. For all three trajectories, the rate of migration of the slopes is largest for the reference situation, and smallest for the deep variant 1. For variant 2, migration of the northern slope varies from 410 m within 50 years
at trajectory L12 to 205 m at trajectory L32. For variant 1, these values correspond to 150 m and 100 m, respectively.

As the trajectories of the reference runs do not completely coincide with the trajectories of the sand extraction variants, the extension of the area of influence is difficult to determine. Moreover, the boundaries are kept at a constant depth during the simulation by the model, which might influence the obtained profiles to some extent. Nevertheless it is clear that erosion northerly of the sand extraction will not reach beyond the sand dam that is located approximately five kilometres northerly of the Maasgeul. The area of influence can be approximated theoretically, by calculating the adaptation length for sediment concentrations to adjust to altered flow conditions. This adaptation length equal the velocity multiplied by the adaptation time [Ribberink and De Vriend, 1989], which was calculated to equal 31 minutes at maximum. With a maximal tidal velocity of 1.1 m/s, the area of influence can be approximated by 2 km, plus the distance over which the slope has migrated. Consequently, the area of influence is limited to an area of 2.5 km within 50 years.

As to the alternative sand extraction schemes it can be concluded that the initial migration rate of a shallow sand extraction will be larger than for a deep sand extraction of the same volume. Therefore, the area of influence will also be larger. Since a sand extraction area will become shallower and broader in time, the migration rate will increase in time.

The nearest gas pipeline lies at km 9.3 in the trajectories L1* and at km 10.8 and km 12.0 in the trajectories L2* and L3*, respectively. This is a very favourable location, since the sand dam safeguards the pipeline from possible erosion due to sand extraction at least for the next 50 years. Since the lifespan of a pipeline amounts 10 to 50 years [personal communication Mr. Verhagen, Directie Noordzee], the gas pipeline will no longer be used by the time erosion has reached it. Hence, there where the planned sand extraction is relatively deep with respect to its surroundings, no problems of erosion near pipelines are to be expected.

Applying the model with the described approximations to a situation at deep water can be considered a worst case scenario, since the model overpredicts morphological activity at deep water, as concluded in the validation. A dynamic run over 50 years for a situation from NAP -33 m to NAP -31 m results in the development as represented in the figure 6.14. The net displacement of the slope is in the order of 600 m, and the area of influence is strictly speaking, approximately seven kilometres in the tidal direction.
Figure 6-14  Long shore development of a slope at deep water

If a similar development would occur in reality, strong erosion at place of the pipelines takes place. The planned distance from the pipeline to the sand extraction of 500 m would be chosen far too small. Only additional field data can provide insight into the extent to which the simulated migration of the slope bears reference with reality.
7 Modelling of Morphological Cross-shore Developments

The predicted developments in longshore direction provide answers to the questions referring to cables and pipelines, deposition rates and future maintenance dredging. For coastline maintenance, however, longshore developments at approximately eight kilometres offshore only are of indirect importance. In order to know how the sand budgets in the nearshore area change, it is necessary to model cross-shore processes under the modified bed level and hydrodynamic conditions.

In the alongshore transport computations, the excavation has been considered to be of an infinite width. However, the influence of pathline divergence upstream of the centre of the excavation, and pathline convergence downstream, undoubtedly will be noticeable at the coastal side of the sand excavation in figure 1.2. A graphical explanation is given in figure 7.1. The plot depicts the difference between velocities with and without an excavation in a certain area. This implies that at the coastal side of the excavation, flow velocities inside the deepened area will be relatively high and just outside relatively low since there is less friction for a larger water depth. To some extent this mechanism leads to conservation of the excavation. Hence, it should be borne in mind that a possible flattening of the slope at the coastal side of the sand excavation, for instance as a result of the wave motion, can be resisted by an adaptation of the local tidal velocities.

Modelling of cross-shore processes in the first kilometres outside the breaker zone is difficult. None of the basic processes in tidal seas can be neglected a priori and even if one can speak of an equilibrium situation, it will be dynamic and delicate and therefore hard to describe in a quantitative manner. Nevertheless, an attempt can be made to determine an upper bound for the migration rate of a bed level disturbance and the loss of sand at deep water.

In the present chapter, first the order of magnitude of the cross-shore transport will be determined using the model UNIBEST TC. An estimate of cross-shore migration rates of bottom disturbances has been determined in the report Marine Sand Extraction [Ribberink and Roelvink, 1989]. A brief review of these results is presented below.

7.1 Cross-shore Sediment Transport

Since longshore morphological developments are directed to the North, the influence on the coast will be largest at the coast of Delfland. Therefore, the UNIBEST TC model will be applied to a cross section at Hook of Holland, taken from the RUMAMO bathymetry. The offshore extent of the cross section will be 14 km. First suspended load and bed load transport rates for a number of representative conditions will be calculated. Subsequently, the model will be run simulating a relatively short period of four years to investigate sedimentation and erosion at deep water. However, the simulation period suffices to determine where and what amount of initial sedimentation and erosion can be expected.

Many of the specific parameters for UNIBEST TC can be derived from the report ‘Beach Nourishment at Delfland’ [Reniers and Roelvink, 1997]. Wave and wind conditions
measured from the Europlatform will be utilised. The longshore tidal motion will be estimated more or less in line with the approach applied in the SUTRENCH analysis (see paragraph 6.1.2). The net transport during a flood or ebb period is kept constant.

### 7.1.1 Characteristic Conditions

Figures 7.2 - 7.5 demonstrate cross shore bed load and suspended load transports in case of wave, tide and wind conditions as summarised in table 7.1. The wind velocities are characteristic for the specific wave condition, as reported in the study ‘Location at the Coast’ [Bosboom et al., 1996]. The cross shore profile is depicted in figure 7.8. When lines are hard to detect at deep water in the pictures, they run along the x axis. A positive sign means onshore transport. For all conditions waves are assumed to propagate in a direction perpendicular to the coast.

<table>
<thead>
<tr>
<th>Wave condition</th>
<th>Tidal condition</th>
<th>Wind condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{rms} = 0.57 \text{ m } T_p = 5.24 \text{ s}$</td>
<td>$V_{ebb, \text{neap tide}} = -0.33 \text{ m/s and}$</td>
<td>no wind, $V_{wind} = 4 \text{ m/s on shore}$, $V_{wind} = 4 \text{ m/s off shore}$</td>
</tr>
<tr>
<td>$H_{ebb, \text{ neap tide}} = 0.27 \text{ m}$,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{flooding \text{ spring tide}} = 0.64 \text{ m/s and}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{flooding \text{ spring tide}} = -0.42 \text{ m}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{rms} = 1.13 \text{ m } T_p = 6.00 \text{ s}$</td>
<td>$V_{ebb, \text{ neap tide}} = -0.33 \text{ m/s and}$</td>
<td>no wind, $V_{wind} = 7 \text{ m/s on shore}$, $V_{wind} = 7 \text{ m/s off shore}$</td>
</tr>
<tr>
<td>$H_{ebb, \text{ neap tide}} = 0.27 \text{ m}$,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{flooding \text{ spring tide}} = 0.64 \text{ m/s and}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{flooding \text{ spring tide}} = -0.42 \text{ m}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{rms} = 1.70 \text{ m } T_p = 6.76 \text{ s}$</td>
<td>$V_{ebb, \text{ neap tide}} = -0.33 \text{ m/s and}$</td>
<td>no wind, $V_{wind} = 10 \text{ m/s on shore}$, $V_{wind} = 10 \text{ m/s off shore}$</td>
</tr>
<tr>
<td>$H_{ebb, \text{ neap tide}} = 0.27 \text{ m}$,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{flooding \text{ spring tide}} = 0.64 \text{ m/s and}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{flooding \text{ spring tide}} = -0.42 \text{ m}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{rms} = 2.26 \text{ m } T_p = 6.76 \text{ s}$</td>
<td>$V_{ebb, \text{ neap tide}} = -0.33 \text{ m/s and}$</td>
<td>no wind, $V_{wind} = 13 \text{ m/s on shore}$, $V_{wind} = 13 \text{ m/s off shore}$</td>
</tr>
<tr>
<td>$H_{ebb, \text{ neap tide}} = 0.27 \text{ m}$,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{flooding \text{ spring tide}} = 0.64 \text{ m/s and}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_{flooding \text{ spring tide}} = -0.42 \text{ m}$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 7.1 Characteristic conditions

Near the coast, suspended load and bed load transport are mainly offshore and on shore directed, respectively. At deep water, the transport direction changes. The tidal motion appears to intensify especially suspended load transports, which seems to depend completely on the presence of a tide at deep water. The bed load transport is barely effected by either the tidal conditions or an onshore wind, even under extreme conditions. The offshore suspended load transport decreases as a result of onshore wind. An offshore wind strongly acts upon both suspended load and bed load transport, generating on shore transport. Bearing in mind that offshore winds do not occur very often, the net transport at deep water probably will be directed off shore.

### 7.1.2 Dynamic Behaviour at Deep Water

In order to obtain an impression of morphological activity in a cross-shore direction at deep water, a period of 4 years will be simulated, based on a time series of wave and wind data of one year. Transports at the NAP -20 line will be visualised, as well as the deposition and erosion during the simulation period.
A time series of the Euro Platform will be applied, containing wave and wind conditions with an intermediate period of 3 hours. In the UNIBEST TC model, the direction of incoming waves has to be onshore directed and should not deviate more than approximately 70° from a direction perpendicular to the coast. The propagation direction of waves that do not satisfy this prerequisite is set at 70 degrees. A time step of one day will be applied. For a subsequent time step, the wave and wind conditions after 6 hours in the time series is being applied, skipping a single set of conditions. Hence, the wave and wind climate of a year is being spread out over a simulation period of four years. This way, calculation time goes down compared to a 'real time' simulation, in which time in the UNIBEST TC calculation corresponds to that in the time series. Seasonal influences still are taken into account though.

In order to allow for large spatial steps, the cross section is smoothed. The spatial resolution of the grid at deep water is 300 m. The cross shore profile is demonstrated in figure 7.10. Alternately ebb and flood are implemented. Tidal velocities have been computed by calculating the total transport during each of the fourteen ebb and fourteen flood periods, and determining which velocity results in the same transport by iterating. Subsequently, the surface elevation has been calculated based on an approximation in which the surface elevation is being correlated to velocities. This way, fourteen tides have been obtained which can represent the tidal motion in UNIBEST TC. Surface elevation and tidal velocities are demonstrated in figures 7.6 and 7.7. The residual velocity amounts 4.4 cm/s to the north.

![Tidal surface elevation representation](image)

**Figure 7-6** Tidal surface elevation representation

![Tidal velocity representation](image)

**Figure 7-7** Tidal velocity representation

The net deposition and erosion during each of the four years, from km 3 to km 14 is depicted in figure 7.10. The morphological activity in the nearshore area is much larger and therefore is kept out of the analysis. Hence, implicitly the assumption is made that the direct
effect of the sand extraction on the near shore area will be negligible. In UNIBEST TC, the time averaged sediment continuity equation has been implemented, which reads:

\[ \frac{dc}{dz} = 0 \]

(7.1)

where:

- \( w_{s,m} \) = fall velocity of suspended sediment in a fluid-sediment mixture [m/s]
- \( \varepsilon_{s,cw} \) = sediment mixing coefficient for combined current and waves [m²/s]
- \( c \) = time-averaged concentration at height \( z \) above the bed [kg/m³]
- \( \varphi_d \) = damping factor dependent on the concentration [-]

Consequently, concentrations are being computed locally so that the near shore area and the outer, off shore, region can be treated separately unless morphological changes in the near shore area enter the area at deep water or vice versa. The reference situation can be compared with the situation with a sand extraction from NAP -20. In general, the erosion and the sedimentation are of an order of magnitude of \( 10^3 \) m/year. The inaccuracy of the model exceeds this value, so the results can only be interpreted qualitatively [personal communication with Walstra and Aaminkhof, Delft Hydraulics, 1997].

Sedimentation and erosion along the profile during the four years of simulation have been plotted in figure 7.10. The x-axis is chosen positive to the coast, which corresponds to the way it is defined in UNIBEST TC. During the first year, in the reference situation as well as in the sand extraction alternative, a tendency to smoothen the increase in bed slope can be observed between km 5 and 6. Probably, this can be considered a spin up effect. After the first year, permanent erosion takes place at the steep part of the cross section, from km 5 to 9. Hence, when moving in an on shore direction, the net transport either decreases if it is off shore directed, or increases when the it is on shore directed.

On the steep part of the cross-shore profile, the difference between the reference situation and the extraction alternative is largest. The relative sedimentation/erosion has the same order of magnitude as the absolute values. An important observation is that the relative erosion/sedimentation is not confined to the region where sand extraction took place. This will be the result of the larger wave intrusion. Modifications induced by the sand extraction especially take place where slope gradients are relatively large.

Bed load and suspended load transport as a function of time at 20 meter water depth are presented in figure 7.9A and B.
A high variability can be observed, with peaks up to 300 m$^3$/year. From year one to three, the transport increases, which is in accordance with the increase in morphological activity as shown in figure 7.10. Integrating the transport over time leads to the time averaged transport rates in table 7.2. The net transport is dominated by suspended load, and is therefore off shore directed.

<table>
<thead>
<tr>
<th></th>
<th>on shore</th>
<th>off shore</th>
<th>net</th>
</tr>
</thead>
<tbody>
<tr>
<td>yearly bed load transport</td>
<td>2.84 m$^2$</td>
<td>-1.06 m$^2$</td>
<td>1.78 m$^2$</td>
</tr>
<tr>
<td>yearly suspended load transport</td>
<td>0.62 m$^2$</td>
<td>-4.66 m$^2$</td>
<td>-4.04 m$^2$</td>
</tr>
<tr>
<td>yearly total load transport</td>
<td>3.46 m$^2$</td>
<td>-5.72 m$^2$</td>
<td>-2.26 m$^2$</td>
</tr>
</tbody>
</table>

Table 7-2 Time averaged transports at NAP-20

The incorporation of the bed slope effects in the UNIBEST TC model is rather intuitive. An extensive discussion on bed slope effects is presented in chapter 8. An upper bound for the maximum sand loss as a result of bed slope effects can be obtained by presuming the onshore directed bed load transport to be reduced to zero as a result of the steep slope at the border of the sand excavation. The 'sand loss', at 8 km from the coast, then would be 2.84 m$^3$/year, per meter in long shore direction. For comparison: the yearly need for beach nourishment at Delfland during the period 1991 - 1996 amounted 42.86 m$^3$/year [Directorate-General for Public Works and Water Management, 1996].
It can be concluded that the nearshore transport mechanisms applied to a situation at deep water will not lead to a substantial loss of sediments from the lower shore face to the inner shelf (see definition of [De Vriend et al., 1993]). Modifications in the middle shore face and active zone have not been investigated. The wave field, even under severe conditions, is barely changed [Kuipers and Phillipart, 1997b]. Hence, it is not very probable that the change of morphological activity due to sand extraction in the near shore zone is larger than surroundings of the extraction.

7.2 Cross-shore Migration Rate

The report Marine Sand Extraction [Ribberink and Roelvink, 1989], describes simulations of the cross-shore developments of excavation alternatives at a depth of NAP -10 to -20 m, over a period of 40 years. For this purpose the cross-shore model CROSTRAN was utilised [Stive, 1986 and Roelvink and Stive, 1988], which is a predecessor of the UNIBEST TC model. The cross-shore profiles in simulations with an excavation seaward of NAP -16 m were strongly schematised. The width and depth of the excavations are varied between 1 and 5 m and 100 and 500 m, respectively. The wave climate was represented by five characteristics wave classes, with corresponding period. In the report, the following conclusions are drawn.

- Within a period of 40 years, cross shore morphological processes barely modify the shape of a sand excavation at a depth of NAP -16. Morphological activity at the -20 m depth contour is even smaller, as the influence of waves noticeably decreases in the offshore direction.

- For all simulations a net onshore displacement of the excavation is found. The maximum migration rate is 1 to 2 m per year. Cause of the migration is a net onshore transport, which decreases for an increasing water depth. Figure 7.11 depicts the rate of onshore migration of bed level disturbances as a function of the water depth. On the basis of this picture, the time necessary for a bed level disturbance to reach the NAP -10 line can be calculated as a function of the water depth, or distance to the coast. The resulting relationship has been plotted in figure 7.12.

- Flattening of the slopes is limited. The spreading effect has been translated into a diffusion coefficient, K, which turns out to be rather insensitive to the water depth and has a magnitude in the order of 700 - 1400 m²/year. A spreading length σ can be approximated by σ ≈ √(Kt), which results in a maximal spreading length of 374 m in a hundred years.
8 Bed Slope Effects

For the purpose of impact assessment of large scale marine sand extraction the SUTRENCH model appeared to lead to satisfactory results. The simulation results were especially good for the northerly slope, where bed level gradients are relatively small. With respect to the southerly slope however, it is highly questionable whether the steeping of the slope, which follows from the SUTRENCH calculations, occurs in real. After construction of a trench, in general slopes erode and become less steep. Initial slopes usually have a gradient in the order of 10%. This development of the slopes goes on until some, possibly dynamic, equilibrium has been reached. This equilibrium will be influenced by the maintenance depth of the trench.

In case of the Maasgeul, the northern slope apparently did not yet reach an equilibrium, since a permanent erosion and a decrease of bed level gradients can be observed. Despite the fact that during the period 1990 - 1993 sedimentation occurs, which, however, was not attended by a change in bed level gradients, the southerly slope probably reached an equilibrium situation. The deposition can be explained by a temporary absence of dredging on the toe of the slope, by which the slope had the freedom to move somewhat to the north.

In this chapter it will be derived what physical processes might contribute to an equilibrium of bed level gradients, and why it can not be simulated by the SUTRENCH model. From the conviction that it is the most important process that has been neglected in the approach that has been followed, quantitative information will be generated on downslope transport as a result of shear stresses in the longitudinal direction of the trench. A simple model will be presented in order to address to the question what longitudinal water level gradients are necessary to compensate for deposition on the southern slope due to transport gradients in lateral direction, as modelled by SUTRENCH.

The work of Damgaard et al. [Damgaard, 1995] can be used for a state-of-art description of knowledge on sediment transport on sloping beds.
8.1 Longitudinal Slope effects

Considering transport of sediment particles by a flow of water, usually three modes of particle motion are distinguished: (1) rolling and sliding motion or both; (2) saltation motion; and (3) suspended particle motion [Van Rijn, 1987]. As in the SUTRENCH model, the first two modes normally are jointly treated as being the bed load transport, the third as being the suspended load transport. Local bed level gradients will directly influence bed load transport, since a sediment particle easily will roll down a hill and will have difficulty rolling uphill. Therefore a perfectly symmetrical short wave, in absence of a tidal motion, can cause flattening of a slope. The influence of bed level gradients on suspended load transport will be less extensive and more indirect. In this section an attempt will be made to explain the equilibrium of the slope of a maintained access channel by a balance between the tendency to deposit material due to an increasing water depth and the tendency to erode due to an increasing bed-load transport on the slope.

8.1.1 Representation of Bed Slope Effects in Bed-load Transport Formulations

In the SUTRENCH model, to some extent bed slope effects are incorporated since the instantaneous critical bed shear stress depends on the slope angle, as follows (Van Rijn, 1987):

In up sloping direction:

\[ \tau_{b,cr_1} = \frac{\sin(\phi + \alpha)}{\sin(\phi)} \quad \tau_{b,cr_2} = \frac{\sin(\phi - \alpha)}{\sin(\phi)} \]  

(8.1)

In down sloping direction:

\[ \tau_{b,cr_1} = \frac{\sin(\phi - \alpha)}{\sin(\phi)} \quad \tau_{b,cr_2} = \frac{\sin(\phi + \alpha)}{\sin(\phi)} \]  

(8.2)

in which:

\( \tau_{b,\alpha,1} \)  
\( \tau_{b,\alpha,2} \)
\( \tau_{b,\alpha,0} \)
\( \phi \)
\( \alpha \)  

= instantaneous critical bed-shear stress in flow direction [N/m²]
= instantaneous critical bed-shear stress against flow direction [N/m²]
= instantaneous critical bed-shear stress at a horizontal bed [N/m²]
= angle of internal friction [°]
= slope angle [°]

These formulae practically imply a linear relationship between the slope angle and the instantaneous critical bed-shear stress, which is graphically demonstrated in the picture below:
In effect, the instantaneous critical bed shear stress influences the transport stage parameter $T$, which expresses the mobility of the particles in terms of the stage of movement relative to the critical stage for initiation of motion:

$$
T = \frac{u_0^2 - u_{*cr}^2}{u_{*cr}^2}
$$

(8.3)

where:

- $u_{*cr} = \sqrt{t_{bc}/\rho}$ = critical bed-shear velocity according to Shields [m/s]
- $u_* = \frac{\sqrt{g u}}{C}$ = effective bed-shear velocity related to the grains [m/s]
- $C = 18 \log \left( \frac{12h}{3d_{50}} \right)$ = Chézy-coefficient related to grains [m$^{0.5}$/s]
- $\rho$ = density of the fluid [kg/m$^3$]
- $u$ = depth averaged flow velocity [m/s]
- $d_{50}$ = particle diameter of bed material [m]
- $g$ = acceleration of gravity [m/s$^2$]

Finally, the bed-load transport rate, $s_b$, depends on the transport parameter:

$$
s_b = 0.053 \sqrt[1.5]{\Delta g d_{50}} D_*^{-0.3} T^{2.1}
$$

(8.4)

in which:

- $D_* = d_{50} (\Delta g/\nu)^{1/3}$ = dimensionless particle parameter [-]
- $\Delta = (\rho_s - \rho) / \rho$ = relative density [-]
- $\rho_s$ = density of the sediment [kg/m$^3$]
- $\nu$ = kinematic viscosity [m$^2$/s]

The rate of deposition on the southern slope is relatively insensitive to the angle of repose. Consequently it will be insensitive to changes bed level gradients.

The transport induced by gravity should be incorporated in a more direct manner. In case of the UNIBEST TC model, the instantaneous critical bed-shear stress depends on the local slope, and is the bed-load transport perpendicular to the shoreline multiplied by a slope factor, $\beta_0$, which is defined as follows:
\[ \beta_z = \left( \frac{dz_b}{ds} \right)^{-1} \left( 1 + \frac{dz_b}{\tan \phi} \right) \]  

(8.5)

with the slope: \( \frac{dz_b}{ds} = \frac{u_{bc} dz_{bc}}{u_b} \), limited by: \( \left| \frac{dz_b}{ds} \right| < \tan \phi \)

where:
- \( u_b \) = the time-dependent (intra-wave) near-bottom horizontal velocity vector of the combined wave-current motion at the top of the wave boundary layer (m/s)
- \( x \) = direction perpendicular to the shoreline (m)
- \( \phi \) = angle of repose (°)

A graphical interpretation is given below.

![Graph](image)

Figure 8-2  Slope factor in the UNIBEST TC MODEL.

In the model the angle of repose is user defined, and it is explicitly stated that it may differ from the natural angle of repose. However, an angle of repose different from the natural one does not represent any physical quantity. Hence an opportunity has been incorporated to take into account bed slope effects, but the predictive value of this aspect of the model in absence of field data is debatable.

### 8.1.2 Increase of Bed-load Transport versus Decrease of Suspension-load Transport on a Slope

One could speculate whether gradients in the bed-load transport, if correctly modelled, could compensate for the deposition on the southern slope of the Maasgeul as a result of gradients in suspended load transport. Looking at the yearly averaged transport perpendicular to the Maasgeul, as depicted in figure 6.9, on the southerly slope the gradients of both the suspended and the bed-load transport lead to deposition. In order to compensate for the deposition due to suspended load transport gradients, the bottom transport should increase where the southerly slope starts.

Using the ‘UNIBEST TC approach’, which is based on a constant angle of repose, \( \phi \), it can be shown on the basis of the qualitative pictures below that incorporating slope effects in SUTRENCHE would lead to flattening of the slope. In the pictures, the indices 1 and 2 refer to
two optional slope profiles: an S curve and a practically linear slope. The angle of repose, $\phi$, can be chosen such that on the upper part of the slope the bed-load transport increases in the direction of the trench. This positive bed-load transport gradient can compensate for the deposition due to a negative suspended load transport gradient. As a consequence, on the lower part of the slope the bed-load transport will have to decrease, at a higher rate than before. This holds for both slope profiles.

Cross Sections of the Southern Slope of the Maasgeul

Figure 8-3 Bed slope effect on results of SUTRENC modelling

Thus, incorporating the slope factor as formulated in the UNIBEST TC model would result in a decrease in sedimentation at the upper part of the southern slope, but also in an increase in deposition on the lower part of the slope. Deposition due to suspended load gradients especially takes place on the lower part of the slope. Hence, the conclusion may be drawn that no simple mechanism has been found by which bed slope effects in the
direction of the bed level gradients could compensate for sedimentation due to a decrease of current velocity on a slope.

### 8.2 Transverse Bed Slope Effects

Up to here, sediment transport in a direction along the Maasgeul has been assumed not to occur. Nevertheless, a residual velocity plot of the Maasgeul area shows that, near the bottom, cross-shore velocity components of the tidal motion are considerable and predominantly onshore directed [see figure 4.5]. The cross-shore residual velocity increases in a shoreward direction, and is in the order of 0.12 m/s. This indicates that transportation of sediments in a shoreward direction will not be negligible. Due to gravity, bed-load transport on the slopes the Maageul as a result of a longitudinal current will have a component directed to the centre of the trench. Hence, another possible mechanism has been found that increases sediment transportation off the slopes. In the next section it will be derived what longitudinal water level slope is necessary to cause such erosion pattern across the southern slope of the Maasgeul, that is opposite to the deposition pattern which resulted from the SUTRENCH calculations at km 4.65.

#### 8.2.1 Transverse Slope Model

Focusing on the southern slope of the Maasgeul, the situation to be modelled is outlined in the graph below.

![Definition sketch of the Transverse Slope Model](image)

Figure 8-4  Definition sketch of the Transverse Slope Model

The assumption of a stationary situation in which erosion due to gradients in transverse bed-load transport equal the deposition on the slope resulting from the Sutrench calculations, yields the following sediment continuity equation:

\[
\frac{\partial z_b}{\partial t} = f(n) - \frac{1}{(1-\Gamma)} \frac{\partial S_{bed,n}}{\partial n} = 0 \quad \Rightarrow \quad f(n) = \frac{1}{(1-\Gamma)} \frac{\partial S_{bed,n}}{\partial n} \tag{8.7}
\]

\[\Gamma\] = porosity of bed material
f(n) = deposition due to current velocity decrease in transverse direction [m/s]
S_{bed,n} = transverse bed-load transport (excluding pore volume) [m²/s]

Essential is the modelling of the transverse bed-load transport, which is due to a downhill gravity component. A linear relationship between the ratio of transverse and longitudinal bed-load transport and the local slope is assumed.

\[
\frac{S_{bed,n}}{S_{bed,l}} = -G \frac{\partial z}{\partial n}
\]  \hspace{1cm} (8.8)

S_{bed,l} = longitudinal bed-load transport [m²/s]
G = direction coefficient [-]

The G-value is very important to morphological models because its value affects the transverse bed slopes in bends and wave length and damping of spatial bed deformations [Talmon, 1993]. A relatively simple expression derived by Sekine & Parker will be used [Sekine and Parker, 1992].

\[
G = 0.75 \left( \frac{\phi_{cr}}{\phi} \right)^{1/4}
\]  \hspace{1cm} (8.9)

\(\phi\) = Shields parameter [-]
\(\phi_{cr}\) = Shields parameter at initiation of motion [-]

The method of Van Rijn (1993) will be followed to calculate the critical Shields value. He represented the Shields curve as follows:

\[
\phi_{cr} = 0.24 \ D_*^{-1}, \ \ 1 < D_* \leq 4 \\
\phi_{cr} = 0.14 \ D_*^{-0.64}, \ \ 4 < D_* \leq 10 \\
\phi_{cr} = 0.04 \ D_*^{-0.1}, \ \ 10 < D_* \leq 20 \\
\phi_{cr} = 0.013 \ D_*^{-0.29}, \ \ 20 < D_* \leq 150 \\
\phi_{cr} = 0.055 \ D_* \quad , \ \ 150 < D_*
\]  \hspace{1cm} (8.10)

with

\[
D_* = d_{50} \left( \frac{g \Delta}{\nu^2} \right)^{1/3}
\]  \hspace{1cm} (8.11)

\(\Delta\) = relative density of the sediment to the fluid [-]
g = acceleration of gravity [m/s²]
D_{50} = mean particle diameter [m]
\(\nu\) = kinematic viscosity [m²/s]

The bed load sediment transport in the main flow direction is modelled by the Engelund & Hansen formula for total sediment transport, which is multiplied by the fraction of bed-load transport.
\[ S_{\text{bed},s} = 0.05 \sqrt{\Delta g D_{50}^3} \frac{C^2}{g} (\phi)^{2.5} (1 - X) \]  
(8.12)

\[ S_{\text{tot}} = \text{total load transport in longitudinal direction} \quad [\text{m}^2/\text{s}] \]
\[ C = \text{Chézy bed friction coefficient} \quad [\text{m}^{1/2}/\text{s}] \]
\[ \rho = \text{fluid density} \quad [\text{kg/m}^3] \]
\[ a = \text{depth} \quad [\text{m}] \]
\[ X = \text{fraction of suspended sediment transport} \quad [-] \]
\[ \phi = \text{Shields parameter} \quad [-] \]

The Shields parameter reads:

\[ \phi = \frac{\tau_{\text{bed},s}}{\Delta \rho g D_{50}} \]  
(8.13)

\[ \tau_{\text{bed},s} = \text{bed shear stress in longitudinal direction} \quad [\text{N/m}^2] \]

For the Chézy bed friction coefficient the White-Colebrook formula will be applied.

\[ C = 18 \log \left( \frac{12.2a}{k_s} \right) \]  
(8.14)

in which is \( k_s \), the equivalent Nikuradse sand-roughness. The average value of the equivalent sand-roughness of a flat mobile bed, reported in literature, is \( k_s = 3d_{50} \) (Van Rijn, 1982). The Sutrench calibration resulted in a wave roughness of 0.001 m, which is the same order of magnitude as 3d_{50}. Therefore, \( k_s = 3d_{50} \) will be applied.

Van Rijn (1994c) has compiled data on the fraction of suspended transport, \( X \), in laboratory channels as a function of \( u_s/w_s \). These data can be approximated by an empirical function.

\[ X = \frac{1}{1 + \left[ \frac{1}{0.8} \frac{k_s}{a} \right]} \]  
(8.15)

in which:

\[ F = \left[ \frac{k_s}{a} \right]^2 - \left[ \frac{k_s}{a} \right] \cdot \left[ 1 - \frac{k_s}{a} \right] \]
\[ Z = Z + \phi; \quad Z = \frac{w_s}{\beta u_s}; \quad u_s = \sqrt{\tau_{\text{bed},s}/\rho} \]  
(8.10a, b, c, d)
\[ \beta = 1 + 2 \left[ \frac{w_s}{u_s} \right]^2, \quad \text{for} \quad 0.1 < \frac{w_s}{u_s} < 1 \]  
(8.16)
F = correction factor in the applied approximation \( S_{sur} = F \mu a c_a \), in which \( c_a \) is a reference concentration [-]

\( Z \) = modified suspension number [-]

\( Z \) = suspension number [-]

\( \phi \) = damping parameter, = 0 for low concentrations [-]

\( \beta \) = ratio of sediment diffusion and fluid diffusion coefficient [-]

The hydrodynamics will be modelled by assuming a constant water level slope in longitudinal direction, and a constant water level in transverse direction. Analogous to the situation in a river, the local bed shear stress and water level are given by:

\[
\tau_{bed,n} = \rho g a l
\]

\( H = z_b + a \) 

\( l \) = water level slope [-]

\( H \) = water level [m]

Additionally the depth-averaged velocity can be calculated according to the Chézy equation:

\[
u = C \sqrt{a l}
\]

Based on these assumptions, the model is one dimensional and consists of two coupled differential equations, forced by a term that represents deposition on the slope. Using equations 8.7-8.9 and 8.12, the model can be rewritten as follows.

\[
\begin{align*}
\frac{\partial S_{bed,n}}{\partial n} &= (1 - \Gamma) f(n) \\
\frac{\partial \tau_{bed,n}}{\partial n} &= -\frac{K}{C^2 (1 - X)} \phi^{-2.25} S_{bed,n}
\end{align*}
\]

in which:

\[
K = \frac{1}{0.0375 [\phi_a (d_{50}, g, \Delta, \nu)]^{1/2} \sqrt{\Delta D_{50}^3}}
\]

Boundary conditions have to be chosen for both \( z_b \) and \( S_{bed,n} \):

\[
\begin{align*}
z_b(0) &= 6 \text{ m.} \\
S_{bed,n}(0) &= 0 \text{ m}^2/\text{s}
\end{align*}
\]

The function \( f(n) \) has to be specified. The deposition will be modelled by two segments, a sinusoidal and a linear one, which has been demonstrated in the graph below. The linear segment ends at a value of zero, because the deposition in the trench will not be eroded by the modelled mechanism.
The model has been implemented in the mathematical package Powersim, in which coupled ordinary differential equations can be solved, by making use of a user defined numerical scheme. The function that represents the deposition on the slope can be represented by an if-statement. The following parameters will be used, largely based on the Sutrench analysis.

<table>
<thead>
<tr>
<th></th>
<th>0.4</th>
<th>0.0218 m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Gamma )</td>
<td>1025 kg/m³</td>
<td>( D_{50} ) = 193 \times 10^{-6} m</td>
</tr>
<tr>
<td>( \rho )</td>
<td>( D_{90} ) = 270 \times 10^{-6} m</td>
<td></td>
</tr>
<tr>
<td>( \nu )</td>
<td>2650 kg/m³</td>
<td>( H ) = 25 m</td>
</tr>
<tr>
<td>( \rho_s )</td>
<td>( g ) = 9.81 m/s²</td>
<td></td>
</tr>
<tr>
<td>( \phi )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8-1 Parameters in the Transverse Slope Model

The damping effect at 19 m water depth will be minimal. Therefore, the ratio of the diffusion coefficient in a fluid-sediment mixture and in a clear fluid, \( \phi \), is assumed to equal 0. With the chosen parameters the critical Shields parameter \( \phi_c \) appears to be 0.051. During calculations a check will be made to see whether the Shield parameter exceeds this threshold along the slope. A second check should be made to verify whether \( 0.1 < \frac{w_D}{u_*} < 1 \), which was a necessary condition for the chosen formula for the ratio of sediment diffusion and fluid diffusion coefficient.
After having substituted the above numerical values and the if-statement, the model finally needs the longitudinal water level slope, I, in order to calculate the transverse transport and bed level as a function of the transverse co-ordinate. By varying I, different bed level slopes will be calculated. By iterating the situation can be obtained for which the bed level slope resembles to the southern slope of the Maasgeul at km 4.65. Apart from the transverse bed-load transport and bed level distribution, the following variables will also be calculated explicitly as functions of the transverse co-ordinate: water depth, shear velocity, depth averaged velocity, Shields parameter, ratio of suspended and total load and the Chézy coefficient. The flow chart that represents the Powersim implementation is shown in figure 8-7.

![Figure 8-7 Powersim Flow Chart of Transverse Slope Model](image)

**8.2.2 Required Water Level Slope**

It appears to be possible to find a value for the water level slope, I, for which the shape of the bed level coincides with the equilibrium bed level of the southern slope at Maasgeul km 4.65, in both a qualitative and a quantitative sense. The required water level slope appears to be $5.6 \times 10^6$. The resulting graphs of the various variables are plotted in figure 8.8 and 8.9. Both above-mentioned conditions are satisfied, since $w/u_*$ varies between 0.58 and 0.66, and the Shields parameter exceeds its critical value. As a result of the simulation velocities are 1.04 m/s at the upper side of the slope and 1.21 m/s in the trench. As shown in figure 8.9, the fraction bed-load transport of total load transport is 0.21 at the flat part of the profile and it decreases to 0.18 down slope. An additional calculation showed that for a bed roughness of 0.06 m velocities at the upper end of the slope and in the trench should equal 1.30 and 1.55 m/s, respectively.

Hence the required longitudinal velocities have to be in the order of 1 m/s, which corresponds with the order of magnitude of the tidal velocity amplitude. Qualitatively, a
mechanism has been found by which sediment can be moved from the lower part of the slope to the trench. Velocities near the bottom of the Maasgeul are not always directed along the trench though.

In general, short waves at deep water result in an increase in bed-load as well as suspended load transport since they stir up sediment. By incorporating waves in the transportation formulae of the transverse slope model, which has not been done in the present study, it is likely that required velocities will have the order of magnitude that occurs in reality.
9 Conclusions

Amongst a multitude of other effects, the morphological impact of large scale marine sand extraction as planned within the framework of the Maasvlakte² project concerns erosion near pipelines, influencing of ecology near the sea bed, the different need for maintenance dredging of the Maasgeul and a change in coastal stability. In the present study, an attempt has been made to provide the necessary information for an assessment of the above mentioned impact, by making use of 2DV morphological models. Additionally, crucial aspects with respect to the applicability of the used models have been investigated.

9.1 Impacts on User Functions

9.1.1 Pipelines and Communication Cables

From NAP-19 to NAP-22, which can be considered the transitional zone between ‘deep’ and ‘shallow’ water, the effect on pipelines of a combination of a deep and a shallow sand extraction in the Maasgeul area appears to be limited by the presence of a sand dam. The slopes of the excavation area migrate at a speed of 4 and 8 meter per year for the computed deep and shallow variant, respectively. The migration rate will slightly increase in time as the depth of the sand extraction decreases. Nevertheless, the distance between the sand extraction and the nearest gas line can be considered safe, since the absolute value of the migration rate is small. This conclusion will hold for an arbitrary other sand extraction of a similar magnitude. At deeper water the model results and field data do not match well. The predictions on the basis of the model imply severe erosion at the place of a gas pipeline. Therefore, in spite of the discrepancy between the model results and data there is some reason for concern. Hence, the obligatory minimum distance of 500 m between the sand extraction and the pipeline should be reconsidered.

9.1.2 Erosion and Deposition in Relation to Sea Bed Ecology

Sedimentation in the sand excavation remains restricted to an area of 300 m width, at the slopes. There, deposition can locally reach up to 1.4 m/year in case of the most shoreward trajectory with the deepest excavation variant. The difference in local deposition rate between alternative variants is only noticeable in the most shoreward area, at approximately 19 m water depth. Hence, disturbance of bed fauna due to high sedimentation rates will be very limited and invariant to the shape of the sand extraction alternative. No investigation has been done on how sea bed ecology is influenced by the computed erosion and sedimentation rates.

9.1.3 Maintenance Dredging

The need for maintenance dredging will be reduced to zero for a very long period as a result of the sand extraction. To keep future maintenance costs as small as possible, it would be best to situate the sand extraction as far south as possible, in order to enlarge the period that the southerly slope needs to migrate to the area where the bottom level may not exceed the
prescribed value. A possible development that might frustrate this benefit, is the growth of sand waves.

9.1.4 Coastline Maintenance

The long-term effect of a large-scale marine sand extraction on the coastal zone is hard to predict. It has been shown that in case of a marine extraction at NAP-20, wave-induced as well as wind induced cross shore processes, under neap tide as well as spring tide conditions will not result in a substantial loss of sediment from the lower shore face to the inner shelf. The edge of the extraction area migrates in the onshore direction, but at a rate of only 1 to 2 m per year. Hence, the sand extraction will not lead to a decrease of the coastal stability as result of wave and wind-induced cross-shore processes. The effect of a change in the wave climate or the tidal motion has not been considered.

9.2 Model Applicability

The quality of the obtained model results is limited by the model formulations concerning bed slope effects. Comparing the SUTRENCH results with field observations leads to the conclusion that the influence of a slope on bed load transport rate is underestimated. This will also hold for the UNIBEST TC model, since basically the same approach is utilised. As a consequence, for the calibration of SUTRENCH model, the bathymetric data of the southern slope of the Maasgeul as well as the dredging information could be used in an indicative way only.

It has been shown on the basis of a simple model that the downhill transport component due to a current velocity of 1 m/s along the slope can lead to an erosion of the slope of the same order of magnitude as the deposition predicted by the SUTRENCH model. Waves stir up sediments and thus cause higher transport rates. Taking into consideration the fact that waves have not been incorporated in the model, the effect of a transverse slope on the sediment transport will be worth considering in areas with significant bed level gradients.

It is difficult to draw general conclusions on long term morphological behaviour of a marine sand extraction at deep water. Strictly long shore directed developments appear to be apparent the effect of cross shore processes seems to be negligible. Not only is the latter statement debatable since no field data have been used, but also might the long shore processes have a cross shore component. Therefore, on the basis of the present report it can not be derived whether the NAP-20 depth contour is an accurate border for the coast near edge of a large scale marine sand extraction.
10 Recommendations

1. An improvement of the applicability of both SUTRENCH and UNIBEST TC to deep water is recommended. In order to achieve this, especially the bed load transport formulations should be looked at. More specifically, the effect of a bed slope on the transport rate should be investigated, possibly in relation with the initiation of motion.

2. Additional research on the coastal stability is needed since the studied long shore and cross-shore behaviour can only partly explain the coastal respond to a large scale marine extraction.

3. Enforcement of field experiments at deep water in the form of depth measurement is desirable. By means of field experiments, information on the morphological activity at a depth larger then 25 m can be gathered. Without such information it will not be possible to determine whether the obligatory distance of 500 m between the a sand extraction and a pipeline suffices.

4. It is advocated to store as much data on depth measurement, on dredged amounts and on the compounding of sediments in the Maasgeul as possible, since it can be of great help when predicting morphologic developments.
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A  Dredging Amounts of the Maasgeul

### Annual Dredged Volumes of the Maasgeul [m³]

<table>
<thead>
<tr>
<th>Year</th>
<th>km 0 - 3</th>
<th>km 3 - 11.3 plus anchor area</th>
<th>Total</th>
<th>km 0 - 5</th>
<th>km 5 - 11.3 plus anchor area</th>
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<td>1,850,000</td>
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<td>60,460</td>
<td>936,165</td>
<td>1989</td>
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### Annual Dredged Volumes of the Maasgeul Specified per Kilometer [m³]

<table>
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<th></th>
<th></th>
<th></th>
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<td>2 to 3</td>
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<td>474,616</td>
<td>321,997</td>
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<td>322,014</td>
<td>291,919</td>
<td>39,941</td>
<td>70,024</td>
<td>162,678</td>
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<td>208,027</td>
<td>271,502</td>
<td>48,890</td>
<td>27,524</td>
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<td>5 to 6</td>
<td>279,051</td>
<td>18,952</td>
<td>35,352</td>
<td>132,376</td>
<td>55,514</td>
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<td>63</td>
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<td>66,790</td>
<td>71,982</td>
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<td>7 to 8</td>
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<td>2,373,558</td>
<td>2,633,603</td>
<td>1,365,741</td>
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</table>

### Annual Dredged Volumes of the Maasgeul Specified per Type of Extraction [m³]

<table>
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<td>Deepening</td>
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<td>2,939,644</td>
<td>2,455,911</td>
<td>1,719,267</td>
<td></td>
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</tbody>
</table>
B RGF Grid and Quick Inn

RGF GRID and QUICKIN can be used to visualise and manipulate a set of scattered x, y, z co-ordinates. Both programs primarily are developed to have a tool that can create a TRISULA bathymetry by interpolation of raw depth data onto a model grid. TRISULA is Delft Hydraulics’s simulation program for hydrodynamic flows and transports in 2 and 3 dimensions. Essential elements of the programs will be briefly described.

RGF GRID is a graphical user interface for generation and manipulation of orthogonal, curvilinear grids. One can start drawing a rough sketch of the intended grid by splines. Then the splines are transformed into a rough grid, that can be smoothly refined by the program. Various grid manipulation options are provided in order to put the grid in the right position with the right resolution.

The main purpose of the QUICKIN program is to create, manipulate and visualise model bathymetries. After having read the co-ordinates, or sample points, and the grid created in RGF GRID, the following operations can be performed:

Grid cell averaging: The method counts the number of sample points in the vicinity of a grid point, and calculates the average of these points. The dimension of the vicinity and a minimum number of averaging points can be specified.

Triangular interpolation: First, a triangle network is created over the entire x, y domain of the data. Every triangle within the network has a sample point in each corner only. Subsequently, the grid points are interpolated linearly in the identified triangles.

Internal diffusion: Grid points that have no depth yet, are modified in their value by the so called internal diffusion mechanism. Initially, grid points with no value are set equal to zero. The algorithm that subsequently adapts these points to their surrounding is the implementation of the diffusion equation. Basically, the mechanism is a smoothing process which is called repeatedly, but that does not change values of already existing depth values. Therefore, a smooth transition with the existing bathymetry is inherent. The number of internal diffusion steps can be specified.

Combination of bathymetries: Separate bathymetries on the same grid can be combined by summation or subtraction.
C Bijker Transport Formula

An extensive description of the Bijker formula can be found in [Van der Velde, 1995]. In what follows, a summary is given. Bijker developed a transport formula for a combination of currents and waves by adjusting existing sediment transport formulas. He used the Kalinske-Frijlink formulae for the bed load transport and the Einstein formula for the suspended sediment transport.

\[ S_b = 5D_{50} \frac{V}{C} \sqrt{\mu g} \exp \left[ -0.27 \frac{\Delta D_{50} \rho \omega}{\mu \tau_c} \right] \]  
\[ S_w = 11.6 \sqrt{\frac{\tau_c}{\rho}} 2D_{50} c_s \left[ I_1 \ln \left( \frac{33h}{r} \right) + I_2 \right] \]  

(C.1)  
(C.2)

Herein \( I_1 \) and \( I_2 \) are integrals depending on the relative bottom roughness \( A = r/h \) and the Rouse number \( z_\ast \), an index concerning the shape of the concentration distribution.

\[ I_1 = 0.216 \frac{A^{(z_\ast - 1)}}{(1 - A)^{z_\ast}} \int_{\lambda}^{1} \left( \frac{1 - \xi}{\xi} \right)^{z_\ast} d\xi \]  
\[ I_2 = 0.216 \frac{A^{(z_\ast - 1)}}{(1 - A)^{z_\ast}} \int_{\lambda}^{1} \left( \frac{1 - \xi}{\xi} \right)^{z_\ast} \ln(\xi) d\xi \]  

(C.3)

\[ z_\ast = \frac{w}{\kappa V_\ast} \]  
\[ A = \frac{r}{h} \]

Bijker carried out the following modifications:
1. The layer in which bottom transport takes place has a thickness that equals the bottom roughness, \( r \), instead of the \( 2D_{50} \) used by Einstein.
2. The ripple factor \( \mu \) is neglected in the ‘transport parameter’ (the first part) of the Kalinske-Frijlink formula.
3. The shear stress \( \tau_c \) in the ‘stirring parameter’ (the second part) of the Kalinske-Frijlink formula is modified to the shear stress for current and waves \( \tau_{cw} \):

\[ \tau_{cw} = \tau_c \left[ 1 + \frac{1}{2} \left( \frac{f_w}{g} \frac{\mathbf{u}_0}{\mathbf{V}} \right)^2 \right] \]  

(C.4)

Subsequently the Rouse number is being modified:

\[ z_\ast = \frac{w}{\kappa V_{cw}} = \frac{w}{\kappa \sqrt{\tau_{cw} / \rho}} = \frac{w}{\kappa \sqrt{\frac{gV^2}{C^2} \left[ 1 + \frac{1}{2} \left( \frac{\mathbf{u}_0}{\mathbf{V}} \right)^2 \right]}} \]
\[
\frac{w}{\kappa \left\{ \frac{gV^2}{C^2} \left[ 1 + \frac{1}{2} \left( \frac{f_w \bar{u}_0}{2gV} \right)^2 \right] \right\}^{1/2}} = \frac{w}{\kappa \left\{ \frac{gV^2}{C^2} + \frac{1}{4} f_w \bar{u}_0^2 \right\}^{1/2}}
\]  
(C.5)

Furthermore Bijker assumes the bottom transport to occur in a layer with a thickness equal to the bottom roughness, \( r \), and the concentration not to differ within the bottom layer. Using the Rouse-Einstein concentration distribution:

\[
c(z) = c_s \left[ \frac{h-z}{z} \right]^{1/2}
\]
(C.6)

and the Prandtl-Von Karman logarithmic velocity distribution:

\[
V(z) = \frac{V_s}{\kappa} \ln \left( \frac{z}{z_0} \right)
\]
(C.7)

where, according to Nikuradse, \( z_0 \) can be approximated by \( r/33 \), an expression for the total transport can be obtained which yields a direct relationship between \( S_b \) and \( S_c \):

\[
S = S_b + S_c
\]

\[
= S_b \left( 1 + 1.83Q \right)
\]
(C.7)

\[
= \left\{ \frac{5D_{50} \sqrt{g}}{C} \exp \left[ - \frac{0.27 \Delta D_{50} C^2}{\mu V^2 + f_w C^2 \bar{u}_0^2 / 4g} \right] \right\} \left\{ 1 + 1.83 \left[ 1, \ln \left( \frac{33h}{r} \right) + I_z \right] \right\}
\]

In which:

\[
\bar{u}_0 = \frac{\omega H}{2 \sinh(kh)} \quad \text{maximum horizontal velocity component just outside the boundary layer [m/s]}
\]

\[
\hat{u}_0 = \frac{\hat{u}_0 T}{2\pi} \quad \text{maximum near bed displacement [m]}
\]

\[
C = 18 \log(12r/h) \quad \text{Chézy friction coefficient [m}^{0.5} / \text{s]}
\]

\[
f_w = \exp \left( -5.977 + 5.213 \left( \frac{a_0}{r} \right)^{-0.194} \right) \quad \text{friction factor [-]}
\]

\[
\mu = \left( \frac{C}{C_{90}} \right)^{3/2} \quad \text{ripple factor [-]}
\]

\[
C_{90} = 18 \log(12r/D_{90}) \quad \text{Chézy coefficient related to } D_{90} \quad [\text{m}^{0.5} / \text{s}]
\]
SUTRENCH Morphological Model for Gradually Varying Flows with Waves

SUTRENCH morphological model computes the evolution in time of flow velocities, sediment concentrations and the bed level in a two dimensional vertical plane. Velocities are being calculated using the -so called- Profile method. The concentration field is being determined by solving the sediment continuity equation, using a finite element solution method. In this equation longitudinal diffusive transport and the time derivative have been omitted. The width of the flow, in the lacking dimension, is being introduced by assuming the local flow velocity and the local sediment concentration to be constant in the lateral direction. Bed levels are being computed by solving the cross section integrated sediment continuity equation using a LAX scheme. The most important equations will be briefly summarised. An extensive description can be found in [Van Rijn and Tan, 1985].

Flow Velocities

Along a trajectory, the water surface velocity is described by a simple first order differential equation which yields an exponential adjustment of the surface velocity to the equilibrium velocity.

\[
\frac{du_h}{dx} = \alpha_1 \frac{u_{h,e}}{h} - \alpha_2 \frac{u_h}{h} - \alpha_3 \frac{u_h}{b}
\]

(D.1)

with the boundary condition \( u_h = u_{h,0} \) at \( x=0 \)

In which:

\( u_h \) = flow velocity at water surface \([m/s]\)

\( h \) = water depth \([m]\)

\( u_{h,e} \) = surface flow velocity for equilibrium flow \([m/s]\)

\( \alpha_1, \alpha_2, \alpha_3 \) = empirical constants \([-]\)

\( b \) = flow width \([m]\)

The equilibrium surface flow velocity in the equation is being calculated by combining a logarithmic velocity profile and the continuity equation:

\[
Q = b \int_{z_0}^{h} u dz \\
\Rightarrow u_{h,e} = \frac{Ln\left(\frac{h}{z_0}\right)}{-1 + Ln\left(\frac{h}{z_0}\right) \frac{Q}{bh}}
\]

(D.2)

\( z_0 \) = zero velocity level (=0.03 \( k_s \)) \([m]\)

\( k_s \) = effective bed roughness for currents \([m]\)
Q = discharge \quad [m^3/s]

The water surface velocity along the trajectory then is determined. The vertical velocity profile is being described by a linear combination of a logarithmic and a perturbation profile, as follows:

\[ u = A_1 u_b \ln \left( \frac{z}{z_0} \right) + A_2 u_b \left( 2 \left( \frac{z-z_0}{h-z_0} \right)^{1/2} - \left( \frac{z-z_0}{h-z_0} \right) \right) \]  \quad (D.3)

In which:
- \( u \) = horizontal flow velocity at height \( z \) above bed \quad [m/s]
- \( A_1, A_2 \) = dimensionless variables

The \( A_2 \) variable can be determined by applying the boundary condition, \( u = u_b \) for \( z = h \) resulting in:

\[ A_2 = 1 - A_1 \ln \left( \frac{h}{z_0} \right) \]  \quad (D.4)

Apart from the continuity equation, an additional equation is required to solve D3. Analysis of flow velocity profiles measured in a channel perpendicular to the flow direction showed that the mid-depth velocity at each location is approximately equal to the mid-depth velocity of a uniform flow with the same flow velocity and water depth at that location [Delft Hydraulics, 1980a]. Making use of this approximation, the horizontal flow velocity profile is completely defined.

The vertical flow velocity is being determined applying the width integrated equation of continuity for the fluid:

\[ \frac{1}{b} \frac{\partial}{\partial x} \left( u \frac{\partial w}{\partial z} \right) + \frac{\partial w}{\partial z} = 0 \]  \quad (D.5)

The vertical flow velocity, \( w \) [m/s], can be calculated by integrating this equation:

\[ w = -\int_{z_b+z_0}^{z_b+z} \left( \frac{\partial u}{\partial x} \right) \frac{dz}{b} - \int_{z_b+z_0}^{z_b+z} \left( \frac{1}{b} \frac{\partial w}{\partial z} \right) (u)dz \]  \quad (D.6)

**Sediment Concentrations**

With respect to the sediment concentrations the computational domain is divided in two parts. In a small layer, of height \( z_b \) near the bed the concentration is constant; in the complementary plane in concentrations are calculated solving the sediment continuity equation. The concentration in the bed boundary layer is computed using next equation:

\[ c_{s,b} = 0.015 \frac{d_{50}}{a} \left( \frac{T_a^{1.5}}{D^0.3} \right) \]  \quad (D.5)
in which $D$ is a dimensionless particle parameter [Van Rijn, 1984a] and $T$ is the shear stress parameter, defined as:

$$T = \frac{(\alpha_{cw} \mu_c \tau_c + \mu_w \tau_w) - \tau_{cr}}{\tau_{cr}}$$  \hspace{1cm} (D.6)

in which:

- $\alpha_{cw}$ = wave-current interaction coefficient [-]
- $\mu_c$ = efficiency factor for currents [-]
- $\mu_w$ = efficiency factor for waves [-]
- $\tau_{cr}$ = critical shear stress [N/m²]
- $\tau_w$ = bed shear stress current [N/m²]
- $\tau_c$ = bed shear stress waves [N/m²]

The variables in equation D6 depend upon the following parameters:

- $h$ = water depth [m]
- $H_s$ = significant wave height [m]
- $T_p$ = absolute wave period of peak of spectrum [s]
- $\phi$ = angle between wave and main current direction [°]
- $D_{50}$ = median diameter of bed material [m]
- $D_{90}$ = 90% diameter of bed material [m]
- $k_{se}$ = current related bed roughness height [m]
- $k_{sw}$ = wave related bed roughness height [m]

Knowing how the variables depend on these parameters is of importance to sediment transport calculations. However, an overview of all the applied equations reaches beyond the scope of this summary. In the publication ‘Principles of Sediment Transport in Rivers, Estuaries and Coastal Seas’ [Van Rijn, 1993], the method of calculation has been extensively described.

The used sediment continuity equation for varying width reads:

$$\frac{\partial}{\partial x} (buc) + \frac{\partial}{\partial z} \left( b \left( w - w_s \right) c \right) - \frac{\partial}{\partial z} \left( b \varepsilon_{s,ew} \frac{\partial c}{\partial z} \right) = 0$$  \hspace{1cm} (D.7)

with, apart from the inlet specifications, the following boundary conditions:

- $c = c_{in}$, \hspace{1cm} at $z = z_s + z_a$
- $w_c + \varepsilon_s \frac{\partial c}{\partial z} = 0$ \hspace{1cm} at $z = z_s + h$

in which:

- $w$ = mean local velocity in z direction [m/s]
- $w_s$ = particle fall velocity [m/s]
- $\varepsilon_{s,ew}$ = sediment mixing coefficient for combined current and wave conditions [m²/s]
The sediment mixing coefficient for combined current and wave conditions is represented by a linear addition of the wave-related and current-related mixing coefficients, as follows:

\[ e_{c,w} = e_{s,c} + e_{s,w} \]  \hspace{1cm} (D.8)

\[ e_{s,c} = \text{current related sediment mixing coefficient} \quad [m^3/s] \]
\[ e_{s,w} = \text{wave related sediment mixing coefficient} \quad [m^3/s] \]

Based on the assumption that the flow gradually varies (\(dh/dx < 0.05, db/dx < 0.05\)), simple mixing coefficient distributions, as used for equilibrium conditions are being applied. The equation for the wave related sediment mixing coefficient reads:

\[ e_{s,w} = e_{s,w,\text{bed}} = 0.00065D_s^2 \alpha_{br} \delta \theta_{b,w} \quad \text{for } z \leq \delta \]

\[ e_{s,w} = e_{s,w,\text{max}} = 0.035 \alpha_{br} \frac{hH_s}{T_s} \quad \text{for } z \geq 0.5h \]

\[ e_{s,w} = e_{s,w,\text{bed}} + \left(e_{s,w,\text{max}} - e_{s,w,\text{bed}}\right) \frac{z - \delta}{0.5h - \delta} \quad \text{for } \delta < z < 0.5h \]

where:

\[ \alpha_{br} = \text{breaking coefficient representing the influence of breaking waves on the sediment mixing process} \quad [-] \]
\[ \theta_{b,w} = \text{peak orbital velocity at bed} \quad [m/s] \]
\[ H_s = \text{significant wave height} \quad [m] \]
\[ L_s = \text{significant wave height} \quad [m] \]
\[ T_s = \text{significant wave period} \quad [s] \]
\[ \delta = \text{thickness of near-bed mixing layer} \quad [m] \]

The current related mixing coefficient is modelled by an parabolic-constant distribution, in which the effect of waves on the mean current velocity profile is incorporated. The applied equation reads:

\[ e_{s,c} = e_{s,c,\text{max}} = 0.25\beta u_{*c} h, \quad \text{for } z \geq 0.5h \]

\[ e_{s,c} = e_{s,c,\text{max}} - e_{s,c,\text{max}} \left(1 - \frac{2z}{h}\right)^n, \quad \text{for } \frac{z}{h} < 0.5 \]

in which:

\[ \eta = -0.25 \frac{\theta_{b,w}}{u} + 2, \quad \text{for } 0 \leq \frac{\theta_{b,w}}{u} \leq 4 \]

\[ \eta = 1 \quad \text{for } \frac{\theta_{b,w}}{u} > 4 \]

\[ u_{*c} = \text{bed shear velocity for equilibrium conditions} \quad [m/s] \]
\[ \eta = \text{coefficient to take into account the effect of waves on the velocity profile} \quad [-] \]
\[ u = \text{mean current velocity} \quad [m/s] \]
\[ \beta = \text{ratio sediment mass mixing and fluid momentum mixing coefficients} \quad [-] \]
Bed Level Changes

After computation of the concentration field, bed level changes are calculated using the cross-section integrated continuity equation, assuming quasi-steady conditions. The equation then reads as follows:

\[
\frac{\partial b z_b}{\partial t} + \frac{1}{\rho_s (1 - p)} \left[ \frac{\partial S}{\partial x} \right] = 0
\]  

(D.12)

in which:

\[ z_b \] = bed level with respect to reference datum [m]
\[ p \] = porosity factor [-]
\[ b \] = width [m]
\[ c = \frac{1}{h} \int_{z_b + h}^{z_b + h} c \, dz \] = depth average concentration [kg/m³]
\[ S = S_b + S_s \] = cross section integrated total load [kg/m³]
\[ S_s \] = suspended load transport [kg/m³]
\[ S_b \] = bed load transport [kg/m³]
\[ \rho_s \] = sediment density [kg/m³]

Suspended load transport can be computed integrating the product of horizontal velocity and concentration:

\[
S_s = b \int_{z_b + a}^{z_b + h} (u c) dz
\]  

(D.13)

Finally, the bed load transport is being calculated as follows:

\[
S_b = abc_a \mu_a
\]  

(D.14)

\[ u_a \] = flow velocity at reference level (z=a) (-)
E UNIBEST TC

Model formulations of the UNIBEST TC model can be found in [Bosboom et al., 1997]. The model can be used to simulate flow velocities, water levels, sediment concentrations and bed level developments along an axis perpendicular to the coast, as a result wave, tidal and wind forcing. The model consists of five sub models:
- wave propagation model
- mean current profile model
- wave orbital velocity model bed load and suspended load transport model
- bed load and suspended load transport model
- bed level model

The wave propagation model computes the wave energy decay along a cross-shore ray including the effects of shoaling, refraction and energy dissipation. The mean current profile model as well as the wave orbital velocity model both are local models. The first computes the vertical distribution of the wave-averaged mean current in the long shore and cross shore direction accounting for wind shear stress, wave breaking, the tidal motion, bottom dissipation in the wave boundary layer and the slope of the free surface. The wave orbital velocity model calculates time series of the near-bed wave orbital velocity. These time series contain contributions due to wave asymmetry, wave group related amplitude modulations and bound long waves and are therefore representative for irregular wave groups.

In the sediment transport model one can distinguish between the bed load and suspended load model. The suspended load transport is assumed to be dominated by the transport as a result of the mean current; the suspended sediment flux is computed as the product of the wave averaged current and concentration profiles, which are obtained from the mean current profile model and a time-average advection-diffusion equation, respectively. The bed load transport is computed as a function of the instantaneous bed shear stress. The near-bed velocity signals, determining the instantaneous bed shear stresses, are composed of the generated time-series for the near bed wave orbital velocity plus the time averaged current velocity near the bed.
FLOAT TRACK IN THE EURO MAASGEUL AREA

FIG 4.3 Z2268

WL DELFT HYDRAULICS
Cross Sections of the Maasgeul at km 5.88

Transverse Direction [m]

Cross Sections of the Maasgeul at km 6.50

Transverse Direction [m]
Northern Slope of Trajectory MG4.65
Time [days]

Regression line: $y = -0.0037x + 81.476$

Northern Slope of Trajectory MG5.25
Time [days]

Regression line: $y = 0.0021x + 146.23$

Northern Slope of Trajectory MG5.88
Time [days]

Regression line: $y = 0.0015x + 168.98$

Northern Slope of Trajectory MG6.50
Time [days]

Regression line: $y = -0.0006x + 169.96$

Development of the Northern Slope of the Maasgeul

WL Delft Hydraulics
SEDIMENT TRANSPORT ALONG CROSS SECTION MG4.65

FIG 6.9

WL DELFT HYDRAULICS
Situatie zandwininput Q16C-1 voor de Verdiepte Loswallen

LONGSHORE TRACKS FOR LONG TERM SIMULATION

WL DELFT HYDRAULICS
Reference Situation

Marine Sand Extraction from NAP-20 m

Relative Sedimentation/Erosion

Depth [m]

SEDIMENTATION/EROSION DUE TO A 2 M EXCAVATION FROM NAP -20

FIG 7.10

WL DELFT HYDRAULICS
NECESSARY TIME FOR DISTURBANCES TO REACH 10 m DEPTH CONTOUR

FIG 7.12

Z2268

NL DELFT HYDRAULICS

[Ribberink and Roelvink, 1989]