



Submarine *LOPE* development

*of dredged trenches
and channels*

Final Thesis Report

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in cooperation with TU Delft and Hydronamic
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Preface

This report describes the research carried out as a graduation project in order to finish the study of Civil Engineering and Geosciences, Section of Hydraulic Engineering, at Delft University of Technology. This thesis attempts to bridge the gap between two separate disciplines with respect to their design approach of side slopes of dredged navigation channels and pipeline trenches. The focus will be on slope development in the first year, generally the period of interest of the contractor. During this period the focus will gradually shift from the geotechnical design approach, primarily based on initial slopes, and the morphological design approach, primarily based on rough backfilling rates in the long term. Therefore well established theories from both disciplines are described and numerical computer programs have been used to create a set of guidelines for slope design.

Because of the interdisciplinary approach some of the chapters may contain familiar material, while other chapters may present totally new information, depending on your particular theoretical background. Please do not fear the unknown, as one of the purposes of this thesis is to bring separate disciplines to each other. The content should be comprehensible to any civil engineer and should give insight in the main instability mechanisms and the morphological behaviour of side slopes. Engineers of both disciplines should take advantage of knowing the complete picture.

The quick reader is referred to Chapter 12 in which guidelines for slope design are presented. If questions with respect to certain topics arise, one could easily study the relevant chapter in more detail.

The very broad subject and the many aspects that are involved in submarine slope design forced the author to study many different sources and researchers. Especially the lack of theoretical knowledge on soil mechanics made this research time consuming. The wide range of soils and hydrodynamic conditions forced me to constantly keep all possibilities open. Besides, the application of four different, not always very transparent computer models took a lot of time, because each program asks for a proper understanding of the input parameters, a large number of simulations and a sound processing of the results.

But, as a certain soccer phenomenon tends to say, every disadvantage has its advantage. This broad subject gave me the opportunity to increase my knowledge on multiple research fields. I think this thesis provides a useful overview of important aspects of submarine slope design, but I realize that it is just a first attempt to combine two disciplines in order to formulate a set of guidelines that should be helpful in designing side slopes. It is obvious that every single subject deserves to be deepened, although that was not possible in the framework of this thesis.

Of course I am most grateful to my supervisors of the examination committee, consisting of Prof. Dr. Ir. M.J.F. Stive, Prof. Dr. Ir. F. Molenkamp, Dr. Ir. J. van de Graaff and Ir. Karoune Nipius, for their advice and discussions on this topic and their remarks on this report.

My gratitude goes also to various researchers of both disciplines, who were willing to answer my many questions or to discuss the topic, as there are in arbitrary order, Theo Stoutjesdijk, Maarten de Groot, Stan Schoofs and Piet Meijers of GeoDelft, Dirk-Jan Walstra, Dick Mastbergen and Professor Van Rijn of WL|Delft Hydraulics, Jacob Jenssen and Erik Madsen of the Technical University of Denmark, Jan Rik van den Berg of the University of Utrecht, Annette Zijderveld of RIKZ, Professor Verruijt, Professor Stelling, Ronald Brinkgreve and Mark Voorendt of Delft University of Technology, Chris Dykstra, François Mathijssen and Peter Hendrickxs of Hydronamic, all other writers of the reference sources, my colleagues of Hydronamic, my fellow graduate students and all who I have forgotten to mention.

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Tim Raaijmakers

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Summary

In shallow areas like coastal seas or harbours the depth is not sufficient for present-day navigation. Because vessel sizes are growing, existing harbours and approach channels have to be deepened. Also the growth of the global population results in an increasing world trade and a growing demand on new harbour areas. Furthermore, the ever increasing energy consumption and the advancing technology of the oil industry together make it possible to drill for oil in deeper water. Pipelines, connecting different oil platforms with each other or an oil platform with the coast, become longer and longer. Often it is necessary to bury these pipelines in a trench.

These two examples show the already big, but still increasing, importance of dredging works in a marine environment. In both cases a sound determination of the required depth and width has to be done. In addition to that, the angle of the side slopes has to be determined. The designer should be able to give an estimation of the probability of failure or the maintenance costs. Two rather separate disciplines, coastal morphology and soil mechanics, have to be combined in slope design.

Although slope angles to a large extent determine on the one hand the slope stability and on the other hand the volume to be dredged, not only at the beginning (capital dredging), but also during maintenance dredging, it is still hard to predict whether a submarine slope is stable and how a slope will develop in time. Therefore slopes are often dredged as steep as geotechnical stability allows. However, slopes should not only be geotechnically stable, but also morphologically, at least for some time or within certain boundaries. So slope design is not only dependent on soil properties, but also on hydrodynamic conditions (currents and waves).

The aims and objectives of this thesis are to gain insight in:

1. submarine slope development in dredged trenches and channels;
2. the relevant processes and failure mechanisms;
3. the importance of soil mechanics and morphology and the (possible) interaction between them;
4. the behaviour of different types of soil (sand-silt-clay) with different particle diameters and soil properties;

with the following restrictions:

- a. the period of interest runs from the end of the dredging activities until the expire date of the period of guarantee (up to one year).
- b. the area of interest is situated at sea outside the sheltered area of a harbour (breakwaters) in regions with significant current and wave action;
- c. the depth of interest (= channel depth) will be between 5 and 20 m;
- d. the soil types of interest are uncemented, homogeneous sand, silt and clay.

Research teaches that slope development can be divided into three categories:

1. macro-instability, which can be subdivided into macro shear failure, liquefaction flow slide and retrogressive breaching;
2. micro-instability, (failure of the outer grains);
3. morphological development, which is caused by sediment transport due to current and waves.

While macro-instability almost always can be interpreted as failure, micro-stability is allowed under certain circumstances and morphological development simply is something to deal with. A set of 15 *reference soils* was composed: five different grain sizes ($d_{50} = 2, 50, 200, 500, 1000 \mu\text{m}$) and three different porosities ($n = 0,35; 0,40; 0,45$), because these two soil properties are most important regarding respectively morphology and soil mechanics. All other soil properties are related to these properties and deduced from literature or calculated from simple correlations. Also two reference geometries were determined: a typical navigation channel and a typical pipeline trench.

Macro shear failure ('slip circle analysis') was studied with the computer program MSTAB. For all reference soils Safety Factors were determined in drained (sand, silts and clays) and undrained calculations (clays). Calculations with MSTAB by GeoDelft show that in situations, where no external loads are present, stability in cohesive soils is governed by macro-stability and in non-cohesive soils by micro-stability, unless weaker soil layers or a water level somewhere through the slope are present. Then the slip circle is attracted to these local weaknesses.

Loosely packed sands are susceptible to liquefaction that can result in a flow slide. Calculations for static liquefaction were executed with SLIQ2D for three different slope heights and multiple porosities.

Retrogressive breaching is only likely to occur in slopes of medium to densely packed fine sand or silt ($d_{50} < 300\mu\text{m}$) that are steeper than the angle of internal friction. Although this instability mechanism controls the dredging process, it will not occur on the side slopes of a navigation channel, which (after dredging) are often not steeper than 1:3. Combinations of failure mechanisms 'liquefaction flow slide' and 'breaching' can occur, for instance a breach that starts at a steep scar produced by a small flow slide and retrogrades for many hours.

Although many possible external loads may occur in nature, only wave loads are considered in this thesis. Many researchers distinguish between the direct, elastic effect and the indirect, plastic effect. The first approach predicts the fluctuation of water pressures and effective stresses within one wave cycle. This approach has no net effect on the mean stresses and it is shown that this direct effect reduces Safety Factors up to 25%, but can cause no failure of a flat seabed. The second approach takes cyclic compaction into account, which causes a gradual water overpressure. It depends on the consolidation properties of the soil whether these excess pore water pressure (EPP) can attenuate or will build up during a storm. Calculations were done in MCYCLE for different reference soils, storms and 2-layer soil systems. Especially loosely to medium packed soils can collapse. Improving the soil properties (e.g. vibro-compaction) appeared to be more effective than flattening the slopes.

Currents and waves that cross a channel will adapt to the expansion in depth. Important phenomena are flow deceleration due to continuity, flow acceleration due to equal driving forces, gradual flow refraction, deviation from logarithmic velocity profiles, distortion of the velocity profile and separation bubbles at steep upstream side slopes, 'inversed' wave shoaling, wave refraction or reflection et cetera.

A sloping bottom affects the threshold of motion ('Shields' for currents and 'Sleath' for waves); a reduction factor for all incoming flow angles, slope angles and angles of internal friction was defined.

Also a so-called 'threshold profile' (no sediment transport anywhere along the slope) was calculated that will develop under very mild hydrodynamic conditions, but also has an indicative meaning for stronger currents, because sediment transport is based on the difference between actual bed shear stresses and threshold values. The idea of 'dynamic equilibrium slopes' is based on the idea that for currents perpendicular to the channel axis the bed load transport is constant along the entire slope.

The width-averaged 2DV-program SUTRENCH developed by WL|Delft Hydraulics was applied to model the morphological development of the 'navigation channel'. Besides a sensitivity analysis, simulations with increasing resemblance to field conditions were executed. Variation of initial side slopes teaches that side slopes converge during the first three months. It was shown that steepening of the upstream slope can occur, when a significant hydrodynamic forcing is present and bed load transport has a large share in total sediment transport.

Simulations with a real tidal and wave climate yielded three important conclusions. Backfilling can well be predicted when using one representative wave height instead of a full-year wave climate. This representative wave height is rather large. Slope development is governed by the largest waves. A simulation over only the period in which these largest waves occur yields accurate slope development. Furthermore it appeared that the order of the waves throughout the year does not affect morphological development, if one is interested in one or more years; of course some variations during the year are visible.

Based on all findings of this research it was tried to improve slope design. Five different slope geometries were evaluated on six criteria. It depends on the soil properties which criteria should have the largest weights. Favourable morphological behaviour can be attained as well as a reduction of the dredging costs or an increase of slope stability.

It can be concluded that in this thesis a wide range of topics related to side slopes of channels has been discussed. Simulations gained insight in slope instabilities and morphological behaviour. Some easy applicable guidelines were presented, but sometimes the theoretical background is still weak. Most important recommendations concern a more accurate modelling of the hydrodynamics (current and waves combined) at steep side slopes and the wave pressures in the water and in the seabed, especially under non-sinusoidal waves. A possible set-up of a sophisticated computer model and of a laboratory experiment are presented. In dredging practice dielectric measurements should be applied more frequently to obtain in-situ porosities. Data from all channel and trench projects should be collected in a database to increase knowledge and amount of fitting data.

1 Introduction

1.1 General

In shallow areas like coastal seas or harbours the depth is not sufficient for present-day navigation. While vessel sizes are growing, existing harbours and approach channels have to be deepened. Also the growth of the global population results in an increasing world trade and a growing demand on new harbour areas.

Furthermore, the ever increasing energy consumption and the advancing technology of the oil industry together make it possible to drill for oil in deeper water. Pipelines, connecting different oil platforms with each other or an oil platform with the coast, become longer and longer. Often it is necessary to bury these pipelines in a trench. Although 'pipe laying companies' often have their own techniques, also dredging of trenches (and landfalls in particular) occurs.

The two examples, mentioned above, show the already big, but still increasing, importance of dredging works in a marine environment. Although the size and time scale (pipeline trenches only have a temporary function; after the pipe is placed, the trench may silt up or will be backfilled) of both examples are quite different, there are a lot of similarities. In both cases a sound determination of the required depth and width has to be done. In addition to that, the angle of the side slopes has to be determined. The designer should be able to give an estimation of the life expectancy to gain insight in the probability of failure or the maintenance costs.

This means many parameters have to be investigated, like current conditions, wave climate, sediment transport and soil conditions. In this thesis the difficulties with respect to slope design will be studied.

1.2 Problem description

It appeared that slope design forms an important part of the overall channel design, but what exactly makes it so difficult? To answer that question first a short description of the present engineering practice will be given.

When designing a navigation channel, usually attention is given to:

1. track and alternatives;
2. channel dimensions (width and depth) depending on vessel sizes, ship movements, keel clearances, local regulations and sedimentological conditions;
3. capital and maintenance dredging volumes;
4. side slopes.

The fourth point of attention, the side slopes, brings many difficulties along. Although slope angles to a large extent determine on the one hand the slope stability and on the other hand the volume to be dredged, not only at the beginning (capital dredging), but also during maintenance dredging, it is still hard to predict whether a slope is stable and how a slope will develop in time.

Another remarkable thing is that research on slope development is conducted by either the soil mechanics department or the morphological department. Nowadays the first approaches between the two research groups are made, for example in Delft Cluster on sand borrowing pits and modelling of turbidity currents.

In engineering practice it is found very difficult to give an estimation of the relevant processes and on what time scale slope development takes place. Therefore slopes are often dredged as steep as the geotechnical stability allows.

However, slopes should not only be geotechnically stable, but also morphologically, at least for some time or within certain boundaries. Besides, the design of a slope will influence the erosion/sedimentation pattern of the entire channel and therefore the capital and maintenance dredging strategy. Predominantly, a steeper slope will decrease the morphological impact (less sedimentation), because of decreasing width and corresponding trapping efficiency. On the other hand, steeper slopes will be more susceptible to instability mechanisms. Therefore the pros and cons have to be balanced against each other to come to an optimal design. And this design isn't only dependent on the soil properties, but also on the hydrodynamic conditions: currents and waves and their orientation to the channel.

Of course, the above considerations not only concern navigation channels; trenches for pipelines or tunnels are subjected to similar points of attention, although other processes and mechanisms will be normative.

Definition of the problem:

Designing slopes of trenches and channels is still too often based on dredging experience or limited calculation methods. Besides, there is a lack of knowledge of slope development in time, because most of the times the design is focused on the stability of the initial slope.

1.3 Aims and objectives

The aims and objectives of this thesis are:

To gain insight in:

1. submarine slope development in dredged trenches and channels;
2. the relevant processes and failure mechanisms;
3. the importance of soil mechanics and morphology and the (possible) interaction between them;
4. the behaviour of different types of soil (sand-silt-clay) with different particle diameters and soil properties.

With the following restrictions:

- a. the period of interest runs from the end of the dredging activities until the expire date of the period of guarantee (up to one year). During this period significant slope development will occur, but the focus will not be on maintenance strategies, which have longer time scales, see Figure 1-1. Please note that the 'final slope' is of course just a snapshot; morphological development does not just stop here;

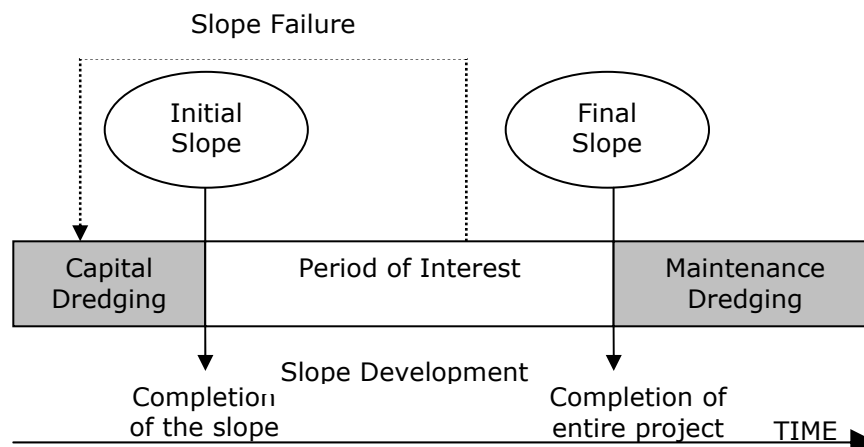


Figure 1-1: Schematization of 'period of interest'

- b. the area of interest is situated at sea outside the sheltered area of a harbour (breakwaters) in regions with significant current and wave action;
- c. the depth of interest (= channel depth) will be between 5 and 20 m outside the breaker zone;
- d. the soil types of interest are uncemented sand, silt and clay;
- e. the soil will be completely homogeneous, so no separate layers are present.

Therefore first a literature study has been done to study the known instability mechanisms. This knowledge is used to design initial stable slopes in several reference cases; this design is mainly based on soil mechanics. Then the development of these slopes will be studied with the help of a computer program (SUTRENCH). Conclusions are drawn which channel geometry shows the least or favourable morphological changes and meets the requirements best during the period of interest.

1.4 Outline of the report

In this thesis research is done on what the dominant processes in different situations are. Therefore a literature study was done on all known instability mechanisms; a short summary will be given in Chapter 2. From these mechanisms the relevant mechanisms for channels and trenches are derived. Also in Chapter 2, the experiences of present-day dredging practice will be sketched; how are slopes designed in engineering practice and with what difficulties the contractors were confronted. In Chapter 3 the reference cases of this thesis are presented. A set of reference soils and two reference geometries will be composed that will be used throughout the calculations. These reference soils are varying in grain size and density. The reference geometries represent a typical navigation channel and a typical pipeline trench. Then, the instability mechanisms will be treated in a logical order. At first the instability mechanisms from soil mechanics are described. Chapter 4 discusses the unloaded macro-stability and micro-stability. Macro-stability is subdivided into shear failure, liquefaction flow slides and retrogressive breaching. In Chapter 5, submarine slopes are subjected to dynamic loads. In this thesis of all possible loads only wave loads are considered.

From Chapter 6 morphology comes into play. First a description of hydrodynamics in trenches and channels is given in Chapter 6. Then the threshold of motion under currents and waves is determined for all reference soils in Chapter 7. If particles once started to move, sediment transport has to be considered. Chapter 8 discusses the application of the Van Rijn transport formulae and the morphology of a trench or channel.

With a numerical model (SUTRENCH) some calculations will be done. In Chapter 9 this model will be explained and in Chapter 10 the results of a sensitivity analysis and simulations with unidirectional and tidal flow will be discussed. In Chapter 11 with the newly gained knowledge it is tried to improve the slope design with a view to decreasing maintenance dredging volumes, slope migration or flattening or increasing slope stability. Some guidelines for slope design in engineering practice are presented in Chapter 12. Finally conclusions and recommendations will be treated in Chapter 13.

2 Mechanisms of slope instability

2.1 Known mechanisms of slope failure in literature

Slope stability and failure has been the subject of intensive research for the last decades. Since the rather pessimistic quote of Ralph B. Peck (co-author of 'Soil mechanics in engineering practice' [Terzaghi et al.]) in 1967 a lot of progress is made: "We have lost much of our confidence in our ability to predict the behaviour of a natural hillside or in the results of our remedial measures....it is evident that nature was able to outwit us and we fear she can and will do so on similar occasions in the future. This, I submit, is the present state of the art."

However, still many questions arise when a slope has to be designed. Especially in marine conditions, where morphology can be just as important as soil mechanics, theoretical foundations of a slope-design often are insufficient.

Subaqueous slope failures occur at various scales in different environmental settings. Schwarz (1982) investigated 286 reported failures of subaqueous slopes covered with uncemented sediment. It became clear that classification is very difficult and that the important variables show a huge spread.

Slide structures

A slope failure can be defined as a gravitational deformation of sediment mass (or rock) leading to a smaller slope angle. These slope failures can be divided into five groups with increasing water content, see Table 2-1.

Table 2-1: Type of slope failures sorted by water content [source: Schwarz, 1982]

Slope Failures		
Water content ↑	Creep	No visible break of particle contacts
	Fracturing / Folding	Break or bending of (an)isotropic beds
	Slide	Block movement with restricted internal deformation
	Mass Flow	Viscous, mostly laminar flow
	Turbidity Current	Viscous, turbulent flow of concentrated suspension

Creep is a very slow type of granular flow and is measured in millimetres or centimetres per year. Fracturing and folding mainly occurs in rocky beds and rocky soil is beyond the reach of this thesis. (Sediment) mass flows can be divided into two classes with increasing sediment concentration (Figure 2-1): (I) slurry flow is a moving mass of water-saturated sediment and (II) granular flow is a mixture of sediment, air and water. This flow does not have to be water-saturated and is supported by grain to grain contact. In submarine conditions (saturated with water) of course only the first one will occur. The well-known debris (loose, heterogeneous material coarser than sand) and mud flows belong to this category of slurry flows.

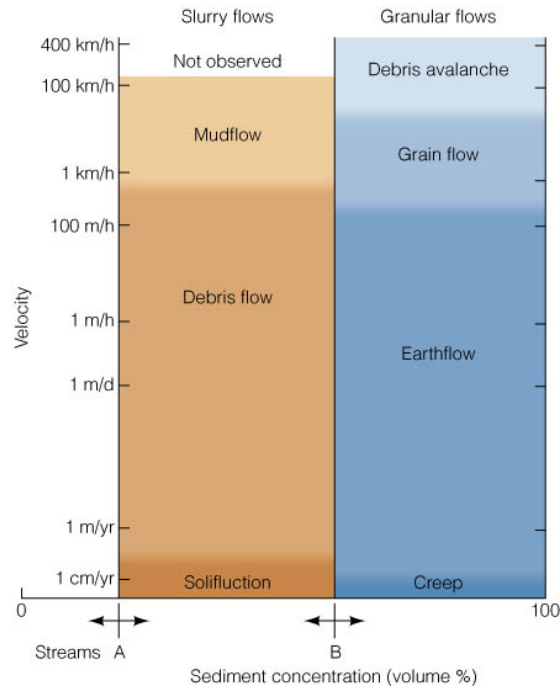


Figure 2-1: Division of mass flows into slurry flow and granular flow
[source: www.earthsci.org/flooding]

Turbidity currents appear for instance during land reclamation and dredging activities (breaching). In this thesis the focus will be on slides. Slides are the most common slope failure mechanisms and can be divided into four subgroups, see Table 2-2. The most common slide is the so-called slump, resulting in a more or less circular slip surface. In this thesis the emphasis will be on this type of slides, which occurs in homogeneous, isotropic soils, unlike the translational planar block slides. Plastic behaviour is typical for a convolute slide.

Table 2-2: Types of slide structures [source: Schwarz, 1982]

Slide Structures	
Minor Block Glide	Shear or break of small single blocks out of undisturbed beds
Rotational Block Slide (=Slump)	Shear of sediment mass along a concave main shear surface. Occurrence in homogeneous, isotropic deposits without or with weak influence of bedding
Translational Planar Block Slide	Shear of larger sediment blocks conformable to well-bedded (anisotropic) deposits
Convolute Slide	Shear of a well-bedded (anisotropic) sedimentary complex under plastic deformation along a curved shear surface.

Another classification in grain-fluid mixture flows is adopted in Mastbergen et al. [1988] with the help of two dimensionless hydraulic scale numbers (Table 2-3):

- Reynolds-number (Re) which is the ratio of the turbulent and the viscous shear stress and distinguishes between laminar and turbulent flow;

$$Re = \frac{uh}{\nu_m} = \frac{q}{\nu_m} \quad \text{Eq. 2-1}$$

in which 'u' is the average flow velocity, 'h' the water depth and ' ν_m ' is the kinematic viscosity of the soil-water-mixture.

- Bagnold-number (Ba) which is the ratio of the inertial and the viscous shear stress and distinguishes between viscous and inertial flow. This is an indicator whether fluid or grain properties are dominant.

$$Ba = \frac{V_g}{1 - V_g} \frac{\rho_s d^2 \gamma_m}{\mu} \quad \text{Eq. 2-2}$$

in which ' V_g ' is the volume of the grains, ' d ' is a representative grain diameter, ' γ_m ' is the shear strain rate of the soil-water mixture [unit: 1/s] and ' μ ' is the dynamic viscosity of the fluid.

A small Bagnold-number ($Ba < 40$) characterizes the regime of macroviscous flow, where the viscous interaction with the pure fluid is important. For a large Bagnold-number ($Ba > 450$) the flow is in the 'grain-inertia regime' where the grain-grain interactions are dominant.

Table 2-3: Division of grain-fluid mixture flows related to Reynolds- and Bagnold-number [source: Mastbergen et al., 1988]

		Reynolds number (Re) ----->			
		Laminar	Turbulent		
<----- Bagnold number (Ba)	Viscous	Poiseuille or Couette flow Mudflows	Turbulent flow Suspended sand transport	Relative grain density ----->	Grain size -----> Concentration -----> Admission of water ----->
	Macro viscous	Macroviscous grain flows Flow slides	Turbidity currents Hyperconcentrated suspension flow		
	Inertial	Inertial grain flows Dry sand flow Bed load sand transport	Rock avalanches Bed load sand transport		
		Mixing length -----> Specific flow rate -----> <----- Viscosity			

Of all mentioned grain-fluid mixture flows by Mastbergen et al. [1988], this thesis will mainly focus on five of them, which will be only very briefly explained here to get an idea of the connection between those types [De Groot, 2004]:

1. Grain flow
2. Turbulent density flow or turbidity current
3. Viscous density flow or flow slide
4. Bed load (sand) transport
5. Suspended (sand) transport

Grain flow (I) occurs when a small number of grains are falling from a breach or a slope equal to or steeper than the natural slope. The grains are jumping and sliding over the seabed, just like bed load transport (IV). In both cases the water pressure is hydrostatic, but the driving force of a grain flow is only gravity, whereas bed load transport is mainly controlled by hydrodynamic forces like current and waves. If more grains are falling from the breach, they can entrain so much water that a turbulent sand-water mixture comes into existence. This sand-water-mixture behaves as a separate, dense fluid with only limited exchange with the surrounding water. The weight of the sand grains is carried by excess pore pressure (which will be explained later on), so no contact forces between the grains exist. This kind of flow is more similar to suspended transport, because turbulence (in combination with the higher sand concentration near the bed) keeps the grains in suspension, while gravity determines the settling process. The most important difference between turbulent density flow

(turbidity current) and suspended transport is the sand concentration, which is much smaller for the latter type of flow. This has a major consequence for the erosion capacity of the flow: the larger the sediment concentration, the larger the bed-shear stress, the larger the erosion.

The grain-fluid-mixture flow with by far the largest grain concentration is the viscous density flow, which occurs during a flow slide and has about the same void ratio of the not-yet collapsed (loose) soil. This flow is characterized by its viscous and slow process.

A special kind of grain-fluid flow that can develop in grain flow and turbulent density flow under specific circumstances is uniform flow. As illustrated in Figure 2-2 these specific circumstances are a constant flow velocity, layer thickness and concentration. Because of the concentration-dependent erosion capacity, the *slope angle of uniform flow* (β_{uniform}) decreases for larger concentrations: a turbulent density flow of higher concentration, and thus higher erosive capacities, is in equilibrium for smaller slope angles.

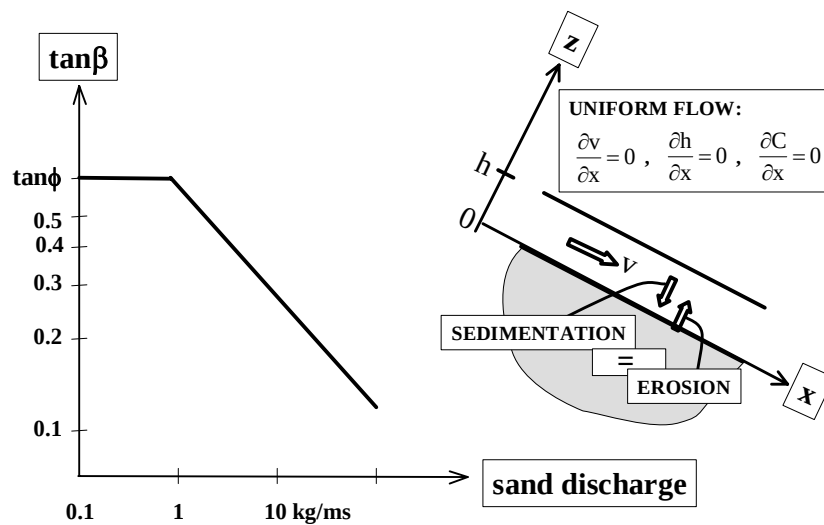


Figure 2-2: Uniform flow; grain flow for sand discharge <1 kg/ms; turbulent density flow for sand discharge >1 kg/ms [source: De Groot, 2004]

Once a grain-fluid flow of type II and III has come into existence it can sustain for a rather long time because of a process of *hindered settling*. Whereas sand in low concentrated flows have typical fall velocities of a few centimetres per second, this fall velocity can be reduced to less than a millimetre per second in high sediment concentrations.

Regions

An attempt was made to classify each failure using specific regions exposed to more or less the same geomorphologic, hydrodynamic and depositional conditions. As can be seen in Figure 2-3 most reported slope failures occur at continental slopes, canyons and deltas. One should keep in mind that this figure is only a rough estimation, because in the past there have been many slope failures which have not been reported.

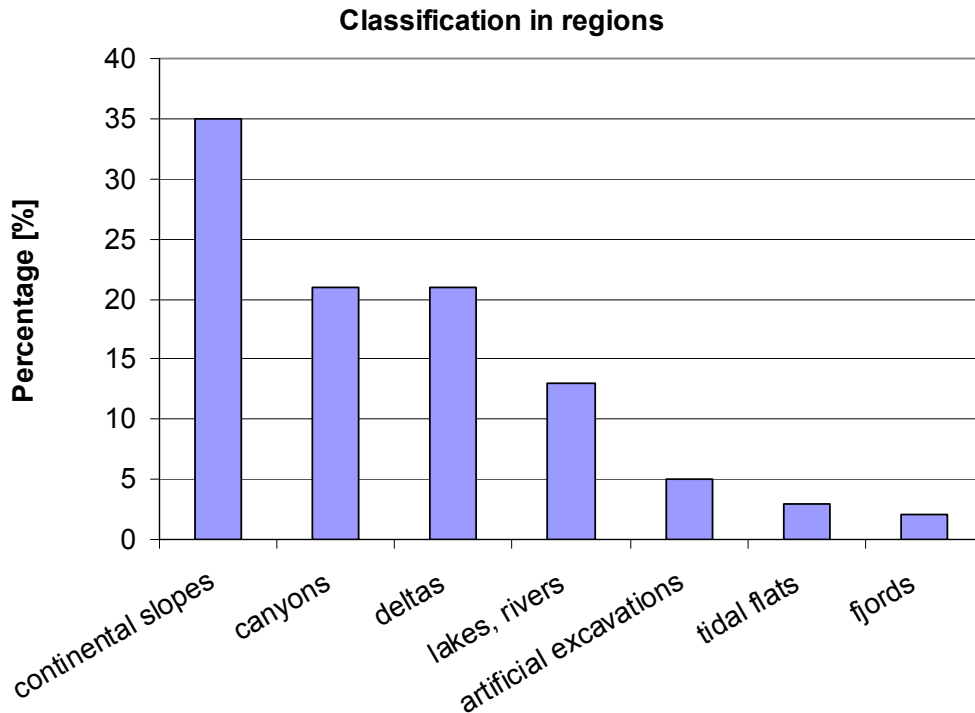


Figure 2-3: Percentage of reported slope failures; classification in regions
[source: Schwarz, 1982]

Summarizing all of the reported slope failures, a topographic schematization has been plotted in Figure 2-4. In this thesis we are especially interested in failures on the continental shelf and marine deltas.

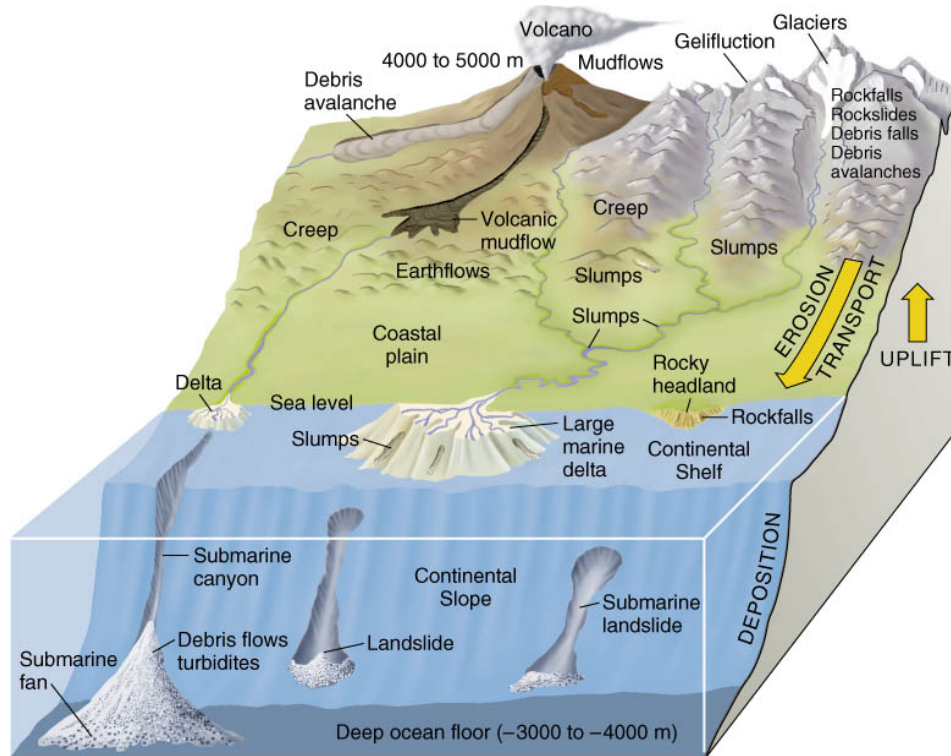


Figure 2-4: An overview of all kinds of slope failures
[source: faculty.etsu.edu/wallaces/MassW.pdf]

Slope angles

When analyzing slope angles at which failures occur, a very broad scattering is found with values less than $0,5^\circ$ (1:115) up to values of 50° - 60° (1:0,8 to 0,6). Considering large slope failures it becomes clear that even very flat slopes can collapse.

Classification in slope angles

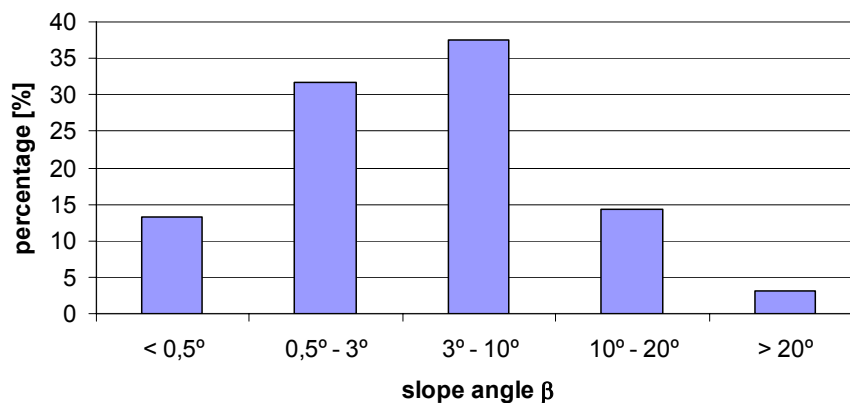


Figure 2-5: Percentage of reported slope failures; classification in slope angles
[source: Schwarz, 1982]

Release mechanisms

The transition from a stable slope to an unstable slope can have different causes. In literature nine so-called release or 'trigger' mechanisms have been reported, varying from gradual releasing causes to very sudden events. These nine causes, however sometimes difficult to distinguish (or overlapping), are:

1. Accumulation
 - a. long-termed high sedimentation rate in case of slowly progressing delta front areas or steep trench slopes. This mechanism is often combined with an additional trigger effect;
 - b. short-termed heavy sediment supply because of large river floods;
 - c. overloading of insufficiently consolidated slope and near-slope areas because of loss of buoyancy (tidal flats) or strong accumulation at river mouths;
 - d. oversteepening of a depositional slope at a shelf break or at a trench slope, possibly increased by tectonic movements;
2. Earthquakes, probably the most effective sudden trigger mechanism;
3. Tectonical reasons. In tectonically active zones favourable conditions for slope failure can be created, although these processes normally are too slow to speak of a 'real trigger mechanism';
4. Currents and waves
 - a. normal currents, just able to move some sediment, especially at former periods of regression, the activity of currents and waves was concentrated to a narrow seam near the shelf break, which could have formed a spill of shelf sediment;
 - b. hurricanes and large waves in delta areas and at canyon heads;
 - c. tsunamis and seiches;
5. Emergence (sea level change). Dropping of the water level implies loss of buoyancy and an increase of effective load on a potential shear surface. In tidal flat areas or during Pleistocene low sea levels this can cause slope failure.
6. Change of certain physical-chemical properties
 - a. increase of sensitivity due to leaching (*Dutch*: uitloggen) of marine cohesive sediments by freshwater influence is a well-known mechanism responsible for the quick clay effect;
 - b. subterranean influx of freshwater into marine sediments results in a weakening of the sediments;
 - c. chemical decomposition of organic or inorganic sedimentary components;
7. Pore water overpressure in an unconsolidated deposit diminishes its shear resistance. This can be caused by different mechanisms, like tectonical stress, rapid sediment accumulation, loss of buoyancy and development of gas (methane) from decomposition of organic debris;
8. Erosion as a trigger mechanism means undercutting of slope deposits, for example along river banks or tidal gullies;
9. Man-made slope failures especially occur during constructing of harbours, dams or dredging works. In this respect also artificially dumped (waste) deposits are mentioned.

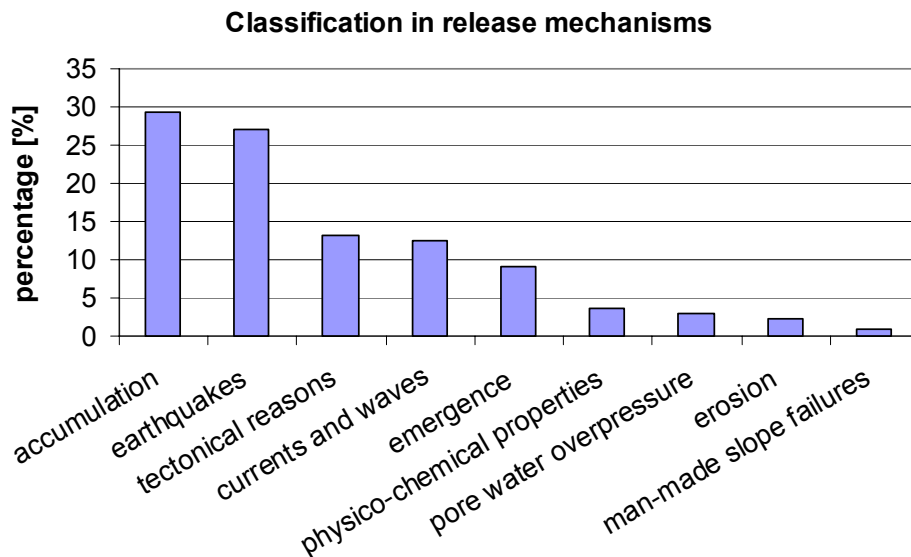


Figure 2-6: Percentage of reported slope failures; classification in release mechanisms

[source: Schwarz, 1982]

One should keep in mind, that in many cases more than one of the above release mechanisms are responsible for slope failure. So Figure 2-6 is only meant to get a general idea. Some of the mentioned mechanisms are related to each other. Currents and waves for example can cause gradual accumulation or erosion, but can also act as a sudden trigger mechanism, like for instance a storm wave or tsunami.

2.2 Relevant mechanisms of slope instability in trenches and channels

In the previous paragraph a lot of slope failures and trigger mechanisms were classified in different categories. Although these failures are characterized by various time- and space-scales, it appears that they all can be combined to a few basic instability mechanisms, which are relevant in the time- (up to one year) and space-scales (few hundred meters) of this thesis.

In Figure 2-7 an overview of these mechanisms is given. Usually a distinction is made between micro- and macro-stability. With micro-stability the stability of the outer grains is meant, in contrast to the stability as a whole, the macro-stability. The stability of the outer grains is strongly depended on the working forces, like for example seepage or the wave force by short waves. Normally in soil mechanics, the term 'micro-stability' is only adopted when speaking of instability due to forces from inside the sand or clay body, like ground water flow or water pressures due to seepage or passing waves. When these forces are induced by waves or currents from outside the body and sediment is picked up, we speak about 'morphology'. Now, 'instability' means transport of particles. As long as the channel profile stays within certain limits, sediment transport is allowed. In the morphology-approach it is not the question whether the soil is 'stable' (no movement at all) but for how long the channel or trench will fulfil its purpose.

Unlike subaqueous slopes, slopes above or around the waterline like embankments or dikes should be stable under all circumstances. Therefore often an armour layer is applied to protect the underlying soil. As mentioned above, slopes of trenches and channels may experience some profile changes as long as their functionality (required profile) isn't endangered.

As a rough design criterion, it is stated that:

- macro-instability should be prevented at all times, because large movements of soil will almost certainly cause failure of the trench or channel;
- micro-instability can be allowed under certain circumstances, like for instance storm waves. One should however be very careful, because small failure can result in larger failure, as will be explained later on;
- transport of sediment will be allowed, although a sound determination of the expected siltation and migration of the profile as a whole and the slope in particular should give insight in improvement of design methods.

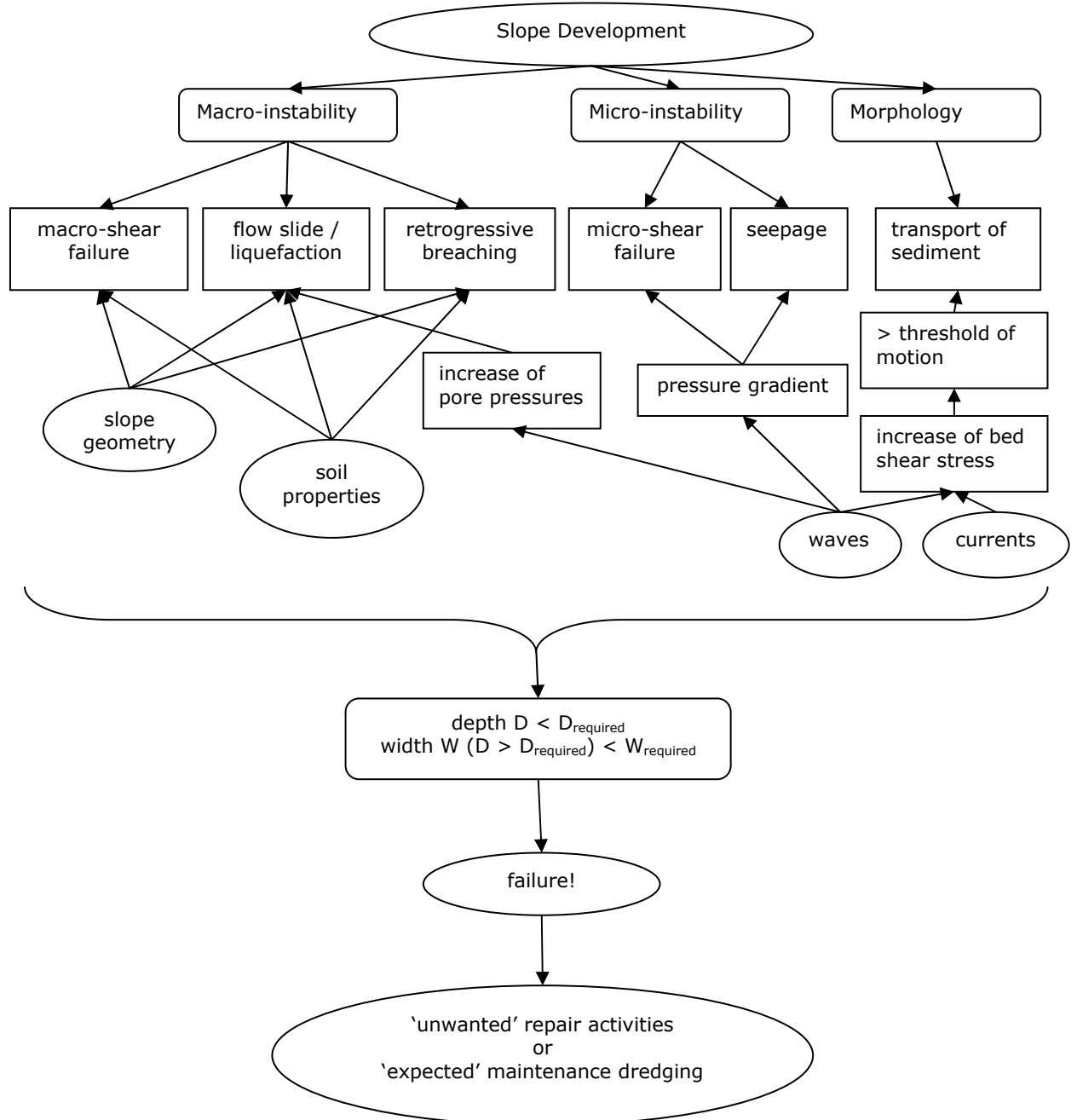


Figure 2-7: Instability mechanisms

2.3 Experiences from dredging practice

To gain a first insight in occurred slope developments, some examples from dredging practice will be treated. Of course a lot of parameters are involved, so guidelines cannot

be deduced from only a few experiences. Besides, understanding of the important processes is far more useful when designing a submarine slope in the future.

Nowadays, in engineering practice the design of a slope generally is based on:

- 1) observed side slopes along existing channels in similar areas;
- 2) results of geotechnical investigations in the area concerned;
- 3) observed slopes during trial dredging and maintenance dredging.

Despite all calculation efforts, the third method still seems to be most reliable. Furthermore, it is a useful tool to check the computations and gain knowledge for future projects. Dredging of trial trenches however is rather expensive and only occurs when the client has interest in reducing costs and/or risks.

The following examples of projects in the past give an idea of slopes, mechanisms and problems to be expected.

Trial slopes and slope production cuts 1980

In 1980 some trial slopes were dredged and a slope stability test was executed by Rijkswaterstaat, Svasek B.V. and Breejenbout Dredging Company to advise on economical underwater slopes [Boehmer et al., 1983]. Routine soil investigations and calculations resulted in uneconomical slopes of 1:10 to 1:15. Several tests have been done, like *box cuts*, *slope cuts* and *slope production cuts* to reduce the uncertainties and to demonstrate the feasibility of dredging steep slopes, despite the earthquake intensities and loosely packed sand layers. These slope stability tests should reveal soil properties which cannot be derived from field exploration or laboratory tests.

Because the first two of the above mentioned tests are also dredging methods that have been and still are widely used techniques to create submarine slopes, they will be explained briefly, see Figure 2-8.

Box cuts are dredged vertical slopes over a limited height which will result in slides causing a sediment gravity flow and slopes that will be flatter than slopes calculated from slip circle analysis (see Paragraph 4.1). Box cutting is the cheapest and easiest way of dredging submarine slopes, because of the high 'slope production'. *Slope cuts* are applied when there are spatial restrictions. Steep slopes for example in harbours, dry docks or in the vicinity of constructions are dredged by a trailing suction hopper dredger. Production rates will be lower and the resulting slopes often fail after the dredging has finished. This construction method is not considered to be very economic.

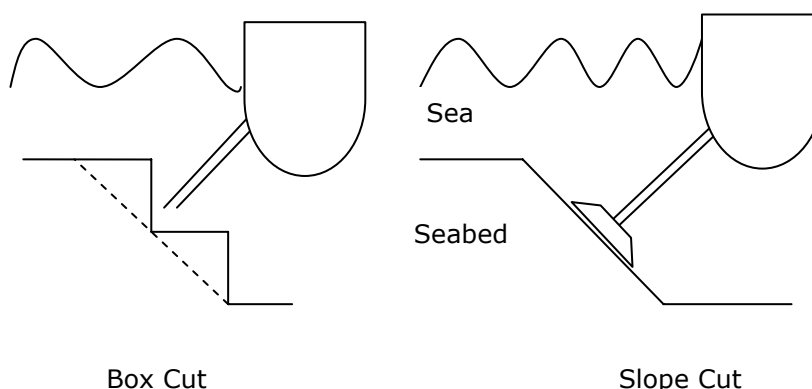


Figure 2-8: Schematization of methods of 'box cut' and 'slope cut'

The third testing method is a special slope stability test: in a *slope production cut*, a flow slide is simulated which will be representative for e.g. earthquake or sedimentation conditions. Slopes will flatten and the operational reliability of the harbour may be harmed.

When the *box cut* tests were done, it appeared that the resulting slopes were of the same order of the slopes calculated in slip circle analysis (1:1,6 to 1:3 in loosely packed sand (RD = 20-30%) and 1:2 to 1:4,5 in medium packed sand (RD = 50-60%)), although it was expected otherwise as was mentioned before. Therefore, the *slope cuts* have been omitted.

The *slope production cuts* were executed by supplying dredged spoil on top of the slope, resulting in a turbidity current and much flatter slopes: 1:5-6. A significant liquefaction flow slide wasn't observed. The flatter slopes can only be explained by the large erosion capacity of the sediment gravity flow.

The most relevant conclusions were:

1. slope stability tests give good insight in slope damages in case of calamities and the corresponding repair and maintenance costs;
2. safety factors against failures appeared to be higher than calculated;
3. the steepness of the designed slopes was increased to 1:5, while first calculations pointed at 1:10, because of the occurrence of earthquakes and the low relative density.

Pipeline trench at Danish North Sea Coast 1982

In 1982 a 1600 m long trench was dredged across the surf zone at the exposed North Sea Coast in Denmark to bring an oil and gas pipeline ashore [Mangor, 1986]. The backfilling and the wave climate were monitored over a period of 16 months. The trench was dredged up to 8 m deep in sand with a d_{50} of 200 μm . The theoretical design slope was 1:5, but after dredging, the slope was about 1:2, so the design width of 28,5 m was 50 m. Additional sediment trapping reservoirs were dredged to allow some backfilling, see Figure 2-9. The wave induced longshore current dominated over tidal and storm surge currents.

Table 2-4: Observed slopes of pipeline trench [source: Mangor, 1986]

	at bars		at troughs	
slope just after dredging	1:1-4		1:1-4	
after unidirectional storm	windward	leeward	windward	leeward
	1:3-5	1:20-50	1:10-20	1:20-50
after 'two-way' storm	~1:10		1:10-20	

Because three more or less parallel surf zone bars were present, the wave breaking and thus the sediment transport primarily occurred at these bars. This affected the morphological development of the trench. The windward (or upstream) slopes moved towards the centre. Especially in a unidirectional, rough wave climate rather steep slopes can develop, while leeward (downstream) slopes become very flat, because of the increased erosive capacities of the wave induced current. After a period with large waves from both sides, the developed side slopes are somewhere in the middle. This phenomenon will be discussed extensively in later chapters.

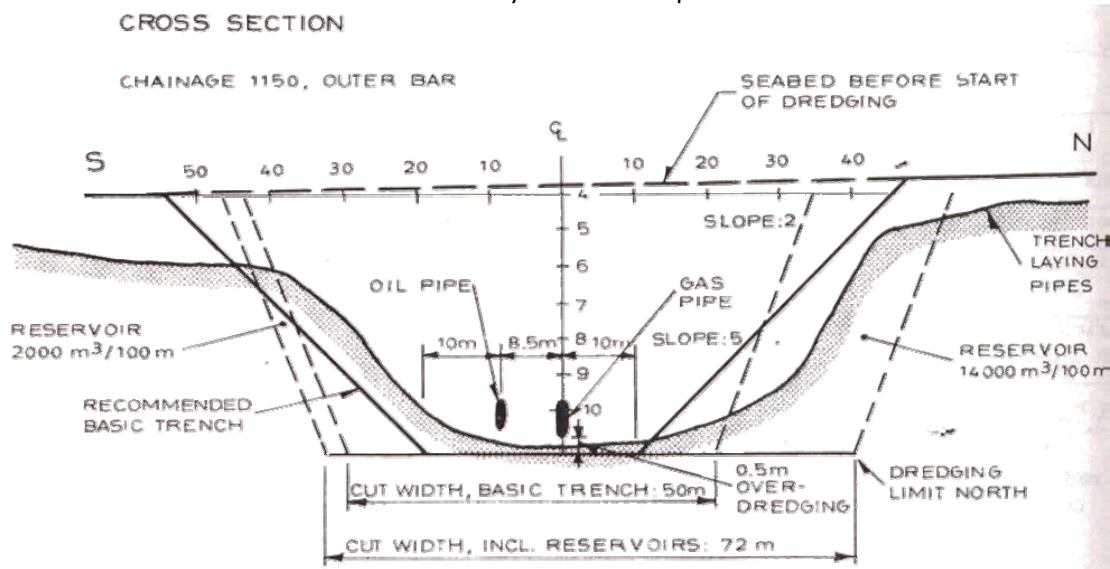


Figure 2-9: Cross-section of trench design [source: Mangor, 1986]

Puget Sound 1986

In February 1986 a large submarine flow slide occurred in Puget Sound near Seattle during construction works of a water transfer system of treated wastewater directly into Puget Sound, thereby restoring the water quality of the Duwamish River [Kraft et al., 1992]. Two outfalls were constructed to water depths of 34m. The soil consisted of Pleistocene silty clay, silt, sand and glacial till and a more recent upper layer of medium packed sand (d_{50} between 160 and 260 μm ; thickness of a few meters) with a relative density of 35%, which is classified as a medium dense sand. The maximum slope at water depths of 23-120 m was about 1:3,5, see Figure 2-10.

Based on these conditions there was no indication that a slide was likely. Further research after the slide showed that in this area no slides have occurred for the last 50 years. However, a major slide occurred, which was very hard to explain. A possible sequence of events causing a flow slide is described as follows:

“Failure of the dredged side slopes, face of the trench, or spoil piles adjacent to the trench could result in material flowing down the slope or trench gradient, perhaps with the assistance of the outgoing tidal current. This flow of suspended material has more erosion and disturbance potential than water alone and could have eroded steeper slopes in the deeper water to an oversteepened condition such that the near-surface sediments could collapse, inducing pore-water pressures. With the decrease in effective stresses resulting from the removal of only one meter or so of material in deeper water, the excess pore water pressures might then be sufficient to cause deeper sliding, in turn moving support to the upslope soils with the slide quickly regressing upslope or shoreward, typical of a retrogressive flow slide, until an equilibrium condition is reached.”

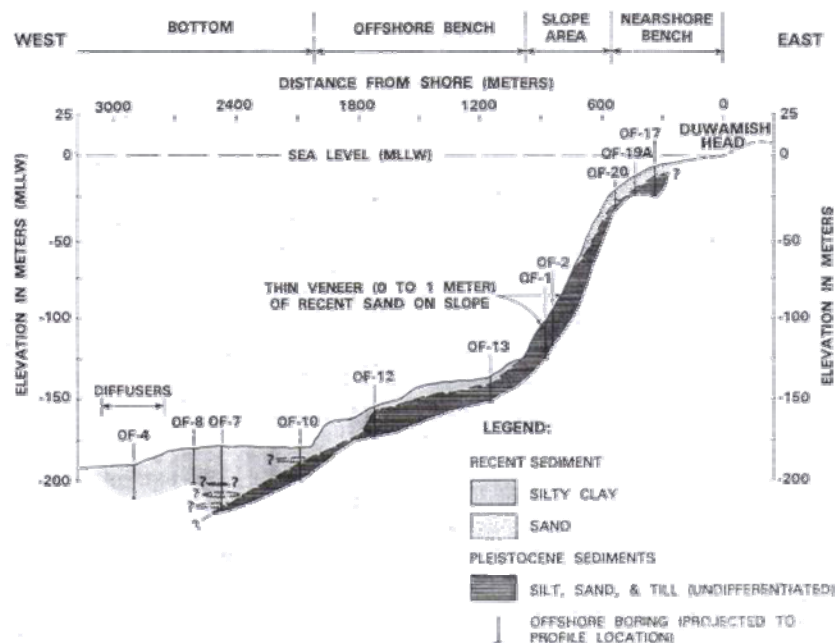


Figure 2-10: Cross-section of seabed at Puget Sound [source: Kraft et al., 1992]

Without explaining all of the above mentioned phenomena, which will be done later on, this sequence of events clearly shows the complex character of the predictability of flow slides. Often a number of unfavourable situations have to occur. From this case the following can be learnt:

- Start all dredging activities in shallow water and proceed sequentially toward deeper water, so work downslope rather than upslope, because any failure of the side slopes or face of the trench and the resulting gravity flow of failed

material will remain inside the trench itself instead of flowing down unobstructedly along the natural sloping bottom.

- Piles of spoil should not be sidecast in a loose condition next to the trench, but have to be removed from the project vicinity. Although the design often anticipates for cautious side casting, the procedures during construction are most of the times less strict.
- Reduce or stop dredging activities during periods with very low tidal elevations, because of the presence of excess pore water pressures in the less permeable, denser deeper soil layers, resulting in a reduction of stability.

One can also question the use of loosely packed sands (waste material) in backfilling of trenches, because of the high susceptibility to liquefaction (see Paragraph 4.2). Gravel is much more suitable backfilling material. As a compromise discrete zones of gravel can be placed within the trench, backfilled with loose dredge spoils.

Postma and his 'equilibrium slopes' 1987

Postma (1987) investigated several dredged pipeline trenches and found that the time that went by to reach an equilibrium profile was strongly dependent on the weather (wind waves) and that this equilibrium profiles showed wide variation, see Table 2-5.

Table 2-5: Equilibrium slope values of sandy bed material observed in dredged trenches for pipelines [source: Postma, 1987]

Location	Soil Data	Dredging Depth	Initial Slope	Equilibrium Slope
Ameland, NL	$D_{50} = 150 - 185 \mu\text{m}$	to 10m -NAP	1:3	1:13 or 1:14
Bacton, UK	$D_{50} = 570 \mu\text{m}$	to 10m -NAP	1:3	1:6 or 1:7
Great Yarmouth, UK	$D_{50} = 260 - 7500 \mu\text{m}$	to 30m -NAP	1:2	1:5 or 1:6
Hythe, UK	silty fine sand (becomes lumpy)	to 15m -NAP	1:3	1:4 or 1:5
Callantsoog, NL	$D_{50} = 300 - 350 \mu\text{m}$	to 15m -NAP	1:4	1:7 or 1:8

In this thesis, it will be shown that in fact there is no such thing as an 'equilibrium slope', because the only equilibrium is the original seabed. Besides, unless there is a completely symmetrical wave and (tidal) current pattern, one should distinguish between the upstream, which is predominantly located at the upstream side of the trench with respect to the current, and the downstream side slope.

It is however often useful to speak of an 'equilibrium upstream/downstream slope', because under certain conditions slopes tend to develop to a specific slope, which remains more or less constant during the sedimentation process, until this slope starts 'interacting' with the other slope or the trench bottom and will flatten.

Pusan 2002

Royal Boskalis Westminster NV has also dredged a lot of trenches and channels. Because of the huge spread in natural conditions, soil properties, dimensions and design purposes, only a few will be treated here.

In 2002 two trenches had to be dredged by trailing suction hopper dredger 'Barent Zanen' in very soft clay and silt (2,5 million m^3) as part of a harbour project in Pusan, South Korea. Two breakwaters had to be constructed, but because of the weak clayey and silty soil, a soil improvement had to be carried out. The life span of these trenches was not very long, because they were filled up with sand, which was borrowed from an area 105 km away, to form a solid foundation layer for the breakwater. The slope heights were 15 m at most and the surrounding water depth was also approximately 15 m. There was hardly any tidal movement and wave heights were very small. The slope development was mainly caused by soil mechanical aspects. This layer of clay and silt had a very low undrained shear strength of about 5 kPa and an undrained shear strength increase of 2,5 kPa/m.

Calculations were executed on this very poor input data. This resulted in slopes of 1:3 (14% probability of failure), 1:4 (8,8 % probability of failure) and 1:5 (6,4% probability of failure). It was decided to dredge slopes of 1:4, because it was decided that the probability of failure should not exceed 10% in case of very poor input data. The dredging work has been executed and the slopes remained stable.

Limiting slopes at ebb-tidal shoals

Buonaiuto and Kraus [2003] investigated a wide range of tidal inlets and ebb-tidal shoals on sandy coasts. They found that slopes of recently dredged entrance channels ranged from 1:7 to 1:10, with stabilized inlets sustaining greater slopes than unstabilized inlets. Wave-dominated environments sustain steeper slopes than tidally dominated regimes. Channels that are maintained by dredging show somewhat steeper slopes that typically decreased by 0,5-1 °/year.

Zuidwal 2004

In order to increase the accessibility of Gas Platform Zuidwal in the Wadden Sea an entrance channel had to be deepened to MSL -7 m, while the water depth varied from 2 to 7 m. The side slopes were designed at 1:6, although no significant wave action was present due to the favourable direction of the channel.

Dredging was performed in a two-layer operation. First the entire area was deepened to MSL -5 m, then the remaining 2 m was dredged. Slopes just after dredging varied from 1:2,5 to 1:5 with small almost vertical parts. After some time the slopes flattened to 1:2,7-13. This flattening process was attributed to manoeuvring with barges. This example clearly shows that especially during construction in shallow water the dredging activities may be more harmful to the side slopes than the hydrodynamic conditions.

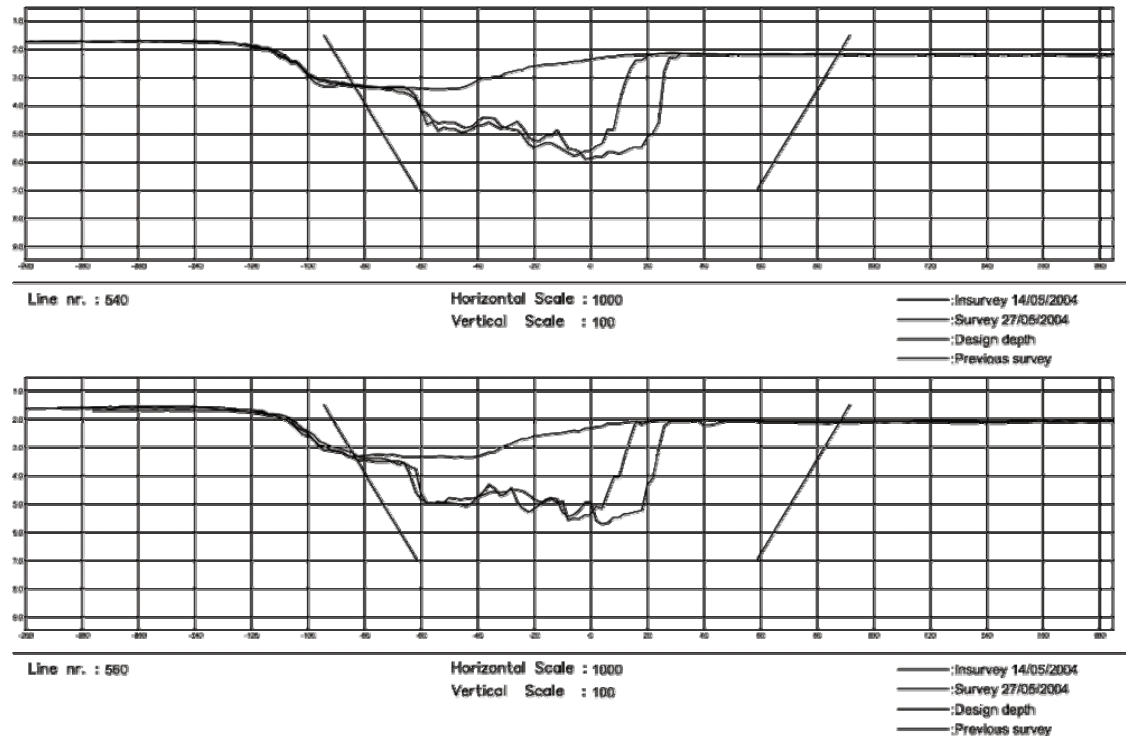


Figure 2-11: Two cross-sections of channel to Zuidwal Gas Platform; the steep right slopes have just been dredged

All these experiences give some insight in the variety of conditions in engineering practice and show that the knowledge of the underlying processes that cause slope failure is still insufficient. In the past, only a few efforts have been made to determine optimal side slopes under different conditions. But even if these processes are understood, dredging of a trial trench will always be recommendable, combined with extensive calculations, at least when there is enough budget.

3 Reference Cases

3.1 Definition of reference soils

3.1.1 Classification of soil types

There are different classifications of soil types; most use the grain size as governing parameter. In this thesis the soils under investigation are sand, silt and clay, which are classified by the Permanent International Association of Navigation Congresses (PIANC) in Brussel [1984], see Table 3-1, in which the 'white' rows are the soil types under investigation:

Table 3-1: Soil classification according to PIANC 1984; the high-lighted soils are treated in this thesis

Main soil type	Particle identification range of size	Grain size [μm]
Boulders		> 200.000
Cobbles		60.000 - 200.000
Gravels	Coarse	20.000 - 60.000
	Medium	6.000 - 20.000
	Fine	2.000 - 6.000
Sands	Coarse	600 - 2.000
	Medium	200 - 600
	Fine	63 - 200
Silts	Coarse	20 - 63
	Medium	6 - 20
	Fine	2 - 6
Clays		< 2
Peats and Organic soils		varies

As can be seen in the PIANC classification, grain size isn't a good criterion to distinguish between clays, peats and some silts; other characteristics are just as important. The main difference between clay and silt is the plasticity: 'Dry silt can easily be dusted off fingers and dry lumps powdered by finger pressure, whereas dry clay sticks to fingers and dry lumps do not powder, but shrink and crack.' [PIANC].

The International Association of Dredging Contractors established a system for identifying the parameters which determine the dredgeability. For the soil types at issue, these are the following:

Sand

- unit weight
- water content w
- specific gravity of grains
- grain size
- water permeability
- frictional properties
- lime content
- organic content

In this thesis all above parameters but the lime and organic content are under consideration.

Silt

- a. unit weight
- b. water content
- c. grain size
- d. water permeability
- e. sliding resistance
- f. plasticity
- g. lime content
- h. organic content

Silts are treated as a completely homogeneous material without any lime and organic contents or plasticity. In literature often confusing definitions can be found. Most of the time, silt as a soil classification is mixed up with 'silt' (Dutch: 'slib') as a deposited sediment in harbour basins. Then 'silt' often represents a random mixture of organic matter and inorganic sand and silt particles, sometimes called 'mud'. It should be clear that in this thesis silt is only treated as a type of soil, characterized by its grain size.

Clay

- a. weight
- b. water content
- c. sliding resistance
- d. consistency ranges (plasticity)
- e. organic content

Clays show a huge spread in all above parameters. The very hard and stiff clays aren't interesting when considering slope stability; under almost all circumstances vertical or nearly vertical walls will be stable. Therefore the focus will be on the very soft to firm clays.

3.1.2 Soil properties

Now it is clear what is understood by sand, silt and clay, some reference soils will be defined and sound assumptions of the various corresponding soil properties will be made. These reference soils will be used throughout this thesis to investigate slope stability problems, morphological behaviour and eventually to deduce guidelines for slope design. In Table 3-2 can be seen that the in total 15 reference soils are divided into grain size and porosity: 5 different grain sizes and 3 different porosities. For some instability mechanisms porosity is the leading parameter; in morphology the grain size of course is the most important soil property. Next some corresponding values for all other relevant soil properties were deduced from literature [Das, 1994; Terzaghi et al., 1996; Verruijt, 1999] and examples from engineering practice. Most of them will be explained here.

Density solid material

All reference sediments are assumed to originate from weathered rock of 2650 kg/m³, which means that the solid particle density is, of course, 2650 kg/m³. Also for clays, this value is a good first approximation.

Porosity

The porosity is the quotient of the pore volume V_p and the total volume of a soil sample V_t :

$$n = \frac{V_p}{V_t} \quad \text{Eq. 3-1}$$

Void ratio

The void ratio is also a measure for the porosity but with the (constant) volume of the grain particles V_g in the denominator.

$$e = \frac{V_p}{V_g} \quad \text{Eq. 3-2}$$

Relative Density

This quantity is related to the void ratio and is an indicator for behaviour under shear deformation (dilatant or contractant).

$$RD = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \quad \text{Eq. 3-3}$$

The maximum and minimum void ratios, $e_{\max}$ and $e_{\min}$, should be determined in laboratory experiments. In this thesis, these values have to be assumed. The theoretical values for $n_{\max}$ and $n_{\min}$ can be calculated by considering equally large spherical grains: the loosest packing is the 'cubic' one with a porosity $n_{\max}$ of $1-\pi/6 = 0,4764$ and the densest packing is the 'rhombic' one (connection lines of grains form an regular tetrahedron) with a porosity of $n_{\min} = 1-\pi/18 = 0,2595$. Using the following relation $e=n/(1-n)$ results in $e_{\max} = 0,9099$ and $e_{\min} = 0,3504$.

According to the Japanese Society of Civil Engineering (JSCE) the maximum void ratio $e_{\max}$ of fine to coarse sands is about 0,95 ($n_{\max} = 0,49$). When the sand contains silt, this void ratio is about 0,90 ($n_{\max} = 0,47$). In this thesis a maximum porosity of 0,48 will be used. The minimum porosity $n_{\min}$ is set to 0,30, so $e_{\min} = 0,43$.

Often the classification below is used to indicate loosely, medium and densely packed sands and silts. For cohesive soils, like clays, this approach does not hold, because of the complexity of the soil skeleton and the difficulties in determining the maximum and minimum porosity.

$RD < 0,33$: loosely packed sand / silt
 $0,33 < RD < 0,67$: medium packed sand / silt
 $0,67 < RD$: densely packed sand / silt

Table 3-2: Reference soils and their soil properties (see Appendix C for complete table)

Name	Description	density solid material porosity void ratio relative density saturated density saturated unit weight dry unit weight unsaturated unit weight friction angle cohesion undrained shear strength median particle diameter of total weight diameter at which 90% by weight is finer hydraulic conductivity intrinsic permeability consolidation coefficient particle fall velocity at T=18°C																	
		ρ_s	n	e	RD	ρ_{sat}	γ_{sat}	ρ_{dry}	γ_{dry}	γ_{unsat}	ϕ	c	s_u	d_{50}	d_{90}	k	κ	C_v	w_s
		kg/m ³	-	-	-	kg/m ³	kN/m ³	kg/m ³	kN/m ³	kN/m ³	°	kPa	kPa	μm	μm	m/s	m ²		m/s
LCS	loosely packed coarse sand	2650	0,45	0,82	0,16	1908	19	1458	15	17	30	0	0	1000	1500	1,9E-04	1,0E-08	3,8E-01	0,1084
MCS	medium packed coarse sand	2650	0,40	0,67	0,43	1990	20	1590	16	18	35	0	0	1000	1500	3,5E-05	1,8E-09	1,2E-01	0,1084
DCS	densely packed coarse sand	2650	0,30	0,43	0,86	2155	22	1855	19	20	40	0	0	1000	1500	1,2E-05	5,5E-10	5,9E-02	0,1084
LMS	loosely packed medium sand	2650	0,45	0,82	0,16	1908	19	1458	15	17	30	0	0	500	750	1,4E-05	7,5E-10	2,9E-02	0,0663
MMS	medium packed medium sand	2650	0,40	0,67	0,43	1990	20	1590	16	18	35	0	0	500	750	8,8E-06	4,4E-10	3,1E-02	0,0663
DMS	densely packed medium sand	2650	0,30	0,43	0,86	2155	22	1855	19	20	40	0	0	500	750	3,0E-06	1,4E-10	1,5E-02	0,0663
LFS	loosely packed fine sand	2650	0,45	0,82	0,16	1908	19	1458	15	17	30	0	0	200	300	2,3E-06	1,2E-10	4,6E-03	0,0252
MFS	medium packed fine sand	2650	0,40	0,67	0,43	1990	20	1590	16	18	35	0	0	200	300	1,4E-06	7,1E-11	5,0E-03	0,0252
DFS	densely packed fine sand	2650	0,30	0,43	0,86	2155	22	1855	19	20	40	0	0	200	300	4,7E-07	2,2E-11	2,4E-03	0,0252
LSI	loosely packed silt	2650	0,45	0,82	0,16	1908	19	1458	15	17	27	0	0	50	75	1,4E-07	7,5E-12	2,9E-04	0,0029
MSI	medium packed silt	2650	0,40	0,67	0,43	1990	20	1590	16	18	32	0	0	50	75	8,8E-08	4,4E-12	3,1E-04	0,0029
DSI	densely packed silt	2650	0,30	0,43	0,86	2155	22	1855	19	20	37	0	0	50	75	3,0E-08	1,4E-12	1,5E-04	0,0029
SOC	soft (organic) clay, low plasticity	2650	0,75	3,00		1413	14	663	7	11	18	5	5	2	3	3,8E-09	2,7E-13	7,6E-07	
MEC	medium clay, medium plasticity	2650	0,60	1,50		1660	17	1060	11	14	18	7	20	2	3	9,0E-10	5,4E-14	4,5E-07	
STC	stiff clay, high plasticity	2650	0,40	0,67		1990	20	1590	16	18	20	10	50	2	3	1,4E-10	7,1E-15	1,4E-07	

Hydraulic permeability

This property is dependent on both soil and fluid characteristics according to:

$$k = \frac{\kappa \gamma_w}{\mu} \quad \text{in which} \quad \kappa = c_{K-C} d^2 \frac{n^3}{(1-n)^2} \quad (\text{formula of Kozeny-Carman}) \quad \text{Eq. 3-4}$$

The coefficient c_{K-C} is taken 0,01, although it will show variations in reality. Normally the hydraulic permeability will be determined in tests, which gives more accurate results than the above formulas, but this approach yields values of the right order of magnitude and clearly shows the influence of grain size and porosity.

Consolidation coefficient

The approximation of the consolidation coefficient of all reference soils is based on the assumption that hydraulic permeability 'k' and the coefficient of compressibility 'm_v' remain constant during the consolidation process and that the pore water is incompressible ($\beta = 0$). The following formula can be deduced from the 1-dimensional, vertical differential equation of consolidation.

$$c_v = \frac{k}{\gamma_w (m_v + n\beta)} \quad \text{Eq. 3-5}$$

The coefficient of compressibility m_v is estimated according to the linear elastic stress-strain-relation:

$$m_v = \frac{1}{K + \frac{4}{3}G} \quad \text{Eq. 3-6}$$

in which the bulk modulus K and the shear modulus G can be expressed in terms of Young's modulus E and Poisson's ratio ν :

$$K = \frac{E}{3(1-2\nu)} \quad \text{and} \quad G = \frac{E}{2(1+\nu)} \quad \text{Eq. 3-7}$$

The values of E and ν are taken from literature. Again this approximation is a rather strong simplification of reality, but yields reliable input parameters.

Particle fall velocity

To gain insight in the spread on the input value for the particle fall velocity three formulae have been considered for the reference soils. Van Rijn [1993] distinguished three ranges of particle diameters: $d_{50} \leq 100 \mu\text{m}$, $100 \leq d_{50} \leq 1000$ and $d_{50} \geq 1000 \mu\text{m}$. For diameters smaller than $100 \mu\text{m}$ he used the well-known Stokes equation:

$$w_s = \frac{(s-1)gd^2}{18\nu} \quad \text{Eq. 3-8}$$

and for diameters between 100 and 1000 μm :

$$w_s = \frac{10\nu}{d} \left[\left(1 + \frac{0,01(s-1)gd^3}{\nu^2} \right)^{0,5} - 1 \right] \quad \text{Eq. 3-9}$$

in which s is the specific density ($s = \rho_s / \rho_w$) and the kinematic viscosity ν [m^2/s] is dependent on the water temperature T [$^{\circ}\text{C}$]:

$$\nu = (1,14 - 0,031(T - 15) + 0,00068(T - 15)^2) \cdot 10^{-6} \quad \text{Eq. 3-10}$$

This results in a kinematic viscosity of $1,31 \cdot 10^{-6} \text{ m}^2/\text{s}$ in water of 10°C and $1,05 \cdot 10^{-6} \text{ m}^2/\text{s}$ in water of 18°C .

Another well-known formula is developed by Delft Hydraulics in 1983 during experiments in water of 10°C and 18°C . These experiments were only executed on diameters varying from 50 to $350 \mu\text{m}$.

$$\log(1/w_s) = A \log^2 d + B \log d + C \quad \text{Eq. 3-11}$$

Table 3-3: Values of constants in equation 3-11 [source: Sistermans, 2002]

	$T=10^\circ\text{C}$	$T=18^\circ\text{C}$
A	0,476	0,495
B	2,18	2,41
C	3,19	3,74

The mentioned coefficients are for fresh water only. In Figure 3-1 the results of both formula have been plotted for two temperatures; the formula of DH'83 in fact has been extrapolated outside its range of 50 to $350 \mu\text{m}$. The variations are quite small, especially for the smaller particle diameters. Although the salinity of the water, the shape or angularity of the particles and the sediment concentration have been disregarded in this approach, they also have a small influence the particle fall velocity.

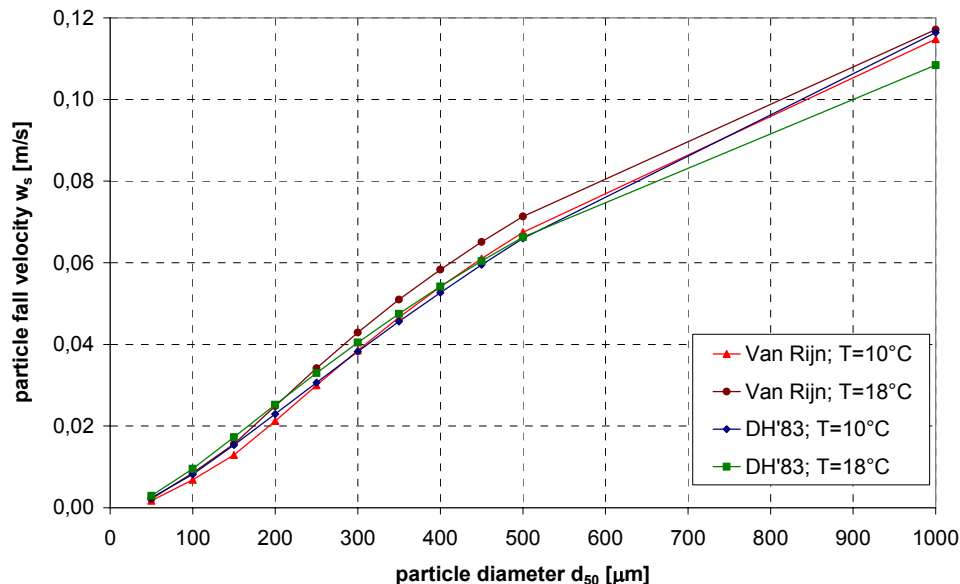


Figure 3-1: Particle fall velocities in fresh water of 10°C and 18°C according to Van Rijn [1993] and Delft Hydraulics [1983]

Grading

All reference soils are defined as homogeneous, which implies a uniform particle size. In reality no such soils can be found. Therefore a narrow grading is assumed:

$$\frac{d_{90}}{d_{50}} = 1,5 \quad \text{Eq. 3-12}$$

With the definition of the reference soils, calculations can be done. It is however not very useful to consider all fifteen soils in all cases. Macro stability calculations for instance aren't really sensitive to grain size, while transport calculations are extremely sensitive to this quantity. On the other hand, morphology isn't very much affected by porosity; only the bed changes will proceed faster. Susceptibility to liquefaction is extremely dependent on porosity as long as sand is under consideration; clays will not be susceptible to liquefaction (quick clay which shows some similarities to flow slides isn't considered in this thesis). Therefore a selection of the appropriate reference soils will be made at each subject. Sometimes the sensitivity to a single soil property is investigated, when this particular property is of utmost importance.

3.2 Definition of reference geometries

Of course trenches and channels have different dimensions and it would be impossible to derive guidelines for a dimensionless trench. Therefore a typical trench and a typical navigation channel are defined, see Figure 3-3 for used symbols.

Trench

A trench often is only a few meters deep and not more than 10 meters wide. If trenches are dredged instead of pipeline burial after the laying process, this often occurs in shallow water, for instance outfalls. Because the trench should be located in the non-breaking area, the depth will be 5 m, assuming that no waves larger than 2,5 m will pass over the trench (either they do not occur or they are already broken). The depth of the trench will be 5 m and the width is assumed to be 10 m, see Figure 3-2.

Navigation channel

Well-known examples of navigation channels are the 'IJ-channel' (Port of IJmuiden) and the 54-km long 'Euro-Maas'-channel (Port of Rotterdam). The depth of the former is kept at 19m, navigable for ships with a draught of 16,5m; the depth of the latter is 24 m for ships with a draught of 22,5 m. The surrounding seabed is of course sloping downward into the sea, but typical heights of side slopes of 10 m are not exceptional. The width of the 'Maasgeul' (last 14 km) is rather larger, viz 400-600 m; in this thesis the width is set to 200 m. Larger widths result in larger computational time and less accuracy around the slopes; smaller widths prevent autonomous slope development (slopes start interacting with each other).

Compared to a pipeline trench, not only the dimensions of the slopes are quite different, also the morphological behaviour will differ a lot, because of the larger trapping efficiency. In order to keep the amount of calculations/simulations within reasonable limits and because the morphological development becomes more complex for very small and sharp trenches, the morphological behaviour is only studied extensively for the navigation channel. Most geotechnical calculations have been executed for slope heights of 5, 10 and 15 or 20 m.

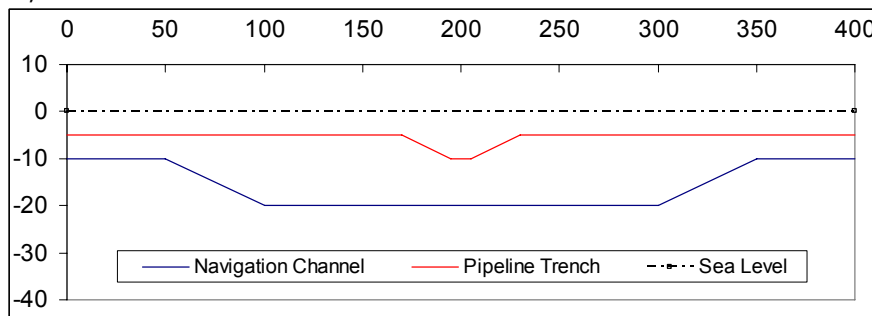


Figure 3-2: Geometries of 'navigation channel' and 'pipeline trench' with side slopes 1:5

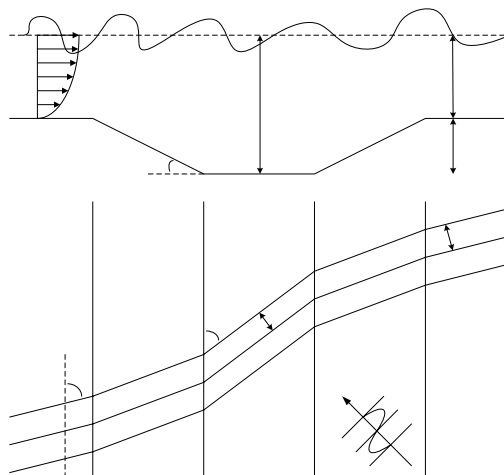


Figure 3-3: Schematization of some important parameters

4 Unloaded macro- & micro-instability

In this paragraph the focus will be on macro-instability. Unlike micro-instability, macro-instability should be prevented in any case, because large movements of soil will certainly endanger the functionality of the trench or channel. The resulting slope after a failure will be flattened to such a degree, that the required depth cannot be guaranteed.

4.1 Shear failure

4.1.1 Considerations on shear failure

The most common instability mechanism for slopes is shear failure. Before calculating slope stability factors, a number of aspects have to be considered to be able to make sound assumptions on the input values.

- Drained and undrained conditions

One should consider the soil behaviour for each design condition. Generally this behaviour is determined by the combination of the permeability of the soil and the speed of the (dynamic) load or construction. In other words, can water pressures dissipate during loading or construction? The permeability of sands is generally a factor 10^5 higher than that of clays.

As a rough guideline for design during and just after *construction* [Slope Stability Engineering Manual, 2003], soils with values of permeability greater than 10^{-6} m/s usually will be fully drained and soils with values of permeability less than 10^{-9} m/s will be primarily undrained. This means that in static design calculations all reference 'sands' (see Paragraph 3.1.2) will show drained behaviour, while all 'clays' show undrained behaviour. The permeability of the reference 'silts' lies in between these two values. In this thesis it is assumed that 'silt slopes' of dredged trenches behave drained, based on the following line of thought. In the first place, the sea bottom consists of totally consolidated soil layers because of its age, in contrary to, for example, surcharge loads, which will cause consolidation of the underlying soil and therefore extra water pressures. In the second place, when dredging a trench or channel and extracting a significant mass of soil, the seabed will be slightly overconsolidated and, as a consequence, the seabed will relax, thus increasing pore volume and reducing water pressures. Therefore a drained calculation seems appropriate for silts.

In the *long-term* condition, all soils will be fully drained by definition, regardless their permeability. For clays, also this condition has to be considered: the less stable condition (construction or long-term) should be normative for design.

Drained analyses use drained strengths related to *effective* stresses, which can be obtained from consolidated-drained tests (CD) or consolidated-undrained triaxial tests on saturated specimens with pore water pressure measurements (CU); undrained analyses use undrained strengths related to *total* stresses, which can be obtained from unconsolidated-undrained tests (UU).

The linear strength envelope according to the Mohr-Coulomb failure criterion becomes:

for sands and silts: $\tau = c' + \sigma' \tan \phi$, in which $c' = 0$ for cohesionless soils;

for clays: $\tau = c = s_u$, in which s_u is the undrained shear strength.

Although not considered in this thesis about purely theoretical, homogeneous and isotropic cases, one should in real situations be aware of:

- Ductile and brittle stress-strain behaviour

For soils with ductile stress-strain behaviour (shear resistance does not decrease significantly as strain increases beyond its peak value), like soft clays, loose sands and clays compacted at water contents higher than optimum, the peak shear strength may be used in slope stability calculations. For soils with brittle stress-strain behaviour (shear resistance decreases significantly as strain increases beyond its peak value), like stiff clays, dense sands and clays compacted at optimum water content or below, a shear resistance lower than the peak shear strength should be used, because of the possibility of progressive failure. This kind of failure occurs under conditions where shearing resistance first increases and then decreases with increasing strain. As a result the slope isn't able to mobilize its peak shear strength at all points along the slip surface.

- Strength anisotropy

The shear strength of soils may vary with orientation of the failure plane, particularly in presence of fissures.

4.1.2 Unloaded shear failure

The unloaded (static) shear failure will be investigated using a slip-circle analysis (Bishop-method). Although shear movements may occur across a zone of appreciable thickness, shear failure is considered to occur along a discrete surface, a so-called slip-circle. The idea is based on stability expressed as a safety-coefficient F , which is strength divided by load. The load is determined by the weight of the soil of a certain slip circle; the strength by the shear along that same circle. Therefore a slip circle is divided into a lot of slices and for each of the slices the weight and the shear is calculated. The stability coefficient F will be computed for a lot of circles and the lowest value indicates the safety against failure: $F_{min} > 1$ means that the slope profile is theoretically stable, but often higher values for the stability coefficient are applied. In this thesis, in case of macro-stability, a safety factor of 1,5 will be considered unconditionally stable: the uncertainty in dredging works on one hand and the often limited knowledge of the soil properties on the other hand require such a conservative stability criterion. In case of micro-stability, a safety factor of 1,25 will be considered sufficient safe, because of the smaller consequences, when failure occurs.

Please note that lower values of safety factors for slopes in existing channels are acceptable, because one has been able to observe the actual performance of the slope over a period of time. It is also recommended to apply lower safety factors for temporary pipeline trenches.

Both Fellenius and Bishop follow this calculation method mentioned above, although Bishop takes the reduction factor (F) and the fact that the slice surface makes an angle (β) with the gravity force into account. Because this reduction factor isn't known beforehand, iteration is necessary. Unlike Fellenius, Bishop's method preserves equilibrium of vertical forces:

$$F = \frac{R \sum_{i=1}^{n_{layers}} \frac{b \tau_{slip}}{\cos \beta}}{M_{soil} + M_{water} + M_{load}} \quad \text{Eq. 4-1}$$

R is the radius of the slip circle and τ is the shear strength along the slice bottom that depends on the angle of the slice bottom and can be defined as:

$$\tau_{slip;\alpha} = \frac{c + \sigma'_v \tan \phi}{F + \tan \beta \tan \phi} \quad \text{Eq. 4-2}$$

in which underscore ' α ' is used to express the angle of the bottom of a slice.

The three moments in the denominator are the driving moments. M_{soil} is the driving moment caused by the mass of soil within the slip circle around its centre. This moment increases if the unit weight of the soil increases. M_{water} is the driving moment caused by the water forces acting from the outside on the surface of the soil structure. External loads can cause an additional driving moment M_{load} .

A lot of calculations have been made with the computer-program MStab by GeoDelft. The two reference cases 'trench' and 'navigation channel' as well as some other slope heights D have been investigated for different slope angles and soil characteristics. Calculations under drained conditions have been done for all reference soils; calculations under undrained conditions only for cohesive soils as was discussed in Paragraph 4.1.1.

Drained conditions

One of the first things to keep in mind is that the water level does not play a role, as long as the (static, flat) water level is situated completely above the slope; the safety factors remain the same, when considering static stability. Later on in this paragraph, the influence of a static water level under the top of the slope will be discussed.

In Table 4-1 can be seen that for non-cohesive soils, like sand and silt, the angle of internal friction is the main parameter. In fact, when considering static macro-stability, it is the only important variable. In a completely homogeneous soil, the safety factor isn't affected by the unit weight: the driving moment of the soil increases, when the unit weight increases, but so is the resisting moment, which is dependent on the weight of a slice. Heavier soils however are often more densely packed and are characterized by a larger friction angle. This is the reason why a medium packed fine sand (MFS) is more stable than a loosely packed fine sand (LFS). And the latter on his turn is more stable than the equally loosely packed silt. In non-cohesive, homogeneous soils a sound determination of this angle will be fruitful.

Table 4-1: Safety factors for different slope angles and reference soils; gray cells represent safety factors larger than 1,5

		γ_{sat}	c	ϕ	$\tan \phi$	Trench				Navigation channel				
		kN/m ³	kPa	°	1:x	1:1,5	1:2	1:2,5	1:3	1:1,5	1:2	1:2,5	1:3	
LFS	loosely packed fine sand	19	0	30,0	1,73	0,87	1,16	1,44	1,73	0,87	1,16	1,44	1,73	micro-stability
MFS	medium packed fine sand	20	0	35,0	1,43	1,05	1,40	1,75	2,10	1,05	1,40	1,75	2,10	
DFS	densely packed fine sand	22	0	40,0	1,19	1,26	1,68	2,10	2,52	1,26	1,68	2,10	2,52	
LSI	loosely packed silt	19	0	27,0	1,96	0,77	1,02	1,27	1,53	0,77	1,02	1,27	1,53	
MSI	medium packed silt	20	0	32,0	1,60	0,94	1,25	1,56	1,88	0,94	1,25	1,56	1,88	slip c.
DSI	densely packed silt	22	0	37,0	1,33	1,13	1,51	1,89	2,26	1,13	1,51	1,89	2,26	
SOC	soft clay	14	5	17,5		2,42	2,70	2,95	3,19	1,54	1,82	2,04	2,26	
MEC	medium clay	17	7	18,0		2,15	2,43	2,66	2,90	1,45	1,68	1,90	2,11	
STC	stiff clay	20	10	20,0		2,26	2,55	2,82	3,08	1,54	1,79	2,03	2,26	

What also can be concluded from Table 4-1 is the fact that slopes steeper than the corresponding 'natural slope' are of course not stable, but this instability mechanism can better be characterized by micro-instability (see Paragraph 4.4), because failure occurs at the slope surface with relatively small amounts of particles (in reality, grain particles would 'rain down' along the slope, until the angle of friction is reached). The so-called slip circles only slightly touch the slope surface. But it appears that in non-cohesive soils even at much flatter slopes, the normative failure mechanism is micro-instability, in contrary to cohesive soils.

The clays (SOC, MEC and STC) always collapse with nice slip circles in which a huge amount of material is involved. Also can be concluded that cohesive soils are more stable; even a steep slope of 1:1,5 can still be considered 'safe'. Such thing as a 'natural slope' is hard to define for cohesive soils, because it also depends on the slope height.

All values are plotted in Figure 4-1 for the "navigation channel". It can be seen that safety factors increase when slopes are flattened, but this increase is more significant with non-cohesive soils. Cohesive soils are not only dependent on the friction angle, but also on cohesion and unit weight.

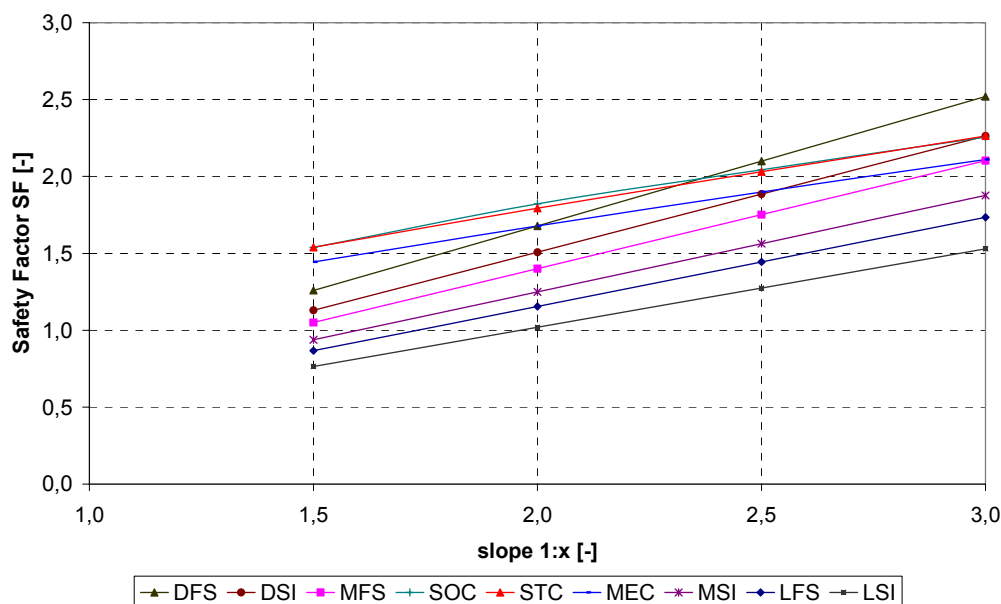


Figure 4-1: Safety Factors against critical slopes; navigation channel; all slip circles permitted

For slopes less steep than the 'natural slope' in non-cohesive soils it is also interesting to know what the safety against macro-failure is, not only because macro-failure has larger consequences, but also to compare static stability with dynamic stability. Therefore a second series of calculations was done in which the tangent line was restricted to depths of at least 0,5 m under the channel bottom and the centre point of the slip circle was located above the highest point of the undisturbed surrounding bed and between the toe and top of the sloping bottom, see Figure 4-2, in which the red dots are all situated within the permitted area of centre points and the green lines are all permitted tangent lines. The slip circle characterized by the lowest safety factor is drawn.

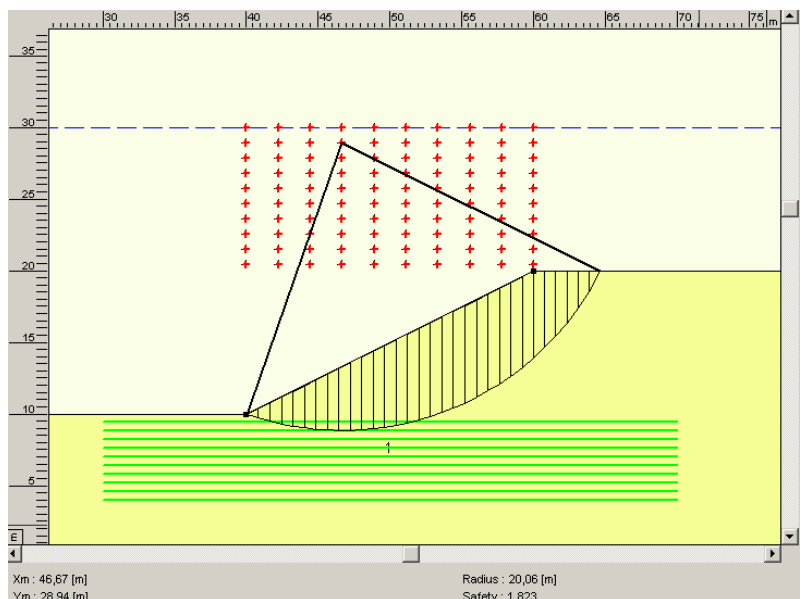


Figure 4-2: Permitted tangent lines and centre points to create 'nice' slip circles; example slip circle in soft clay

Constraining the permitted slip circles in this way resulted in larger slip circles with of course slightly higher safety factors, see Table 4-2. It can be concluded that only steep

slopes (>1:2) of loosely packed sediment run some risk of collapse due to macro-instability, but most of the times, when no external loads occur, when the soil is completely consolidated and homogeneous, macro-stability will be guaranteed. Again for the "navigation channel" the safety factors have been plotted, see Figure 4-3.

In the limiting case of infinite slopes, safety-factors of macro-stability and micro-stability will coincide. Slope collapse due to macro-instability is most likely in the presence of weaker soil layers or fluctuating water levels. That's why in literature slope failures of this kind have been reported, although this thesis seems to demonstrate that submarine slope failures due to macro-instability are not very likely; it should be clear that this is solely caused by the assumption of homogeneous soils that are completely situated below sea level.

Table 4-2: Safety factors for different slope angles and reference soils with restricted slip circles; grey cells represent safety factors larger than 1,5

		γ_{sat}	c	ϕ	$\tan \phi$	Trench				Navigation channel				
		kN/m ³	kPa	°	1:x	1:1,5	1:2	1:2,5	1:3	1:1,5	1:2	1:2,5	1:3	
LFS	loosely packed fine sand	19	0	30,0	1,73	1,01	1,47	1,72	1,97	1,12	1,37	1,63	1,90	slip circles
MFS	medium packed fine sand	20	0	35,0	1,43	1,51	1,80	2,08	2,39	1,36	1,67	1,97	2,31	
DFS	densely packed fine sand	22	0	40,0	1,19	1,81	2,14	2,49	2,86	1,63	2,00	2,37	2,77	
LSI	loosely packed silt	19	0	27,0	1,96	1,10	1,30	1,51	1,74	0,99	1,21	1,44	1,68	
MSI	medium packed silt	20	0	32,0	1,60	1,35	1,59	1,86	2,13	1,21	1,49	1,76	2,06	
DSI	densely packed silt	22	0	37,0	1,33	1,63	1,92	2,24	2,57	1,46	1,79	2,12	2,48	
SOC	soft clay	14	5	17,5		2,42	2,70	2,95	3,19	1,54	1,82	2,04	2,26	
MEC	medium clay	17	7	18,0		2,15	2,43	2,66	2,90	1,45	1,68	1,90	2,11	
STC	stiff clay	20	10	20,0		2,26	2,55	2,82	3,08	1,54	1,79	2,03	2,26	

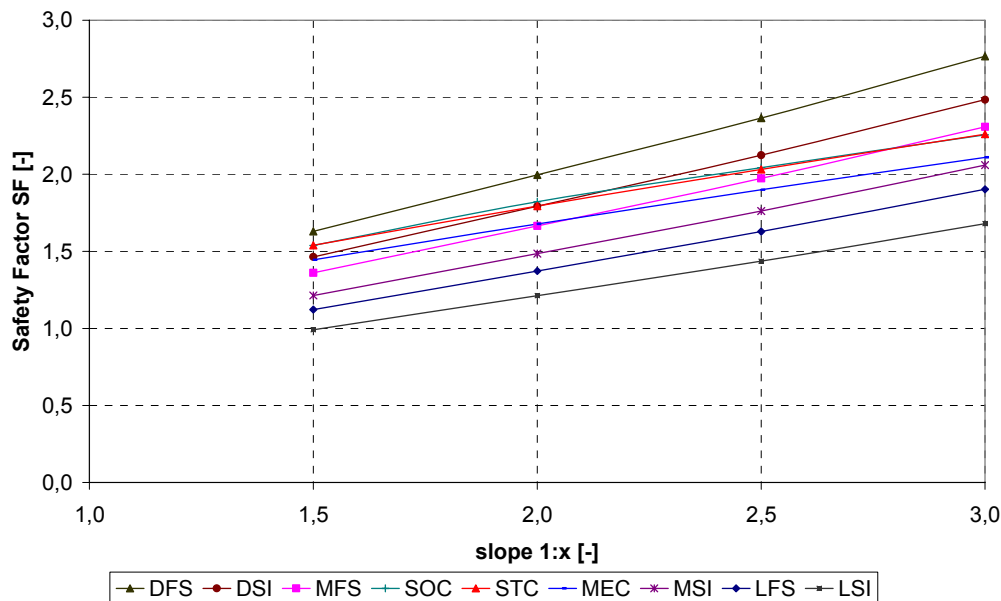


Figure 4-3: Safety Factors against critical slopes with restricted slip circles (only macro-stability!); navigation channel

When slopes emerge from the sea level, the situation gets less favourable, because of loss of buoyancy as was mentioned earlier. In a marine environment, a large tidal amplitude can cause this emergence. This phenomenon plays a role in regions like the Wadden Sea.

To investigate this problem of fluctuating water levels, a rough approximation was done, assuming flat water levels and fully consolidated soils. In the initial situation the soil is saturated and consolidated. When the water level drops, the pore water pressure slowly will dissipate resulting in consolidation until a situation has developed with a layer of (consolidated) unsaturated (note: not completely dry) soil on top of a layer of saturated soil. In case of a slow fluctuation of the water level and relatively permeable soils (like most sands), pore water overpressures will not develop.

The stability is only worsened due to loss of buoyancy. Calculations have been done in MStab for medium packed fine sand (MFS) and medium packed silt (MSI). The unit weight of the soil above the freatic level was varied between the saturated and the unsaturated value. Again distinction was made between macro-stability (with restricted slip circles) and micro-stability.

It was found, that a water level of 0,4 times the slope height D (so 40% of the slope is situated below sea level) was most unfavourable, when considering macro-stability. The Safety Factors for macro-stability were reduced over 15% in case of the saturated soil and around 13% in case of unsaturated soil with respect to the initial situation. The saturated soil of course reduces stability more because of the larger driving moment ($\gamma_{\text{sat}} > \gamma_{\text{unsat}}$), while the counteracting moment remains the same.

Furthermore it appeared that micro-stability isn't normative with water levels somewhere along the slope, to be more precisely: $0 D \leq \text{water level} \leq 0,75 D$. For these levels perfect slip circles with more or less equal safety factors were found. If the water level is well under the channel bottom or above the top of the slope, micro-stability becomes normative again. This phenomenon is clearly demonstrated in Figure 4-4 for slopes of 1:3 in MFS and MSI. For water levels between 0 and 0,75D, there is a clear dip in the safety factor, caused by failure due to macro-instability. Around this unfavourable zone, the safety factors are governed by micro-instability (see Figure 4-1).

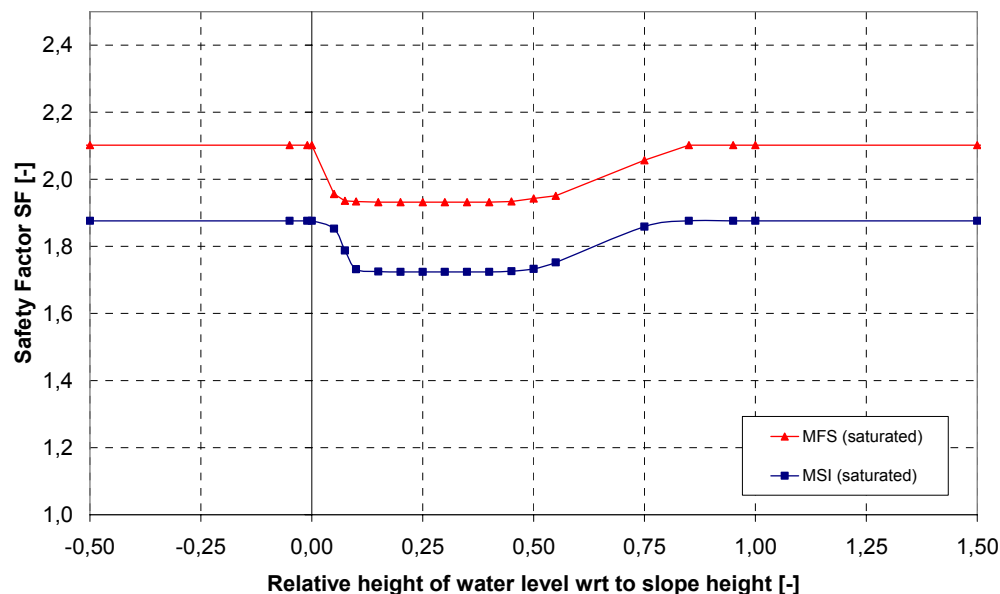


Figure 4-4: Safety Factors in case of fluctuating water levels in fully saturated MFS and MSI; navigation channel; 1:3 side slopes

From the above observations it can be concluded that macro-stability is normative in non-cohesive soils if slip circles can be attracted to weaker points in the slope, in other words if the soil mass isn't a completely homogeneous, regular slope. This means that macro-stability will be normative if weak layers or unfavourable water levels are present.

Another striking result is the apparently inconsistent fact that under drained conditions soft clay is more stable than medium clay and that soft and stiff clay seems to be more or less equally stable in these drained ϕ, c -calculations. To explore this problem, the effect of variations of the governing variables is investigated.

In the drained ϕ, c -calculations, the three soil parameters γ_{sat} , ϕ and c have been changed (see Table 4-3), starting from the properties of stiff clay: $\gamma_{\text{sat}} = 20 \text{ kN/m}^3$; $\phi = 20^\circ$ and $c = 10 \text{ kPa}$. The just stable slope angle ($\beta_{\text{SF}=1}$) and the matching slope are calculated.

Table 4-3: Variations in parameters ϕ , c and γ_{sat}

ϕ °	$\beta_{\text{SF}=1}$ °	1:x
0	13,4	4,20
5	25,5	2,10
10	39,8	1,20
15	53,1	0,75
20	60,3	0,57
25	67,2	0,42
30	73,3	0,30

c kPa	$\beta_{\text{SF}=1}$ °	1:x
0	20,0	2,75
2	28,4	1,85
4	36,5	1,35
6	45,0	1,00
8	52,4	0,77
10	60,3	0,57

γ_{sat} kN/m ³	$\beta_{\text{SF}=1}$ °	1:x
14	87,9	0,04
16	78,7	0,20
18	69,2	0,38
20	60,3	0,57
22	53,5	0,74

All values for the three variables are known from literature, but a certain combination of ϕ , γ_{sat} and c can be very uncommon. In Figure 4-5 these variables are plotted, note the different y-axis. On the left y-axis the friction angle ϕ and the saturated unit weight γ_{sat} are indicated, successively the blue and brown line; on the right y-axis (on a different scale) the cohesion c is indicated, the green line. The point where the three lines intersect represents the 'normal' stiff clay (STC), which is the reference soil.

It can be concluded that:

- a decrease of cohesion means a decrease of stability. A non-cohesive soil ($c = 0$) is completely governed by the angle of internal friction: $1/\tan \phi = 2,75$;
- a decrease of the angle of friction means a decrease of stability. A purely cohesive soil ($\phi = 0$) is completely governed by cohesion: the critical slope depends on the slope geometry. In this case a slope will be stable when flatter than 1:4,2;
- an increase of (saturated) unit weight means a decrease of stability. Heavier soil results in a larger driving moment, which is only partly compensated by the larger friction at the surface of the slip circle.

Because of these conclusions, it is possible that clay with a smaller cohesion, friction angle and unit weight can be more stable. It just depends on the exact values of the parameters.

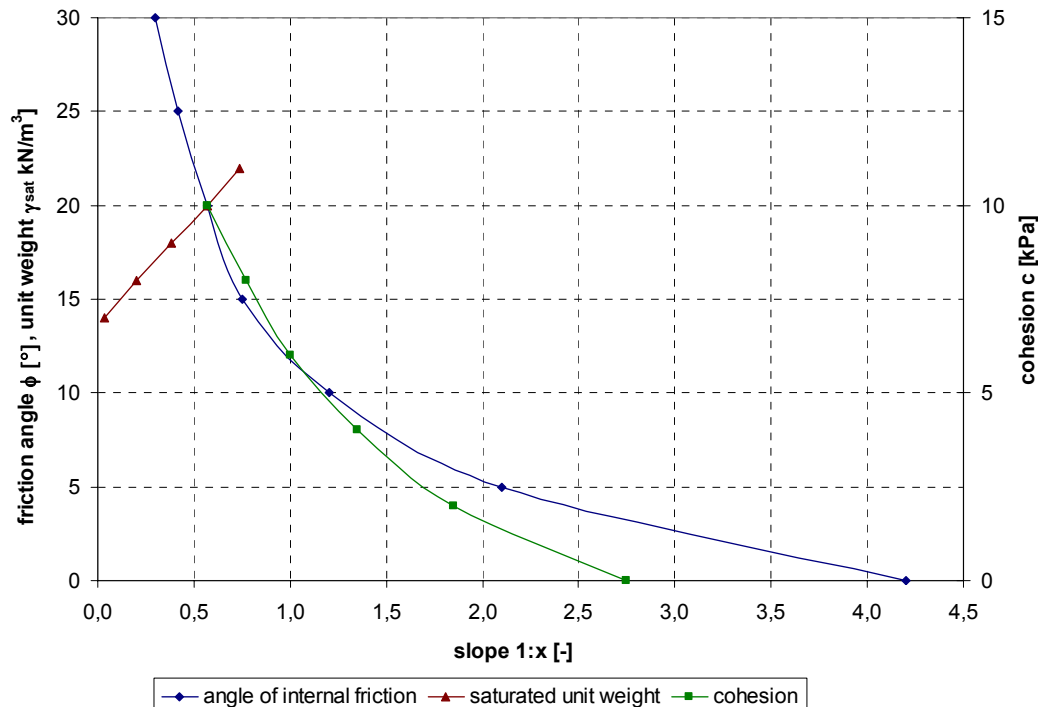


Figure 4-5: Critical slopes (SF=1) for varying friction angle, unit weight and cohesion

Undrained conditions

Comparing drained calculations with undrained calculations can only be justified if both the drained and undrained soil properties of a certain soil are accurately known. However, no definite relation between those properties exists; all depends on the specific stress paths of a certain soil. Because a wild guess of the undrained shear strength would make no sense, the following relation, not theoretically founded and therefore only indicative, is used, based on the idea that the isotropic effective stress will remain more or less constant and can be estimated (no contractant or dilatant behaviour) as follows [Verruijt, 1999]:

$$s_u = c \frac{\cos \phi}{1 - \frac{1}{3} \sin \phi} + \sigma_0' \frac{\sin \phi}{1 - \frac{1}{3} \sin \phi} \quad \text{Eq. 4-3}$$

The total (constant) isotropic stress is defined as ($\sigma_{xx} = \sigma_{yy}$):

$$\sigma_0' = \frac{1}{3}(\sigma_{zz}' + 2\sigma_{xx}') \quad \text{Eq. 4-4}$$

When the stress-strain relationship is considered linear elastic, the ratio between horizontal and vertical effective stress (under water) becomes:

$$K' = \frac{\nu}{1 - \nu} \quad \text{Eq. 4-5}$$

in which ν is the Poisson ratio.

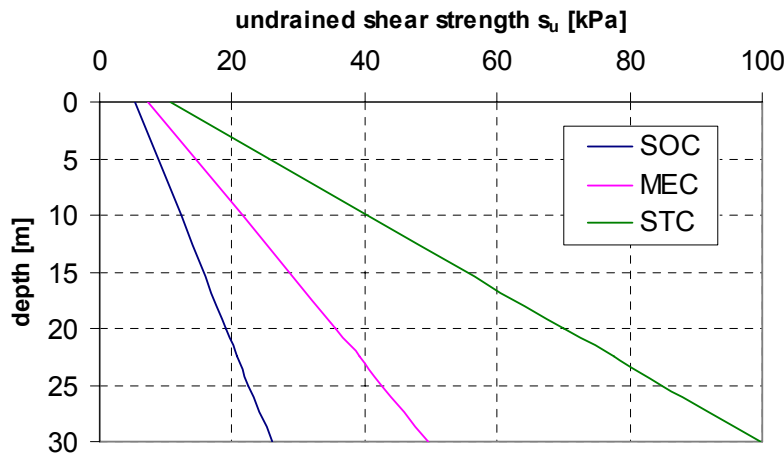


Figure 4-6: undrained shear strength s_u against depth

Together with the saturated unit weight of the reference clays, the undrained shear strength is known over the entire depth, see Figure 4-6. As a rough criterion one often uses that the shear strength increases in depth with 0,20 to 0,25 times the vertical effective stress. In case of the reference soils, this value is a bit less for SOC (0,17) and a bit more for STC (0,30) with MEC (0,21) in between.

In MStab a number of 36 calculations have been done for different slopes (1:1,5; 1:2; 1:2,5 and 1:3) and different slope heights (5, 10 and 20m), all completely situated below sea level.

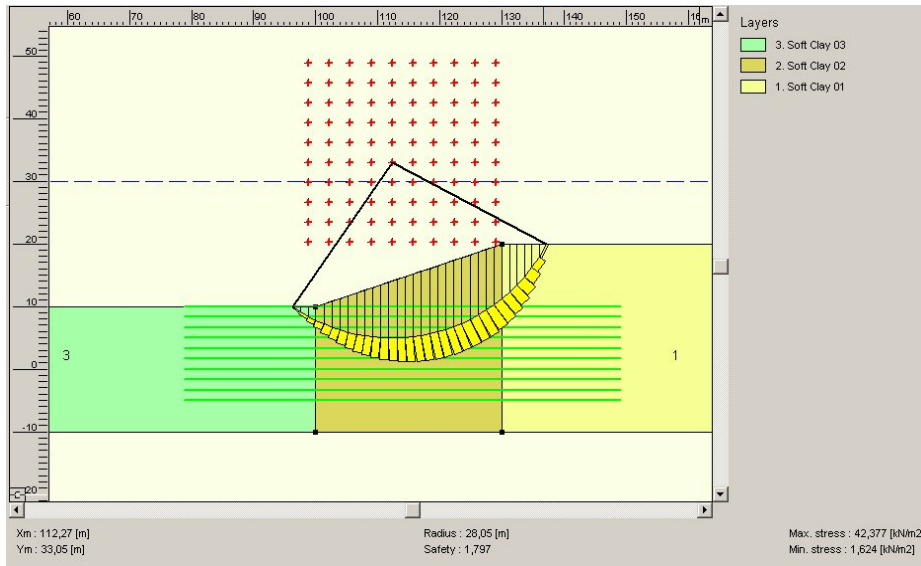


Figure 4-7: Bishop slip circle in three-layer seabed in soft clay (SOC)

In MStab the shear strength on top and at the bottom of a soil layer has to be entered. Because of the varying thickness over the trench, this would result in a varying gradient ($\delta s_u / \delta z$). The shear strength at the trench bottom would increase faster with depth, thereby increasing stability. Three soil layers were applied to give a good representation of the undrained shear strength, see Figure 4-7.

The results of the undrained calculations (s_u) were compared to the results of the drained calculations (c, ϕ) and plotted in Figure 4-8 and Figure 4-9; the solid lines represent the drained, the dashed lines the undrained calculations.

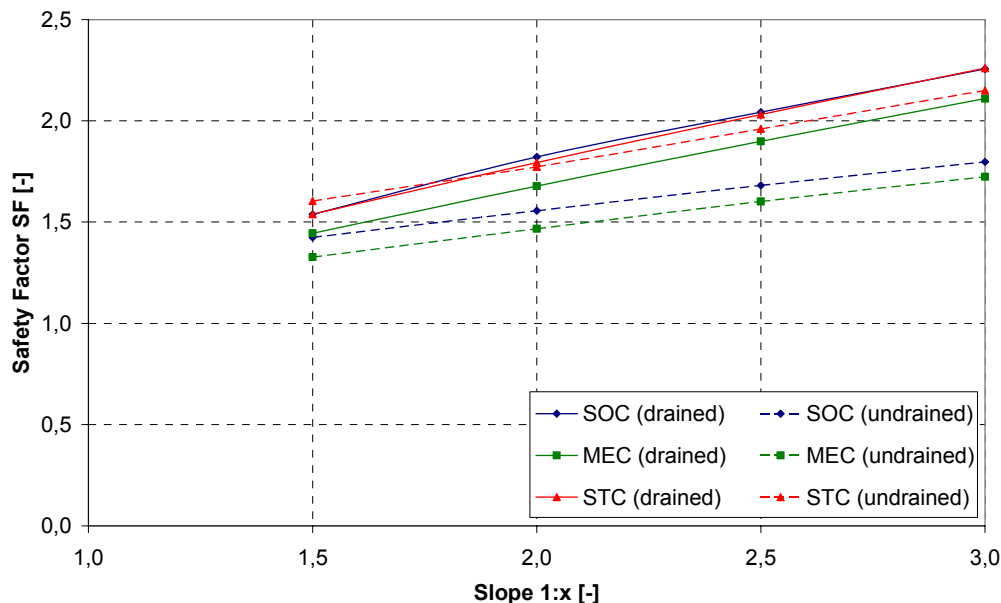


Figure 4-8: Safety factors for different initial slopes and reference soils under drained and undrained conditions

It can be observed that in all reference clays rather steep slopes remain stable in both drained and undrained calculations. Slopes of 1:2 with a height of 10 m are unconditionally stable in all considered clays (knowing that these clays were not that stiff or firm). In case of undrained calculations, the safety factors are somewhat lower, especially for flatter slopes. Flattening the side slopes is a more effective measure if the soil shows drained behaviour: the 'drained' lines increase faster.

Again the medium clay (MEC) was the weakest of the three, due to the combination of a rather large saturated unit weight and small increase of the undrained shear strength with depth (in the linear elastic relation dependent on cohesion, friction angle, Poisson ratio and unit weight) compared to the soft clay (SOC). SOC and MEC show similar behaviour: the larger strength-related properties (c , ϕ and s_u) of SOC are almost completely counterbalanced by its large load-related property (unit weight). One should keep in mind that this statement only holds for the unloaded static situation. Large dynamic loads can have a larger impact on softer clays.

When varying the slope height (and keeping the angle of inclination constant at $1/\tan(\beta)=2$), it appeared that all clays show similar behaviour under drained and undrained conditions, see Figure 4-9. In both cases, the safety factors are strongly reduced for slope heights from 5 to 10 m, but this effect gradually starts to diminish for larger slope heights. Furthermore, it can be concluded that undrained calculations yield somewhat flatter critical slopes than drained calculations and this difference becomes larger for higher slopes. Slope heights of 15 m or more become critical for SOC and MEC in undrained conditions and 1:2-slopes.

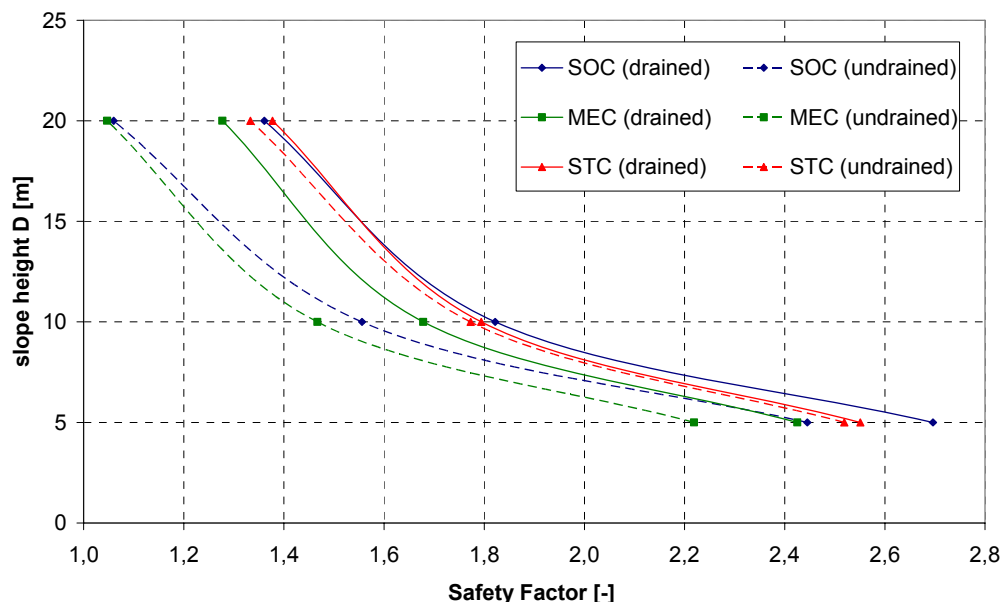


Figure 4-9: Development of safety factor for varying slope height under drained and undrained conditions; 1:2 side slopes

Another remark is made with respect to the consolidation properties of submarine clayey slopes. When subjected to rapid sedimentation, the clayey material in the seabed is in a state of underconsolidation. Towhata and Kim [1990] conducted several triaxial compression tests in order to understand the undrained behaviour of underconsolidated clay. The test results indicated that the undrained strength of the clay increases with the degree of consolidation, while the cohesion and friction angle in terms of effective stress are independent of the degree of consolidation. These results show that over the entire depth ('old' and 'new' material) the same cohesion and friction angle can be applied, but that the exact undrained shear strength can vary over depth. Just after dredging of the channel, the channel bottom and side slopes will be overconsolidated, because in fact an overburden has been removed. Freshly deposited sediments are in a state of underconsolidation, which reduces the strength. The more rapid the sedimentation takes place, the larger this effect will be. A more conservative slope design is therefore advised when large sedimentation rates are to be expected.

Finally, in addition to the determination of the undrained shear strength, the following line of thought will be briefly mentioned. Dolinar and Trauner [2000] claim that the undrained shear strength in fully saturated clays is dependent on the quantity of free water. The quantity of water in clay can be divided into free pore water between the grains ('intergrain' water) and water that is strongly bound *between* the layers of clay particles ('interlayer water'). The quantity of intergrain water (w_e) is further subdivided into free pore water (w_{ef}) and firmly adsorbed water *around* the grains (w_{ea}):

$$w_e = w_{ea} + w_{ef} = d_a A_{SeC} + w_{ef} \quad \text{Eq. 4-6}$$

in which:

$$\begin{aligned} d_a &= \text{thickness of firmly adsorbed water} & [10^{-8} \text{ m}] \\ A_{SeC} &= \text{specific surface of clay} & [\text{m}^2/\text{g}] \end{aligned}$$

The quantity of free water is a good measure for the thickness of the water film around the clay grains. Dolinar and Trauner found that the quantity of free water of different soils is the same at the same undrained shear strength. According to this line of thought, the undrained shear strength for different clay samples is the same at the liquid limit (w_L), because at this limit the water content of free pore water is the same. The same conclusion can be drawn for clay samples at the plastic limit (w_P):

$$\text{at the liquid limit for all clays:} \quad s_{u(w_L)} = 2,66 \text{ kPa} \quad \text{Eq. 4-7}$$

$$\text{at the plastic limit for all clays:} \quad s_{u(w_P)} = 266 \text{ kPa} \quad \text{Eq. 4-8}$$

In most cases, however, the undrained shear strength will lie between both values.

Table 4-4: Test results on three clay samples; (I) well crystallized kaolinite; (II) poorly crystallized kaolinite; (III) Ca-montmorillonite [source: Dolinar and Trauner, 2000]

quantity	symbol	unit	liquid limit			plastic limit		
			w_L			w_P		
			I	II	III	I	II	III
specific surface of clay sample	A_{SeC}	m^2/g	10,05	23,50	97,42	10,05	23,50	97,42
undrained shear strength	s_u	kPa	2,66	2,66	2,66	266	266	266
intergrain water	w_e	%	40,0	50,9	110,8	25,8	29,8	50,0
free pore water	w_{ef}	%	31,9	31,9	31,9	23,1	23,3	23,1
adsorbed water around grains	w_{ea}	%	8,1	19,0	78,5	2,7	6,5	26,9
thickness of adsorbed water layer	d_a	10^{-8} m	0,81	0,81	0,81	0,27	0,27	0,27

Please note that purely cohesive soils ($\phi=0$) can be still stable if the slope is a vertical wall as long as the slope height stays within certain limits. Research on this critical slope height was done by many researchers (Mohr-Coulomb, De Josselin de Jong, Pastor and Fellenius). The upper and lower limit of the slope height under drained conditions can be defined as follows:

$$\frac{3,64c}{\gamma} \leq h_c \leq \frac{3,83c}{\gamma} \quad \text{Eq. 4-9}$$

When dredging temporary shallow pipeline trenches, vertical cut slopes can be a good solution, see Table 4-5.

Table 4-5: Upper and lower limits of vertical wall height

reference soil	h_c [m]	
	lower	upper
SOC	1,29	1,36
MEC	1,53	1,62
STC	1,83	1,92

4.2 Liquefaction and flow slides

4.2.1 Static liquefaction

A liquefaction flow slide is a phenomenon in which a mass of sand on a subaqueous slope suddenly starts behaving like a viscous fluid. Although the use of this term is widely adopted, it is misleading:

- as sliding suggests downslope movement of a mass on a planar failure plane, with no internal deformation, which is not the case in these slides;
- because it also suggests that liquefaction is the sand-supporting mechanism during the whole event, whereas it only concerns the mechanism initiating the failure.

Liquefaction occurs when the framework of loosely packed fine-grained sand, saturated with pore water, suddenly collapses as a result of some external triggering agent. This is caused by contractant behaviour which means that the total volume of sand particles tends to decrease when subjected to shear deformation. The pore water prohibits this reduction in volume and an increase in pore pressure occurs, see Figure 4-10.

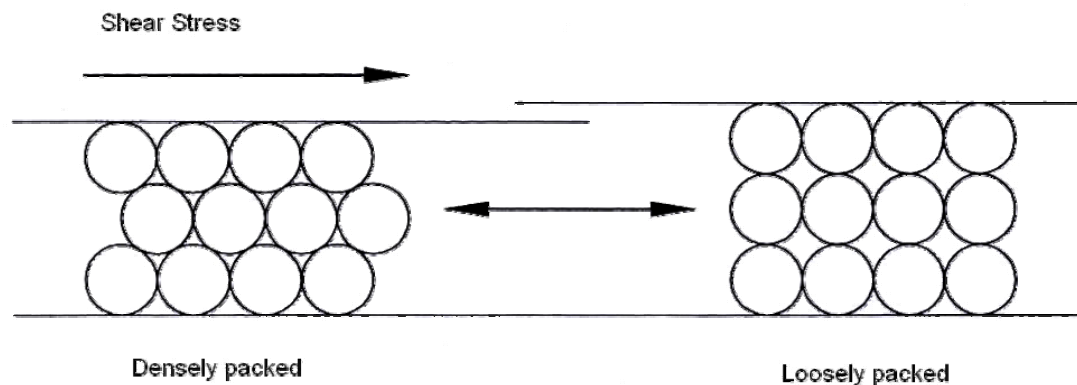


Figure 4-10: Schematization of (reversible) behaviour of densely and loosely packed grains subjected to shear stress

The above mentioned sudden 'triggering agent' can appear in many guises, like cyclic shear stresses during earthquakes, waves, dredging operations or small slides. With (medium) fine sand in a layer of several meters thick, the drainage period is several minutes. Thus, the load increase by the tide or the load increase by scour at the toe or sedimentation at the top of a slope cannot be considered "sudden". However, these loads can increase stresses, thereby reducing stability.

It is generally believed that a liquefied sand flow may accelerate and, if the slope is steep and sufficient long, can become turbulent, because the thickness of the flow and high velocity result in Reynolds numbers well above the criterion for turbulent flow. In the relatively small space scales of navigation channels and pipeline trenches it is however questionable if such a hyperconcentrated density flow can develop into a turbidity current.

It should be noted that the understanding of liquefaction flow slides is based on larger events like the channel bank failures in the Scheldt estuaries (Silvis, 1995), although this kind of failures may occur more frequently in a smaller form in channels.

A rough criterion used in the Province of Zeeland tells that slopes steeper than 1:3 and higher than 5 m are very susceptible to flow slides, although failures of flatter slopes have been reported. Although, one may conclude that flow slides in trenches are unlikely because of the small slope height, often less than 5 m.

Furthermore flow slides do not occur only in sand. They are also found in other materials with a loose packing like in loose gravel, ore or coal deposits. Cohesive soils with more than 20% of particles finer than 5 μm , or with liquid limit (LL) of 34 or greater, or with the plasticity index (PI) of 14 or greater are generally considered not

susceptible to liquefaction [Slope Stability Engineering Manual, 2003]. In a somewhat different form they are also found in clays with certain specific properties. Notorious are the so-called quick-clay-slides in Norway, Sweden and Canada.

As first rough rules-of-thumb it can be stated that soils with the following conditions are susceptible to liquefaction:

1. Soil conditions
 - a. low relative densities (see Paragraph 3.1.2):
 $RD < 60 \%$: risk of liquefaction is present;
 $RD < 30 \%$: risk of liquefaction is most likely;
 - b. poorly permeable sands (fine to medium sands; grain sizes of 50 – 1000 μm);
2. Slope geometry
 slopes higher than 5 m and steeper than 1:3 (or up to 1:7 if slopes become higher);
3. The presence of a trigger mechanism
 like a small earthquake or a large wave;

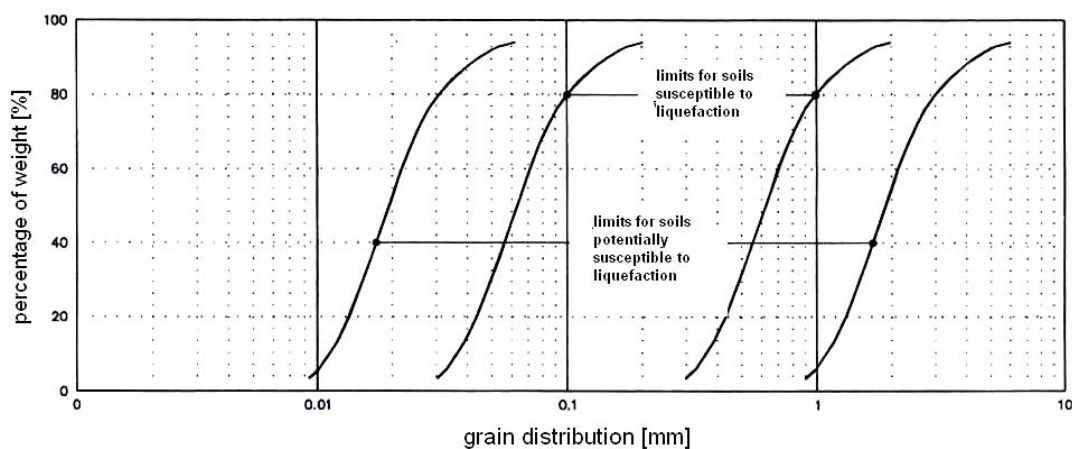


Figure 4-11: Susceptibility to liquefaction (Ishihara, 1986) [source: Stoutjesdijk, 1994]

Many researchers state, that some kind of trigger mechanism will always occur. Therefore attention should be given to the first conditions. If soils are susceptible to liquefaction, design a more conservative slope!

Liquefaction is a prerequisite for the occurrence of a flow slide. Liquefaction takes place if at least one sand element is in a metastable stress state. This means that the undrained response to any quick change in load, however small it may be, consists of a sudden large increase in pore pressure [Stoutjesdijk, 1998].

The slope geometry alone does not give enough information on the risk of a flow slide, although in the past some empirical relationships have been developed. Better is to integrate the degree of liquefiability of the soil and the slope geometry parameters. This approach has been adopted by GeoDelft [Silvis 1988, 1995, Stoutjesdijk 1994, 1995, 1998] when developing a numerical model, which forms the base of a computer program, SLIQ2D (*static liquefaction 2-dimensional*). This abbreviation clearly tells that this program can only be used under static conditions. Loads due to dredging works or cyclic loads, like waves and earthquakes, are not accounted for.

The actual process of and the excess pore pressures during a flow slide have been investigated during research on the construction of sand fill dams [Bezuijen, 1988]. This research showed that these excess pore pressures remained zero for a long time, but suddenly rose to values up to 90% of the original vertical effective stress. Initial stable slopes became unstable and sliding occurred and lasted till the excess pore pressures disappeared. The time scale was in the order of minutes and the resulting slopes were in the order of 1:10-30. After sliding, the porosity was only slightly decreased (up to 3 %), which means that in some cases the soil was still susceptible to liquefaction.

An important quantity which describes whether a certain soil will show contractant or dilatant behaviour when subjected to a shear stress is the critical density. Distinction is made between dry and wet critical density. Dry critical density is determined by subjecting soil sample of different densities to a shear stress in a triaxial shear test apparatus under drained conditions. The density at which neither a volume increase nor a volume decrease occurs is called the 'dry critical density', see Figure 4-12, located in the right-hand graph where the line crosses the y-axis.

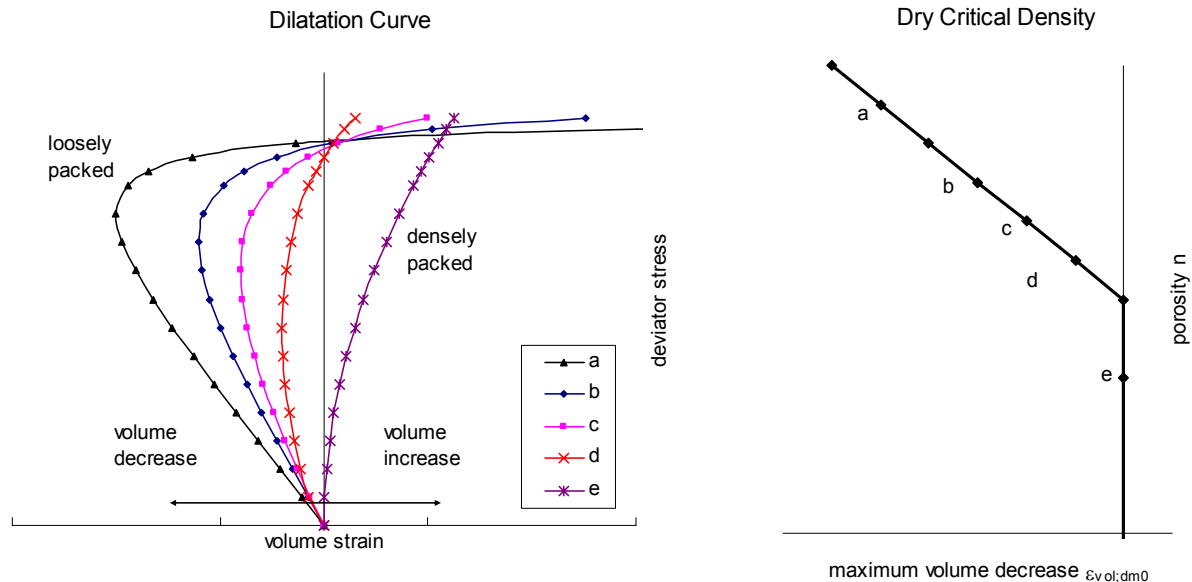


Figure 4-12: Dry critical density; 'a' up to 'e' represent soil samples with different porosity

On the other hand there is the wet critical density test under undrained conditions. Now if a shear stress is put on the soil sample, the water isn't able to flow out and the volume of the sample cannot change. The result will be an increase in pore pressure. The density at which the soil just starts to collapse is the wet critical density. The dry and wet critical density are not the same, see Figure 4-13. If the sand is more densely packed than the dry critical density, an increase of the deviator stress will result in an increase of the isotropic stress, line A. Line B and C represent the situation of a packing between the dry and wet critical density. Subjected to a shear stress, the sand skeleton will decrease its volume and water overpressure occurs. This does not necessarily result in liquefaction, as will be the case if the sand is more loosely packed than the wet critical density: line D. Any small change of the shear stress may cause liquefaction and therefore collapsing of the soil.

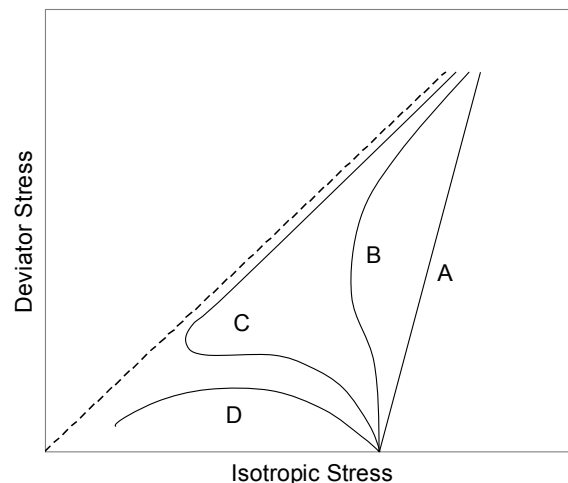


Figure 4-13: Stress paths for different densities

It should be clear that the dry critical density is a safe upper boundary. Densities larger than the dry critical density are unconditionally stable. A density between the dry and wet critical density is indicated as 'possible susceptible to liquefaction' and a density smaller than the wet critical density is 'very susceptible to liquefaction'. It should be noted that susceptibility does not necessarily have to result in a flow slide; the geometrical properties have to be taken into account.

SLIQ2D considers dry critical densities and soils under drained conditions, but introduces a small undrained change to investigate the susceptibility to flow slides. The underlying concept is that the building up of soil layers is a very slow process (drained behaviour), whereas a sudden change in load will cause undrained behaviour. Because of this concept, this program is not applicable under dredging conditions or other quick stress changes.

In SLIQ2D a lot of calculations have been done with Pluto (finite element method) to obtain the initial soil stresses for different geometries. SLIQ2D uses these stresses and determines whether a small change in shear stress under undrained conditions causes a volume decrease. This can be expressed in a simple way:

$$d\gamma_{xy} = \frac{1}{\lambda} d\tau_{xy} \quad \text{Eq. 4-10}$$

Parameter λ is the eigenvalue of the stiffness matrix describing the incremental deformation of the soil elements in the critical zone of the slope as a function of a sudden stress change [Molenkamp (1989) as described by Stoutjesdijk, 1994]. This eigenvalue becomes negative when an increase of the shear strain γ_{xy} causes a decrease of the shear stress τ_{xy} .

In SLIQ2D the slope geometry is defined and then the program successively:

- 1 reads the input;
- 2 divides the geometry in 500 points;
- 3 calculates the initial stresses starting from a very flat slope;
- 4 calculates the eigenvalue λ ;
- 5 steepens the slope a little bit and repeats step 3 and 4.

The output contains for each grid point the flattest slope at which the eigenvalue becomes negative. The program considers a certain slope geometry to be unstable if at one point the eigenvalue becomes negative. In reality there will be some residual strength, so the output will be conservative.

Input

The input consists of geometry, material and fit parameters. The geometry can be represented as a straight slope or a bended slope with the steepest part at the toe of the slope. In the latter case two slope heights and the slope angle of the upper part have to be defined. Under the assumption that failure will occur in the lower part, only this part is steepened.

The soil is represented by the unit weight under water, which is about 10 kN/m³, the elastic Poisson ratio of about 0,3 and the non-elastic (plastic) Poisson ratio of 0,4.

To obtain reliable input for the fit parameters, relatively much effort has to be made. Because of the sensibility to the different soil parameters, some tests have to be done:

- make borings to obtain soil samples;
- determine in-situ density with electrical conductivity measurements;
- examine drained triaxial test to determine the stress-strain curve

With the results, the input parameters can be calculated.

Figure 4-14 shows a cross-section of the grid points in the bed where the soil starts to liquefy. This graph shows a cross-section through a side slope; the top of the slope is located at the upper left corner. One should keep in mind that this graph is a distorted view of reality. SLIQ2D uses 500 grid points with changing 'real' x- and z-coordinates, if the bed profile is steepened. Therefore the peak points in reality are situated at much larger real x-coordinates, because of the smaller angle of a just stable slope; the stable points belong to a steep slope and have smaller x-coordinates. Nevertheless, the above presentation is chosen, because it is able to give a quick impression of the number of unstable points and the 'rate of instability' (= the height of the peak). Moreover, the location of initiation of instability can be observed. It is easy to get an impression of the area above a certain isoline (slope of 1:y) and to determine the measurements to increase stability. For example, a large flat bump means that stability can easily be reached by flattening the slope a little bit. Slender, high peaks probably does not give severe problems because of the residual strength of the neighbouring soil particles, but wide, high peaks demand serious measures. The soil may have to be compacted. In any case, an additional research study is recommended.

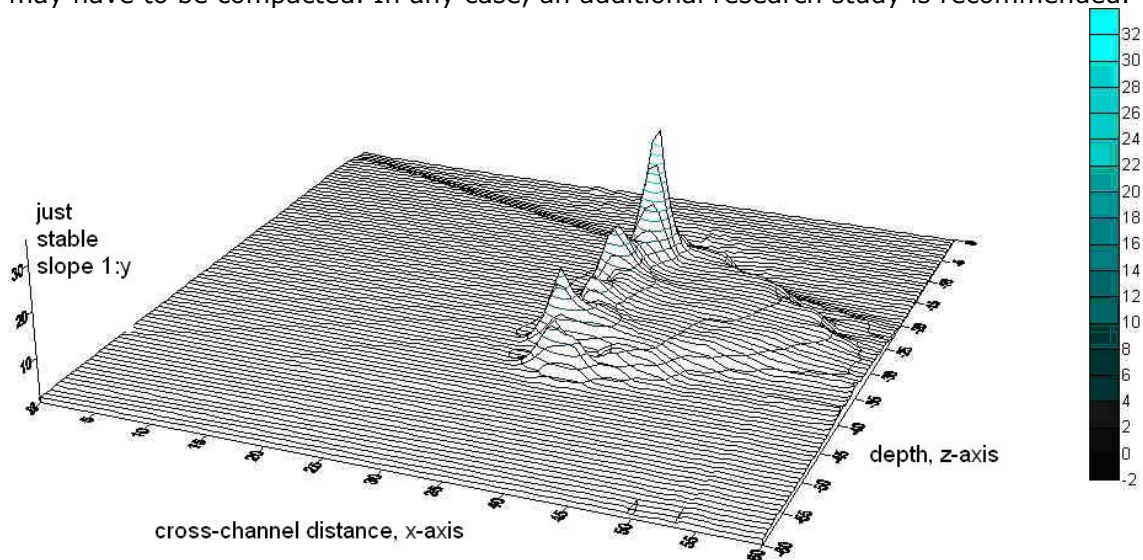


Figure 4-14: An example of the output of a calculation with SLIQ2D; the plane is a cross-section of a side slope of a trench

The accuracy of the results is expected to be limited; nevertheless the safety against failure is considered sufficient. Because of the use of values of a dry triaxial test, the results probably are conservative. Points of interest, however, are the spread in the input parameters, the influence of thin layers of clay or mud and the determination of in-situ stresses. On the other hand, some redistribution of soil stresses is likely, water pressures can dissipate a little and liquefaction in a very small area (a few grid points) probably does not cause failure.

The erosion length is defined as the length over which a flow side extends. Stoutjesdijk developed a probabilistic expression based on data of flow slides collected by Wilderom in 1979. This length $L_{\text{flowslide}}$ is proportional to the slope height D :

$$L_{\text{flowslide}} = \bar{K}_{fs} D \quad \text{Eq. 4-11}$$

in which K_{fs} is normally distributed with mean $\mu = 2,63$ and standard deviation $\sigma = 1,69$, see Figure 4-15:

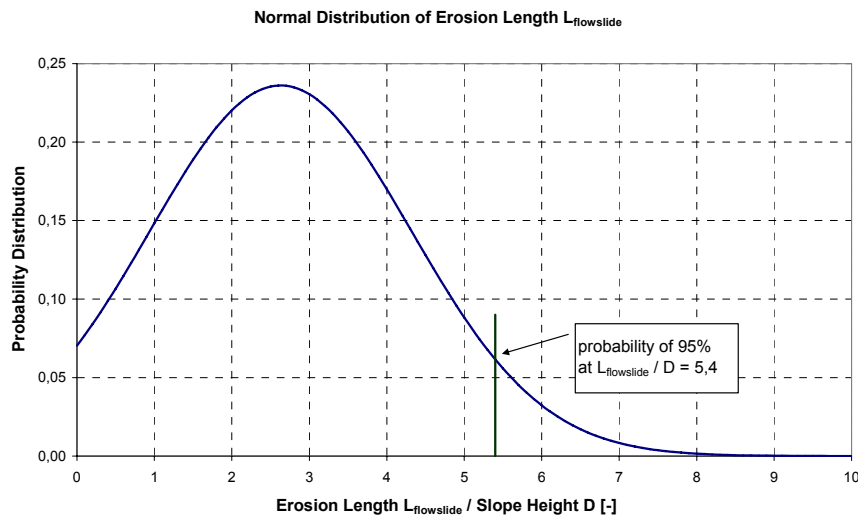


Figure 4-15: Normal distribution of the erosion length divided by the slope height

Knowing this erosion length, the erosion volume can be estimated by making a few assumptions:

- the final flow side surface crosses the initial slope at half of the slope height D ;
- the shape of the final eroded part of the geometry can be described by a square root function, see Eq. 4-11 and Figure 4-16:

$$z(x) = \sqrt{\frac{D}{4\bar{K}_{fs} + \frac{2}{\tan \beta}}} \sqrt{x} \quad \text{for } 0 \leq x \leq \left(\bar{K}_{fs} + \frac{1}{2 \tan \beta}\right)D \quad \text{and } 0 \leq z \leq \frac{1}{2}D \quad \text{Eq. 4-12}$$

- the change in volume is negligible, because the decrease in porosity usually is very small; so the 'disappeared' erosion volume on top of the slope is equal to the sedimentation volume at the toe of the slope.

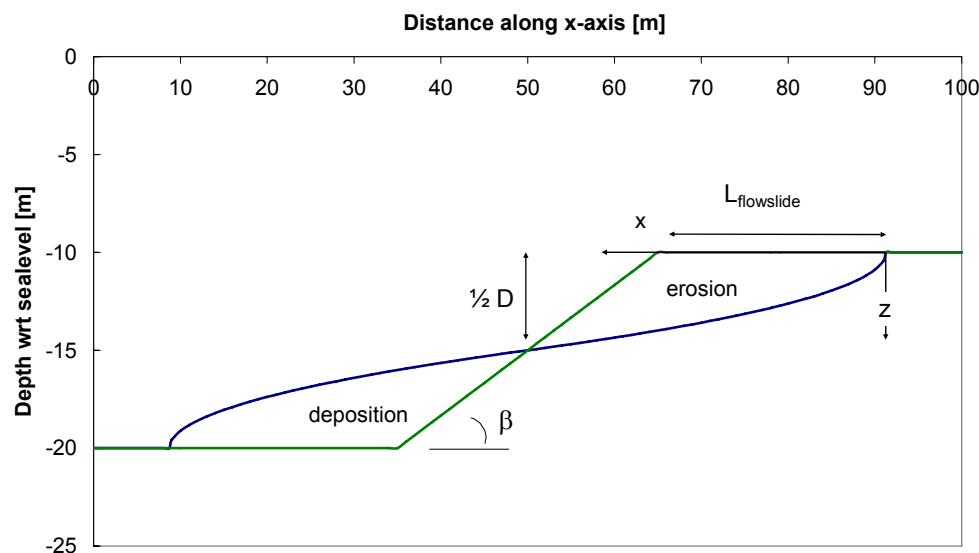


Figure 4-16: Assumed square root profile of geometry after flow slide

Now, the erosion volume per unit width can with the help of some mathematics be expressed as:

$$V_{fs} = \frac{1}{3} \left(\bar{K}_{fs}(\mu, \sigma) + \frac{1}{8 \tan \beta} \right) D^2 \quad \text{Eq. 4-13}$$

For example, the erosion volume of a slope height of 10 m and an initial slope angle of 1:3 already becomes 100,7 m³/m, when using the mean value for the erosion factor K_{fs} ; considering the 95 % probability area, this erosion volume can increase up to 192,5 m³/m. The corresponding post-failure average slopes are respectively 1:8 and 1:14.

It should be noted that the above estimations show a huge spread, because of the wide variation in erosion lengths and observed geometries. However one should understand that erosion volumes this large almost always cause failure of the navigation channel, i.e. the required depth and/or width cannot be guaranteed anymore.

4.2.2 Computations with SLIQ2D

With SLIQ2D a number of calculations have been done for slope heights of 5, 10 and 20 m and for soils with porosities of 0,42 (dry critical density) to 0,48 (maximum porosity). The soil properties are primarily characterized by the relation between porosity and maximum dilatant volume strain according to Figure 4-17.

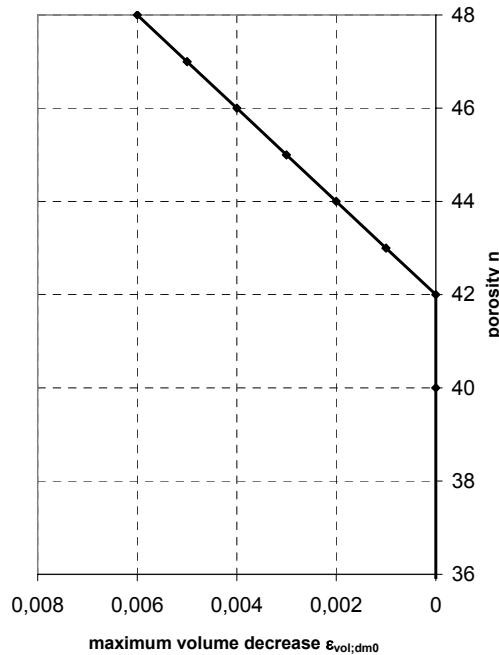


Figure 4-17: Relation between porosity and maximum dilatant volume strain, based on assumptions for minimum and maximum porosity and dry critical density

As was mentioned above, also the exact stress-strain curve has to be known to generate the input data. Of course this curve is different for every soil, but Figure 4-18 (which is a rotated version of Figure 4-12) shows a frequently observed curve, based on relative strains and shear stresses, which will be used in the calculations with SLIQ2D. The corresponding formula is:

$$\epsilon_{vol_d} = \epsilon_{vol_{dm0}} \left(A s^m - \frac{B s^r}{s_{max} - s} \right) \quad \text{Eq. 4-14}$$

in which:

$$A = \frac{1}{s_2^m} + \frac{m}{s_2^m \left(r - m + \frac{s_2}{s_{\max} - s_2} \right)} \quad \text{and} \quad B = \frac{m(s_{\max} - s_2)}{s_2^r \left(r - m + \frac{s_2}{s_{\max} - s_2} \right)}$$

$\varepsilon_{\text{vol}_d}$ = dilatant volume strain
 $\varepsilon_{\text{vol}_{dm0}}$ = maximum dilatant volume strain
 s = relative shear stress
 m, r = 'fitting constants'
 s_2 = relative shear stress at which $\varepsilon_{\text{vol}_{dm0}}$ occurs
 $s_{\max}$ = asymptote of relative shear stress

The values of the parameters s_2 , $s_{\max}$, m , r , A and B are plotted in the graph. The sensitivity of these parameters is investigated in the sensitivity analysis.

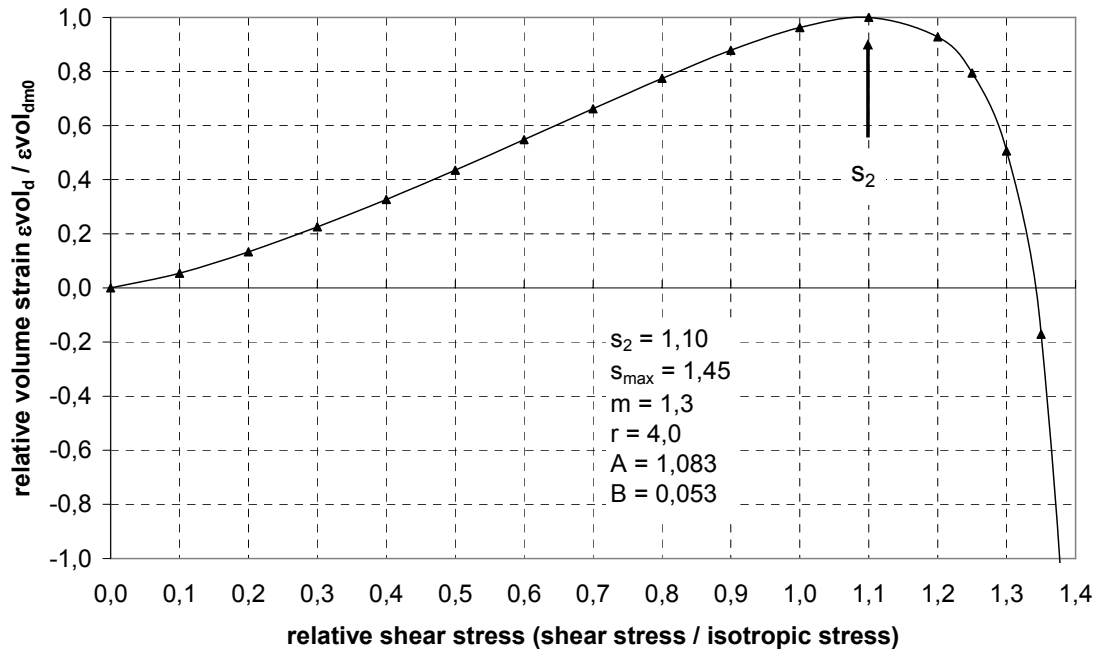


Figure 4-18: Graph of relative volume strain against relative shear stress

A factor 'v' is introduced that discounts for the fact that the isotropic stresses in the field can differ from the isotropic stress in the test (50 kPa) on which the above formulations (dilatation curve) are based. A recommended value for this factor 'v' is 1,25;

The second curve that needs to be fit is the decompression curve, which describes the volume strain as function of the isotropic stress:

$$d\varepsilon_{\text{vol};c} = -\frac{d\sigma'_{\text{vol}}}{K_s} \quad \text{Eq. 4-15}$$

The decompression modulus ' K_s ' is dependent on the isotropic stress, in other words the stress-strain relationship is not linear. The following relation yields the last two SLIQ2D-input parameters ' K_{s0} ' and ' u ':

$$K_s = K_{s0} \left(\frac{\sigma'_{\text{vol}}}{\sigma'_{\text{vol};0}} \right)^u \quad \text{Eq. 4-16}$$

in which K_{s0} and $\sigma'_{\text{vol};0}$ respectively represent the decompression modulus and the isotropic stress at the begin of decompression. Common values for both parameters are:

$$40.000 \leq K_{s0} \leq 60.000 \text{ kPa}$$

$$0,5 \leq u \leq 1,0$$

Sensitivity Analysis

With this 'schematized soils' a number of calculations has been done. At first the sensitivity of the most important input parameters has been investigated. In Table 4-6 can be seen that the 'fitting parameters' of the above mentioned stress-strain-curve hardly have any influence on the slope stability. This is favourable, because determination of this graph takes a lot of effort. On the other hand, it can be concluded that a good estimation of the maximum volume strain and the decompression modulus is rather important. The almost linear relationship between porosity and maximum volume strain for porosities larger than the dry critical porosity can be helpful.

Table 4-6: Sensitivity of the most important input parameters

parameter	description	unit	default value	min	max	$(1/\tan \beta)_{\text{default}}$	$(1/\tan \beta)_{\text{min}}$	$(1/\tan \beta)_{\text{max}}$	sensitivity
n	porosity	-	45	44	46	4,75	4,75	4,5	p
n_{max}	maximum porosity	-	48	47	49	4,75	4,75	4,75	0
n_{min}	minimum porosity	-	30	-	-	4,75	-	-	-
evol_{dm0}	maximum volume strain	-	0,003	0,002	0,004	4,75	3,25	5,5	N
m	fitting constant	-	1,3	1	1,6	4,75	5	4	p
r	fitting constant	-	4	3,5	4,5	4,75	4,5	4,25	p
s₂	relative shear stress at evol _{dm0}	-	1,1	1	1,2	4,75	5	4	p
s_{max}	asymptote of shear stress	-	1,45	1,35	1,55				
v	power of isotropic stress	-	1,25	0,5	3	4,75	4,5	5,5	n
u	power of decompression curve	-	1	0,5	1	4,75	4,5	4,75	n
K_{s0}	decompression modulus	kPa	50000	40000	60000	4,75	3,75	5,5	N

N = increase of parameter strongly influences stability negatively; n = little negative influence; 0 = hardly any influence;
p = little positive influence; P = strongly positive influence

Calculations for different heights

For sands with different porosities, the slope heights have been varied, see Figure 4-19. On the right y-axis, the relative density is presented. Because this parameter is linearly related to the void ratio, but not to the porosity, the scale is a bit distorted.

Two important conclusions can be drawn:

- the slope height is very important: doubling of the slope height requires reducing the slope steepness with a factor 2;
- the in situ porosity has to be determined as accurately as possible if the soil is susceptible to liquefaction.

This means that small variations between design and construction can reduce stability significantly. Dredging of an overdepth of just 1 m has to be considered in the design phase.

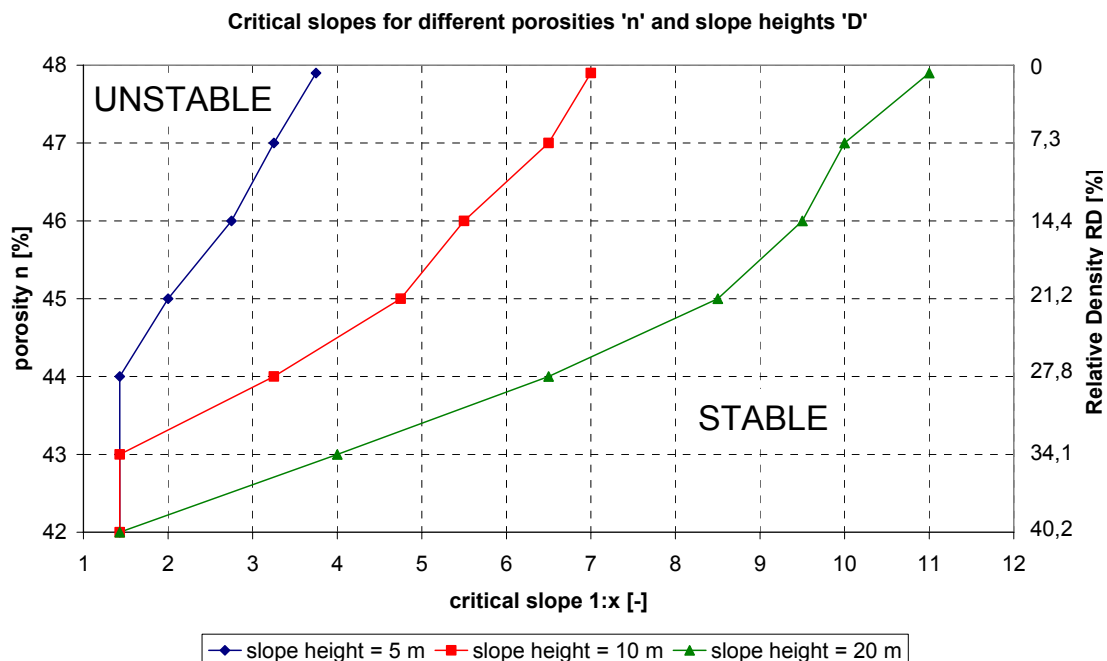


Figure 4-19: Critical slopes for different porosities (Relative Densities) and slope heights

Bended slopes

Also has been investigated whether improvements in the slope geometry can be applied to reduce the dredging volumes maintaining the same stability, or to decrease the required width. After all, instability originates in the lower half of the slope, so flatter lower slopes and steeper upper slopes could be more stable.

Without explaining how these bended slopes can be implemented in SLIQ2D, the results are presented in Table 4-7 and compared to the default run. Grey cells show an improvement compared to the default run.

Table 4-7: Results of calculations with bended slopes compared to the 'default' run

		bended profiles				
		default	I	II	III	IV
h_1	m	10	5	5	5	2,5
$\tan\beta_1$	-	1:4,75	1:3,75	1:5	1:4	1:5
h_2	m		5	5	5	5
$\tan\beta_2$	-		1:5,63	1:4,5	1:5,33	1:4,25
h_3	m					2,5
$\tan\beta_3$	-					1:5,31
$\tan\beta_{\text{average}}$	-	1:4,75	1:4,69	1:4,75	1:4,67	1:4,70
A_{dredged}	m^3/m	238	258	231	250	286
$L_{\text{slope, x}}$	m	47,5	46,9	47,5	46,7	47,0

All bended profiles are also plotted in Figure 4-20. Bended profiles I and III have steeper upper parts and flatter lower parts. With these profiles the upper channel width can be reduced, but the dredging volume of the slope increases. Bended profile II has a flatter upper part and a steeper lower part, which reduces the total dredging volume, but increases the upper width. 'Outsider' profile IV has its steepest part right in the middle of the slope. The upper width is hardly reduced, while the total dredging volume is increased. This profile seems to have no advantages at all, but this slope geometry resembles the morphological profile after some time best. This profile is therefore expected to 'keep in (its initial) shape'. Depending on the hydrodynamic conditions in combination with the sediment properties, this channel geometry may perform best in the long term.

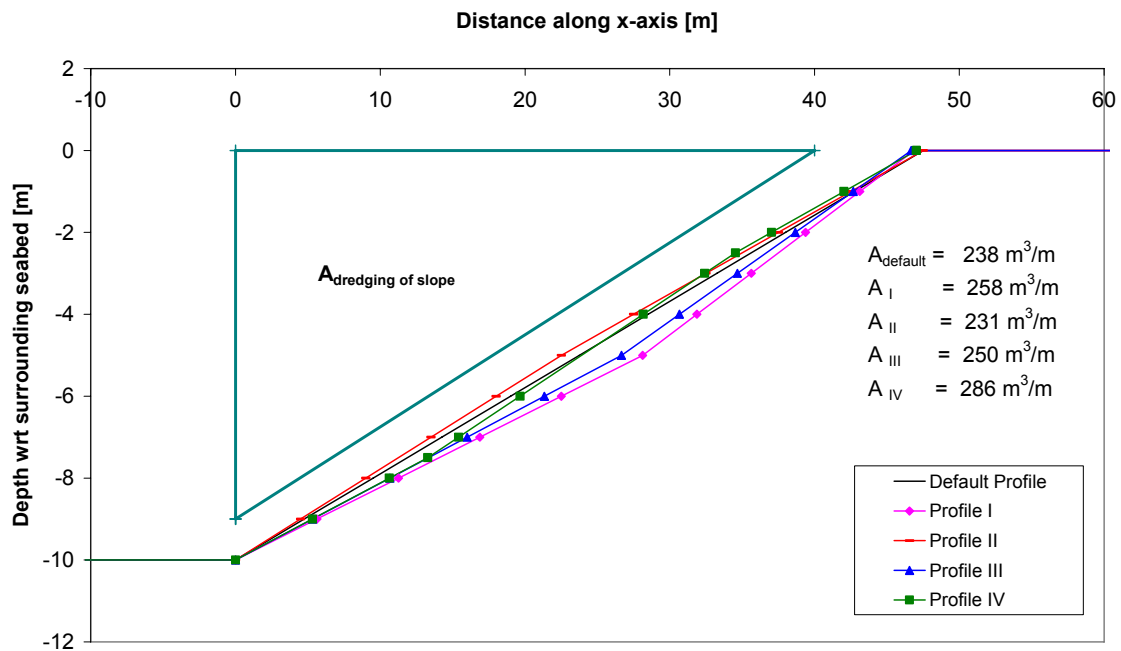


Figure 4-20: Schematization of bended profiles and total slope dredging volume

Despite all good intentions, the differences of the upper channel width and slope dredging volume between the default profile and the 'bended profiles' remain small. The minimal obtained reduction in upper channel width or total dredging volumes does not compensate for the extra effort to design and dredge these rather sophisticated profiles. This measure of 'bending slopes' is therefore not recommended, unless some other benefits can be gained. As will appear in later chapters, sometimes dredging of a sedimentation reservoir at the side slopes prolongs the lifespan of a trench or channel. Then, the measure of 'bending profiles' can be extremely useful, because the upper width remains limited while the channel is provided with a reservoir at the same time. Also it was mentioned that profile IV may have some morphological advantages.

Conclusions on SLIQ2D-calculations

- Soils with densities larger than the 'dry critical density' ($n < 0,42$) are unconditionally stable;
- Soils with densities smaller than the 'dry critical density' ($n > 0,42$) can still be stable, depending on the soil geometry;
- Slopes with heights smaller than 5 m are almost unconditionally stable. Only very loosely packed soils ($RD < 10$) are susceptible to liquefaction.
- When considering dredging of an overdepth, it is dangerous to just increase the slope height when the soil is susceptible to liquefaction, because of the strong sensitivity to the slope height. A flatter toe of the slope can improve stability and creates a more natural morphological profile.
- Slopes with steeper upper parts and flatter lower parts can be equally stable as constant slopes, while the required navigation depth is reached earlier. The necessary width can also be reduced, although the total dredging volume can increase. But unfortunately, very large reductions of dredging volumes are not to be expected (less than 5 %), but adaptation to a more morphological profile is very well possible without giving concessions to slope stability or dredging costs.

Soil porosity and the dielectric constant

Although the susceptibility to liquefaction is very sensitive to the maximum dilatant volume strain and especially the porosity, it appeared that in dredging practice often very little is known about these parameters.

From conversations with geotechnical engineers it became clear that they fear a limited applicability of the research results. Therefore some supplementary research has been done on possible correlations between porosity and other soil properties. It seems that the most reliable method of soil investigation related to porosity deals with the electric conductivity. Until recently the suggested formulations were rather complex and not very useful for practical purposes. Kaya [2002] deduced a simple formulation, based on his own test results and independent data from other tests on a wide range of soils. For any soil-water mixture this formulation is:

$$n = 0,0136\varepsilon + 0,02 \quad \text{Eq. 4-17}$$

in which

ε = the dielectric constant [-]

The dielectric constant is the relative dielectric permittivity of a certain medium (here: soil-water mixture) and thus a measure of the relative ability of a material to store charge for a given applied electric field. It can be determined from capacitance measurements, in which the storage of charge in a certain medium between two electrically charged plates is measured. Such an instrument can be employed in Cone Penetration Tests. The dielectric constant can be obtained from the measured capacitance 'C' as follows:

$$\varepsilon = \frac{Cd}{\varepsilon_0 A} \quad \text{Eq. 4-18}$$

in which

C = capacitance

d = distance between the plates

ε_0 = dielectric permittivity of vacuum = $8,854 \cdot 10^{-12}$ F/m

A = surface of the plates

[F] or [C/V]

[m]

[F/m] pr [C/Vm]

[m²]

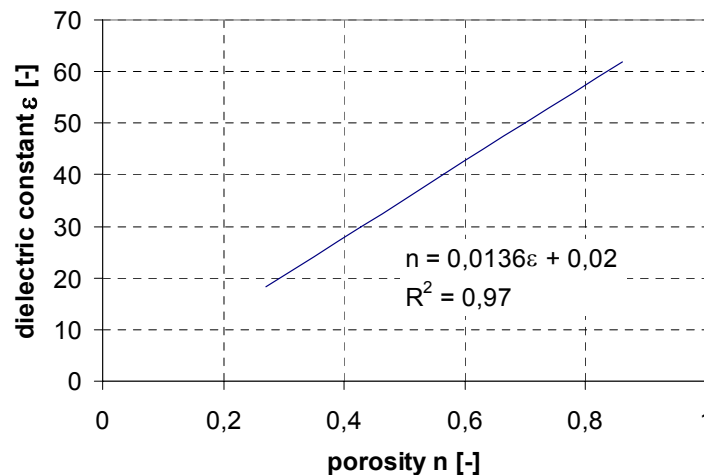


Figure 4-21: Dielectric constant against porosity [source: Kaya, 2002]

Formulation 4-13 and Figure 4-21 are valid at low MHz frequency range (13-50 MHz) and regression analysis showed good agreement with test results: $R^2 = 0,97$.

4.2.3 Soil composites

The liquefaction behaviour is extremely sensitive to the amount of other soil contents. Most theoretical research, just as this thesis, is done on homogeneous soils, while small amounts of contaminants can have a significant impact on the strength of the soil. Two well-known mixtures will be mentioned briefly.

Silty sand

The effect of fines like silts in sand has often been studied, with very contradictory results. Some researchers (Seed 1983, Pitman 1994) state that the liquefaction potential is reduced by the presence of fines; others (Lade et al. 1997) state the opposite. Most important difference between the research studies is the relative density of all the tests. Some maintain the same void ratio of the original sand, filling the voids with fines and thereby increasing the relative density, others maintain this absolute density or use an equal natural settling process, which in my opinion is the most useful method when considering submarine slopes. When submarine slopes are susceptible to liquefaction, the sediment is often deposited under calm marine conditions. Therefore only the results of Lade and Yamamuro (1997) will be briefly presented and explained. Four different sands (Nevada 74-300 μm , Nevada 175-300 μm , Ottawa 74-300 μm and Ottawa 74-250 μm) with increasing non-plastic fine contents (14-74 μm) were investigated. It appeared that a so-called 'dry funnel deposition method' yielded homogeneous soils which were very well comparable to alluvial or marine sedimentation ($n = 0,40-0,46$). The tests were done at very low confining pressures (25 kPa), because then static liquefaction is most prevalent. These low pressures are typical for sea beds. Their most important conclusion was:

All sands contaminated with fines are less resistant against liquefaction than non-contaminated sand, even though their relative and absolute densities increase.

This conclusion is inconsistent with normal soil behaviour, because soil should show a more dilatant behaviour at higher densities. So apparently void ratio and relative density are no good criteria when considering the liquefaction potential of soil mixtures.

The explanation of this apparently inconsistent soil behaviour is based on the following hypothesis: "in silty sands of low to moderate densities, a particle structure can develop in the soil between the larger and smaller grains resulting in high volumetric compressibility, which in turn results in static liquefaction at low pressures".

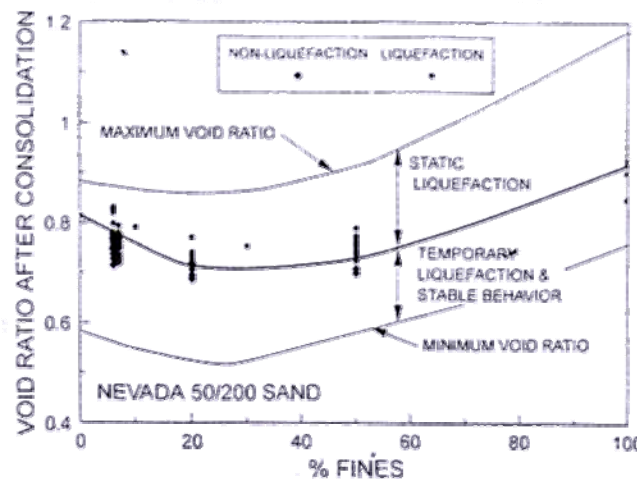


Figure 4-22: Maximum and minimum void ratio and the 'static liquefaction envelope' against percentage of fines [Lade et al. 1997].

The test results are demonstrated in Figure 4-22 for Nevada 50/200 sand ($d=74-300\mu\text{m}$), which appeared to be susceptible to liquefaction for all fine contents, unlike the other sands that appeared to be not susceptible to liquefaction for very small fine contents. The maximum and minimum void ratio are plotted for different fine contents. The line of the minimum void ratio, which is obtained when the soil is energetically densified, shows that a small amount of fines increases the density by filling the void spaces between the grains, leaving the larger grains in full contact with each other. By enlarging the content of fines, the larger grains lose contact, thus reducing the density. It appears that a fine content of approximately 20-30% is most effective in just filling the voids, without pushing the larger grains from each other.

The line of the maximum void ratio is obtained by deposition of the soil under minimum input of energy. During this deposition some, but not all fines will settle just in the voids, causing a slightly smaller dip in this line, compared to the minimum void ratio. With larger amount of fines, more fines start to occupy locations between the contact points of the larger grains, thus reducing the relative density.

All test results are also plotted and the static liquefaction envelope was determined. It can be seen that this line slowly shifts towards the minimum void ratio, indicating that with increasing fine content, soils with higher relative densities are still susceptible to static liquefaction, even up to relative densities of 60% for fines contents over 50%. Although relative densities may increase at larger contents of fines, the overall behaviour is not much affected, because during shearing, these fines will roll loosely around in the voids between the larger grains. When the percentage of fines is further enlarged, the original void spaces no longer exist and the fines start to dominate the soil behaviour.

The soil behaviour during loading is schematized in Figure 4-23; the silt grains between the larger sand grains will move towards the voids when compressed or sheared, increasing contractant tendencies. Only if the shearing continues and the sand grains come into better contact with each other, the soil behaviour can become dilatant, but only for small percentages of fines.

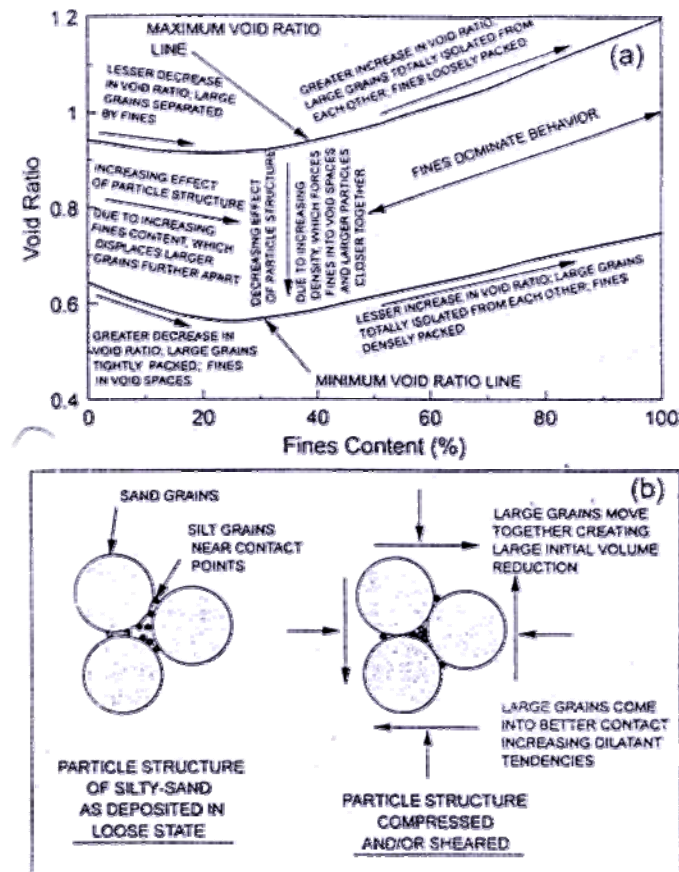


Figure 4-23: Schematization of development of void ratio with increasing fines content

As was mentioned before many research studies on actual earthquake-induced liquefaction events state that sands with a significant fines content should be more resistant against cyclic liquefaction. It is however more likely that this apparent resistance is caused by effects of stress history (overconsolidation) and aging (creep or cementation); both effects are not significant in natural submarine slopes (or hydraulic fills), which are characterized by low effective stresses and no overconsolidation. Sediment is continually and slowly added to the slope, which tend to reduce the effect of creep. And after all, most reported shallow submarine slope failures are situated in silty sands.

Sand-gravel composites

Evans et al. [1995] investigated the liquefaction resistance of sand-gravel composites. Undrained cyclic triaxial tests were performed on sand-gravel composites with gravel contents (GC) of 0%, 20%, 40% and 60%, all with a relative density of the composite of 40%. They found that maximum and minimum densities of sand-gravel composite sediments increased significantly with increasing gravel content and reached peak values at a gravel content of about 60%. This observation is in line with Lade and Yamamuro [1997], when inversely reasoning: they found peak values for a fines content of about 30%, so content of the coarser material (sand) of about 70%.

The liquefaction resistance of sand-gravel composites may increase considerably with increasing gravel content. Or more precisely, the development of the pore pressure and axial strain of the first two composites (GC=0% and 20%) under cyclic loading showed classic liquefaction behaviour of moderately loose sand; the other composites (GC=40% and 60%) showed more classic cyclic mobility behaviour of dense sand, see Figure 4-24 and Figure 4-25.

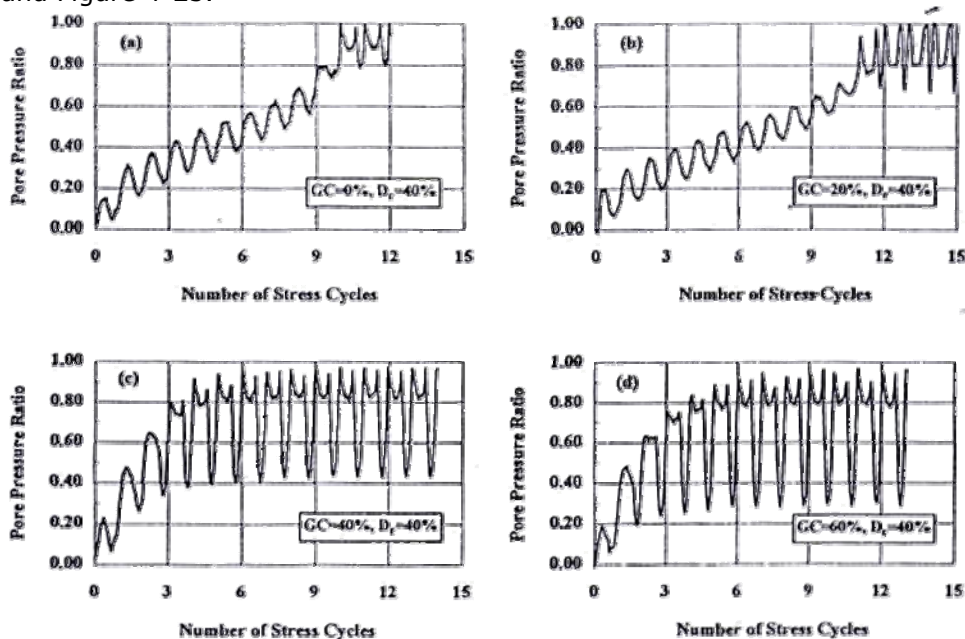


Figure 4-24: Pore pressure response of sand-gravel composite specimens with gravel contents of (a) 0%, (b) 20%, (c) 40% and (d) 60%

Samples (a) and (b) show hardly no axial strain response during the first 10 stress cycles, but then suddenly a large response occurs, indicating failure. The pore pressure ratio becomes 1 and the grains loose contact. Before failure occurs, the amplitude of the pore pressure ratio remains small, whereas samples (c) and (d) can undergo far larger CSR's according to the larger amplitudes of the pore pressure ratio. The axial strain slowly increases, but is present from the beginning of loading; no 'explosive' behaviour is observed.

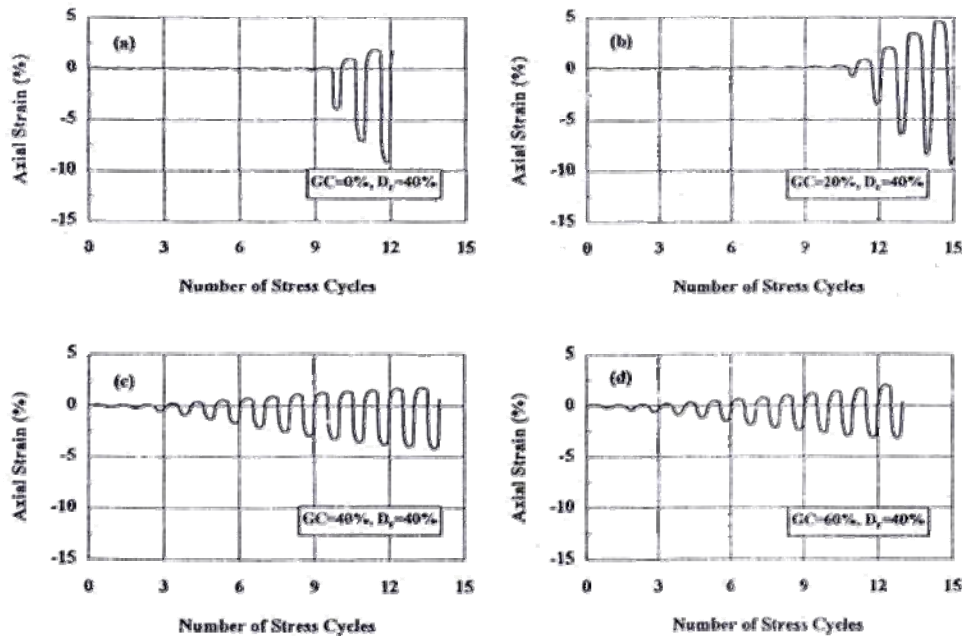


Figure 4-25: Axial strain response of sand-gravel composite specimens with gravel contents of (a) 0%, (b) 20%, (c) 40% and (d) 60%

Evans et al. [1995] also concluded that the relative density isn't a good measure to indicate liquefaction potential. When the gravel content is increased and the composite relative density is kept constant, the relative density of the sand matrix is reduced. For example, when the grain content is 60%, the relative density of the sand matrix is 0% (at the loosest packing). However, the resistance against liquefaction increases. Therefore a comparison was done between a clean sand ($RD = 40\%$) and a sand-gravel composite ($GC=40\%$, $RD_{\text{sand}} = 40\%$, so $RD_{\text{composite}} = 52\%$). It appeared that the cyclic stress ratio to cause 5% double amplitude strain in 10 load cycles ($CSR_{5\%,10}$) increased from 0,15 to 0,36 for respectively the clean sand and the composite and the behaviour transformed from loosely packed to densely packed.

Another test was done to find a clean sand equally stable as a sand-gravel composite ($GC=40\%$, $RD_{\text{composite}} = 40\%$, so $RD_{\text{sand}} = 29\%$). It appeared that a RD of 66% was needed to obtain an equally stable clean sand, again proving that the addition of gravel reduces the liquefaction potential. This test also proved that cyclic loading tests on sand-gravel composites can be estimated by tests on clean sands at a representative density.

Summarizing, both research studies prove that smaller particles between larger particles can significantly increase liquefaction potential. An approach based on the relative density only is not very meaningful. It should also be noted that soil tests on homogeneous soils as well as on composites of two fractions are a strong simplification of nature. A very widely graded soil can perform much better, because the unfavourable particle structure of Figure 4-23 will not develop, because all fractions in between are present.

4.3 Retrogressive breaching

Opposite of liquefaction slope failures, breach failures occur at medium to densely (hexagonally) packed fine sand and this failure type is restricted to very steep submarine slopes. Thin superficial layers of sediment show dilatant behaviour, because, when subjected to shear deformation, its volume expands, causing a negative pore pressure with respect to the hydrostatic pressure, resulting in increased shear resistance. This situation of underpressured sand is able to maintain a steep (steeper than the angle of repose) slope, until pore water flows in and the slope will gradually fail.

The necessary precondition for the initiation of a breach failure is a local steep slope disturbance, such as that produced by dredging activities or scour by channel flow. In conditions met in freshly deposited sand the difference between dilatant and contractant behaviour is often very small. Therefore retrogressive breaching is frequently mistaken for a liquefaction flow slide. A breach failure could also originate from the scar produced by a small liquefaction slide; in that case both failure mechanisms may be present.

Two types of breaches can be distinguished, initiated by a supercritical ($Fr > 1$) or subcritical ($Fr < 1$) flow. The first type is found in series of bed geometries formed at high Froude numbers ($Fr = u/\sqrt{gh}$) that can be considered as an adaptation of chute-and-pool bedforms to conditions in fine sand and silt. Large breaches of this type occur at the bursting of an embankment (Visser, 1998). The second type of breaches can be generated incidentally on steep, perhaps up to more than 5 m high submerged slopes on bars or navigation channel banks. The interest is in the second type, which is often mistaken for a liquefaction flow slide. The difference lies in the time span: a breach retrogrades slowly and may be active for a large number of hours, producing a quasi-steady turbidity current. Breach failure can occur when the top of the slope is steeper than the angle-of-repose ϕ .

The other division in types of breaches is in 'controlled' and 'uncontrolled' breaching. In dredging practice one speaks of controlled breaching. By increasing or decreasing production rates a controlled breach can be maintained with a more or less constant wall velocity. This wall velocity is defined as the retrograding velocity of the breach and has an empirical relation to the hydraulic permeability ($V_{wall} \approx 25k$). For the reference sands under consideration this yields values in the order of 1 mm/s. When breaches become higher and steeper, this grain-by-grain failure turns into larger failure in slices. Breaches can also become self-maintaining and continue after the dredging is finished. This slow and gradual failure mechanism of breaching is rather different from the faster and intermittent liquefaction flow slides.

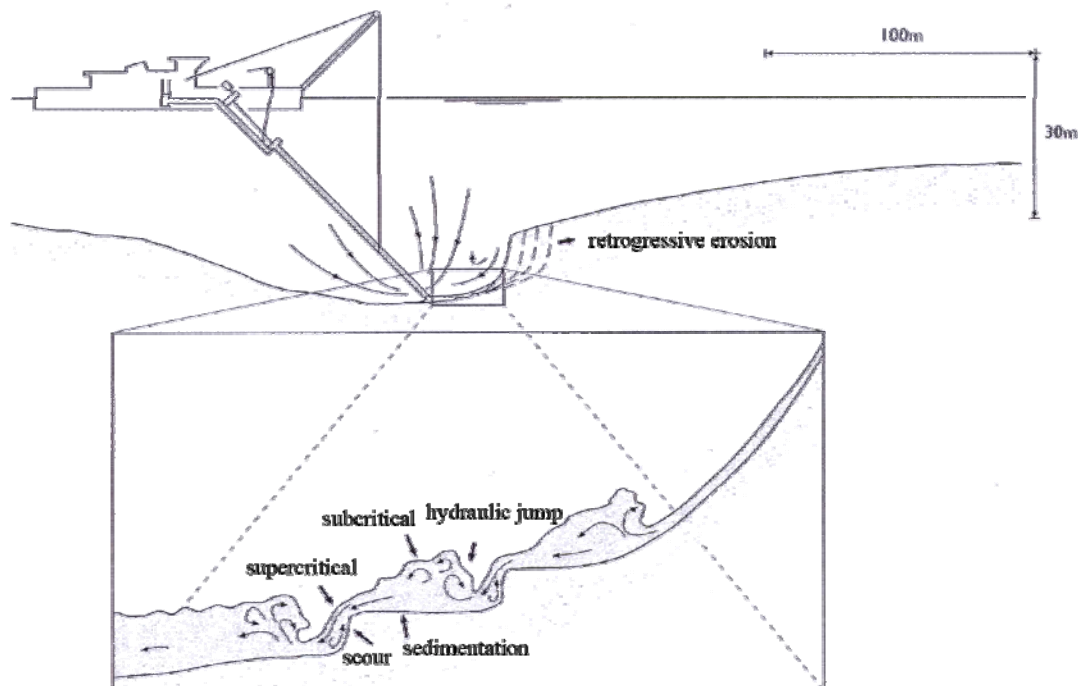


Figure 4-26: Flow from an active breach to a suction entrance with hydraulic jumps, subcritical flow with sedimentation and supercritical flow with scour

In Overijssel research is done by Delft Cluster (GeoDelft and Delft Hydraulics) on sand borrowing pits [WL|Delft Hydraulics, 2001, Mastbergen, 2002]. The growing demand on sand resulted in the question whether existing pits could be deepened without risking slopes instability. The former regulations of the Province of Overijssel appeared to be too conservative and too general. The new approach takes grain diameters and depth of the pit into account.

For illustration a graph is presented in which the required distance from the border to the toe of the slope is drawn for a certain depth for two different grain diameters, see Figure 4-27. Dredging 'under the concession line' is not allowed, even if it means that the total amount of available sand cannot be extracted.

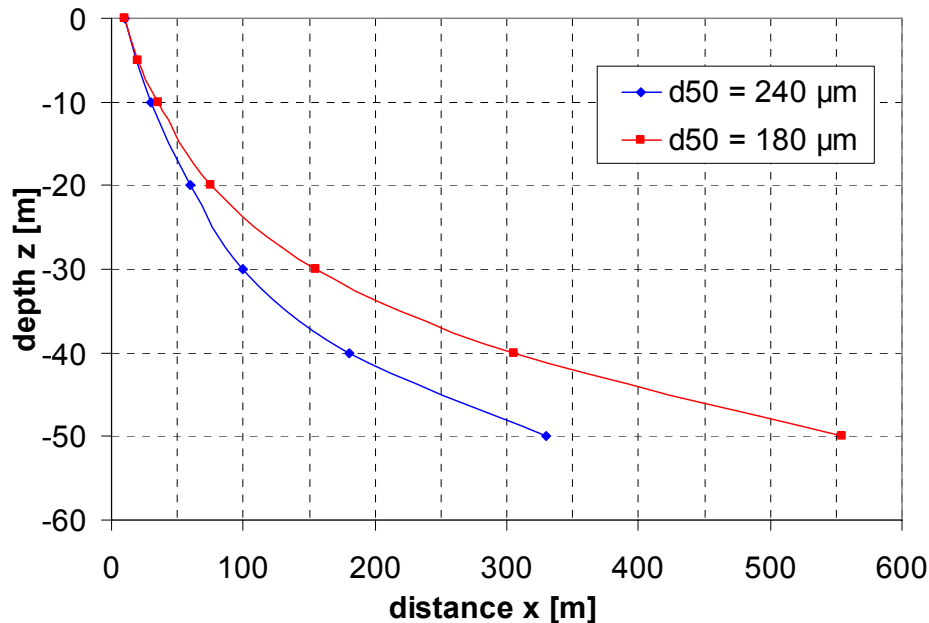


Figure 4-27: Concession line for sand borrowing pits in the Province of Overijssel for two different grain sizes [source: WL|Delft Hydraulics, 2001]

It is noticeable that coarser sand allows for steeper slopes and that with increasing depth of the sand borrowing pit, the maximum slope angle reduces. Furthermore, it can be concluded that slopes gentler than 1:3 and not higher than 20 m are unconditionally stable with respect to breaching.

Unlike controlled breaching, uncontrolled breaching is a natural process which is still not commonly known as an instability mechanism (Van den Berg 2002, Mastbergen 2003). This breaching process runs autonomously and can take several hours, in contrast with the suddenly occurring flow slides. An initially steep slope is a prerequisite for uncontrolled breaching, so this mechanism is not very likely in channels, because of the limited slope height (about 10 m) and the relatively flat (flatter than the angle of repose) slopes which will remain after dredging. This mechanism however can be of major importance in large submarine canyons.

4.4 Micro-instability on an infinite slope

In this paragraph the stability of the outer grains of the slope will be discussed. The slip circle analysis in paragraph already proved that in non-cohesive, homogeneous soils micro-stability is the normative failure mechanism. This resulted in slip circles with large radii that only slightly touched the slope surface. In this paragraph the unloaded slope will be considered. Common loads that affect micro-stability are seepage and/or pressure gradients, like for instance ground water flow through dikes between different water levels or water pressure gradients after long-term high water in rivers or short-term wave attack (wave troughs cause an overpressure inside the slope). Please note that these wave forces are not to be confused with the erosive or stirring forces causing instability of the outer grains, see Chapter 7 (threshold of motion) and 8 (sediment transport).

The upper boundary of the slope angle (steepest possible slope), when it comes to micro-stability, will be found when considering an infinite long slope without any additional forces like seepage flow.

In Paragraph 4.1.2 a vertical wall of cohesive soil was considered; now an infinite slope in non-cohesive soil without any currents and waves is considered. If we define a coordinate system with the ξ -axis along the slope (positive upward) and the η -axis perpendicular to the slope (positive downward), we obtain the following two equations of equilibrium parallel and perpendicular to the slope:

$$\begin{aligned} \frac{\partial \sigma'_{\xi\xi}}{\partial \xi} + \frac{\partial \sigma'_{\eta\xi}}{\partial \eta} + \frac{\partial p}{\partial \xi} + \gamma_{sat} \sin \beta &= 0 \\ \frac{\partial \sigma'_{\xi\eta}}{\partial \xi} + \frac{\partial \sigma'_{\eta\eta}}{\partial \eta} + \frac{\partial p}{\partial \eta} - \gamma_{sat} \cos \beta &= 0 \end{aligned} \quad \text{Eq. 4-19}$$

When the groundwater is in rest, the water pressure can be assumed to be hydrostatic:

$$p = p_0 - \gamma_w z = p_0 + \gamma_w \eta \cos \beta - \gamma_w \xi \sin \beta \quad \text{Eq. 4-20}$$

If the water level is assumed to be infinitely high, because the slope is assumed to be situated completely below water level and the slope is infinitely long:

$$p_0 \rightarrow \infty \Rightarrow \frac{\partial p_0}{\partial \xi}, \frac{\partial p_0}{\partial \eta} \rightarrow 0 \Rightarrow \frac{\partial p}{\partial \xi} = -\gamma_w \sin \beta, \frac{\partial p}{\partial \eta} = \gamma_w \cos \beta \quad \text{Eq. 4-21}$$

This assumption does not fulfil reality, but this approach is useful to determine the upper limit in case of very long, deep slopes. Substituting Eq. 4-19 into Eq. 4-17 yields:

$$\begin{aligned} \frac{\partial \sigma'_{\xi\xi}}{\partial \xi} + \frac{\partial \sigma'_{\eta\xi}}{\partial \eta} + (\gamma_{sat} - \gamma_w) \sin \beta &= 0 \\ \frac{\partial \sigma'_{\xi\eta}}{\partial \xi} + \frac{\partial \sigma'_{\eta\eta}}{\partial \eta} - (\gamma_{sat} - \gamma_w) \cos \beta &= 0 \end{aligned} \quad \text{Eq. 4-22}$$

On the assumption that the stresses are independent on the coordinate along the slope (after all, the origin may be chosen anywhere along an infinite slope), these equations can easily be integrated. The constants of integration must equal zero, because the effective grain stresses equal zero at the slope surface:

$$\begin{aligned} \sigma'_{\eta\xi} &= -(\gamma_{sat} - \gamma_w) \eta \sin \beta \\ \sigma'_{\eta\eta} &= +(\gamma_{sat} - \gamma_w) \eta \cos \beta \end{aligned} \quad \text{Eq. 4-23}$$

If these two equations are combined with the Coulomb failure criterion for non-cohesive soils, it gives:

$$\left| \sigma'_{\eta\xi} / \sigma'_{\eta\eta} \right| < \left| \sigma'_{\eta\xi} / \sigma'_{\eta\eta} \right|_{\max} \Rightarrow \tan \beta < \tan \phi \quad \text{Eq. 4-24}$$

The above equation can of course be expressed in the stability or safety factor F:

$$SF = \frac{\tan \beta}{\tan \phi} \quad \text{Eq. 4-25}$$

As was concluded in Paragraph 4.1, the unit weight of a non-cohesive soil does not affect micro-stability in absence of groundwater flow. In absence of seepage a submarine slope can be as steep as a slope above the water level. In reality, additional forces occur, like waves.

5 Wave Loads

5.1 Direct Elastic Behaviour

5.1.1 Load or reduction of strength?

Until now, attention is only paid to unloaded conditions. Loads were only treated as a sudden trigger mechanism. Of course, in practice some loads will occur. One can think of loads from constructions, propellers of manoeuvring ships, earthquakes and of course waves. In this thesis only wave loads are considered extensively, because waves are common all over the world. Earthquakes however are also notorious for their destructive capacities. These and other loads are described briefly in Paragraph 5.3.

Bottom pressure pulses due to ocean waves can be a major factor in initiating submarine landslides. These waves produce pressure changes within the water below the surface and pressure pulses on the surface of the sediment. Because of this difference in pressure under crest and trough, the passage of a wave induces shear stress in the soil. As the wave passes, soil at a particular point experiences cyclic fluctuation in magnitude and direction of these induced stresses.

At this point it is very important to distinguish between the *direct, elastic* effect of waves and the *indirect, plastic* effect. In this paragraph the direct effect is treated, which causes a fluctuation of water pressures and effective stresses, but is assumed to have no net effect: these transient stresses dissipate with each wave cycle. On the other hand this fluctuation of effective stresses in the soil causes a tendency to compact, resulting in gradual water overpressure (indirect effect). This overpressure can cause liquefaction, which will be investigated in the next paragraph.

To investigate the problem of direct wave effects often many extensive simplifications are assumed, which will be explained further on.

Often the rule of thumb is used that excess pore pressures (EPP's) become negligible when the depth of the water d becomes greater than half of the wavelength. Because of the maximum depth of interest of 20m, excess pore pressures will always occur, because wavelengths are easily larger than 40m.

The loss of stability under wave action is analyzed on the concept that failure is gravity controlled, but that wave action can reduce the (undrained) strength of the soil. So in fact waves do not act as loads, but more as a reduction of strength. The title of this chapter in fact is wrongly chosen, when looking at submarine slopes, which are completely situated below sea level. In the breaker zone wave forces can damage revetments and can be spoken of 'loads'.

5.1.2 Water pressures below sea level

There are several classical theories that describe the subsurface pressure fluctuations at a given point due to the passage of a surface wave. Most researchers have used the linear wave theory, but also second order Stokes' wave theory and even higher order wave theories can be applied. Which theory has to be used is of course dependent on the wave characteristics: wave height H , wave period T and the depth of the water h . These characteristics can be described by two well-known parameters H/gT^2 and h/gT^2 (LeMéhauté, 1976). The first one is a measure for the wave steepness (H/L), the second one for the relative water depth (h/L).

In this thesis wave periods in the order of 5-15 s and wave heights in the order of 2,5-10 m are dealt with in water depths varying from 5 to 20 m. Because we are interested in the macro-stability of a slope, we look at the largest possible waves. Smaller, more frequent waves are important when considering sediment transport which will be treated later on.

Unlike smaller waves, which can be approximated by the linear theory, depth-limited waves with a wave height of about half the water depth are in the 'cnoidal theory regime'. This means that the linear theory as well as the second order Stokes' theory do not apply and the more complex cnoidal formulations described by elliptic integrals for wave pressures should be used, see Table 5-1.

Table 5-1: Application of the appropriate wave theory

		T [s]											
		5		7		9		11		13		15	
h [m]	H [m]	h/gT ²	H/gT ²	h/gT ²	H/gT ²	h/gT ²	H/gT ²	h/gT ²	H/gT ²	h/gT ²	H/gT ²	h/gT ²	H/gT ²
5	2,5	0,020	0,010	0,010	0,005	0,0062	0,0031	0,0041	0,0021	0,0030	0,0015	0,0022	0,0011
		cnoidal		cnoidal		cnoidal		cnoidal		cnoidal		cnoidal	
10	5	0,040	0,020	0,020	0,010	0,0123	0,0062	0,0083	0,0041	0,0059	0,0030	0,0044	0,0022
		breaking		cnoidal		cnoidal		cnoidal		cnoidal		cnoidal	
20	10	0,080	0,040	0,041	0,020	0,0247	0,0123	0,0165	0,0083	0,0118	0,0059	0,0089	0,0044
		breaking		breaking		cnoidal		cnoidal		cnoidal		cnoidal	

Most research studies however only consider linear and sometimes second order waves, while especially in the upper layer of the water depth cnoidal theory shows different behaviour. However, when approaching the bed, the difference in wave pressures between cnoidal and linear wave theory decreases and an approximation according to the linear wave theory is therefore considered sufficiently accurate:

$$p_{\text{wave}} = \frac{\rho_w g H}{2} \frac{\cosh kz}{\cosh kh} \cos(kx - \omega t) \quad \text{Eq. 5-1}$$

For an impermeable rigid flat bed the amplitude of the wave pressure at the bottom ($z=0$) can now be expressed as:

$$\hat{p}_{\text{wave}; z=0} = p_0 = \frac{\rho_w g H}{2 \cosh kh} \quad \text{Eq. 5-2}$$

The assumption of a flat bed may be too favourable; in 1977 Mallard and Dalrymple analyzed the effects of a deformable seafloor on the water pressures. They found that these pressures are up to 15% higher for very soft cohesive sediments, but may be ignored for most sands.

Liu (1973) cast doubt on the assumption of impermeability of the seabed and investigated a permeable, rigid flat bottom of infinite thickness. This resulted in a reconsidered expression for p_0 :

$$p_0 = \frac{\rho_w g H}{2} \frac{1}{\cosh kh - \frac{K}{T\nu} \sinh kh} \quad \text{Eq. 5-3}$$

Considering the reference soils, the second term in the denominator is only significant if the intrinsic permeability κ is in the order of $T\nu$ ($\approx 10^{-5}$), which is the case for very rough gravel (e.g. filter layers); sand, silt and clay are less permeable for short waves. Neglecting this second term introduces an error of at most 0,002 %.

It seems inconsistent that the wave period T is in the denominator; after all, the seabed should be more permeable, if the wave period increases, but with increasing T , the wavelength L also increases and the horizontal wave particle velocity over-all increases. Therefore, longer waves experience a less permeable seabed.

Tsui et al. [1983] compared these theoretical computations with measured values from experiments. First they investigated wave pressures (linear and second order wave climate) on a completely impermeable, rigid and horizontal concrete bottom. The predicted pressures lie within a confidence band of 0,02 kPa, while the measured pressures varied from 0,069 to 0,207 kPa.

Then they placed a layer of Chattahoochee sand on the concrete bottom: two different porosities were considered, a loose ($n = 0,52$) and dense ($n=0,38$) packing as well as two different layer thicknesses 0,23 m and 0,33 m. According to Liu, the bottom could still be considered 'impermeable'. It appeared that indeed the wave pressures on top of the dense sediment showed good resemblance to theory. However with increasing thickness of the sediment layer and with increasing wave periods, the measured values of the wave pressures became significantly smaller than the theoretical values.

Tsui et al. introduced the effective water depth h' . This effective water depth h' is larger than the real water depth h , but is chosen such that the measured wave pressure at depth h is the same as the theoretical value at depth h' , see Figure 5-1 in which symbol ' d ' is used for the water depth.

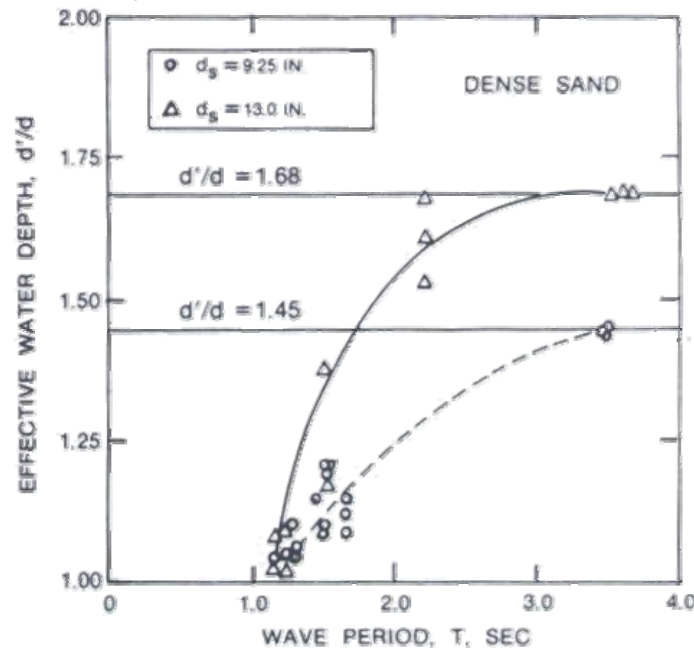


Figure 5-1: Ratio of effective water depth (d') and real water depth (d) for two sediment layers ($d_s = 0,23$ m and $d_s = 0,33$ m) on an impermeable layer [source: Tsui et al., 1983]

This effective water depth was found to increase with wave period T and thickness of the sediment layer d_s . Unlike Liu, they concluded that long period waves extend into the entire thickness of the sand layer. The effective depth then equals the sediment thickness and the wave-induced pressure on top of the sediment layer is smaller than the theoretical value. The waves under consideration in this thesis, especially the larger storm waves, have wave characteristics similar to the waves in these tests. So, the fact that the seabed consists of permeable sediment instead of an impermeable layer, makes it very likely that the actual water pressures at the sediment surface will be somewhat lower than obtained from expression 5-3. This phenomenon however isn't widely adopted and still no analytical expression based on the soil stratification and wave characteristics exists. It is therefore very hard to calculate with effective depths and the expression of 5-3 will be considered as a safe upper boundary.

5.1.3 Water pressures inside seabed

Now these water pressures on top of the bed have to be translated into pore-water pressures inside the bed, because they affect the resulting effective stresses in the soil. Theoretically, both transient and residual pore-water pressures are generated in the seafloor by wave loading. The transient pressures instantaneously result from the travelling wave; the residual pressures are caused by the variation of dynamic wave pressures in time and space. These pressures aren't instantaneously related to the wave loads, but depend on the intensity and duration of wave loading and the drainage characteristics of the seafloor, see Paragraph 5.2.

Putnam (1949), who assumes the sand skeleton and the water to be incompressible and hydraulically isotropic and only considers the transient pore-water pressures, gave one of the first expressions. In his work he mainly investigated the wave damping. However, implicitly he assumed the pore-water pressures to decay according to an inverse 'e-power'. Please note that the positive direction of the z-axis is upward:

$$\hat{p}_{\text{wave;bed}} = p_0 \frac{\cosh k(d_s + z)}{\cosh kd_s} = p_0 \frac{[e^{+kz} + e^{-k(2d_s+z)}]}{[1 + e^{-2kd_s}]} \quad \text{Eq. 5-4}$$

in which z is the depth into the soil layer, k is the wave number ($k=2\pi/L$) and d_s is the thickness of the seabed on a rigid impermeable layer. Sleath extended this approach and included hydraulic anisotropy. With k_x and k_z as the hydraulic permeabilities in horizontal and vertical direction, the above equation changes into:

$$\hat{p}_{\text{wave;bed}} = p_0 \frac{\cosh \left(k \left(\frac{k_x}{k_z} \right)^{1/2} (d_s + z) \right)}{\cosh \left(k \left(\frac{k_x}{k_z} \right)^{1/2} d_s \right)} \quad \text{Eq. 5-5}$$

This equation has been validated in wave tank tests and despite a lot of scatter, the experimental results are in line with theory, at least for sandy bottoms.

Most research studies are based on the uncoupled analysis: the coupling of the sand skeleton and pore water in resisting the waves is ignored and the mechanical properties of the sediment are neglected. Biot (1944) presented a coupled theory for a poroelastic solid which accounts for elastic deformation of the porous medium, compressibility of the pore fluid and Darcy flow (consolidation however is not accounted for). This theory yields a relatively simple solution for a seabed of infinite thickness which is hydraulically isotropic and under the assumption that the stiffness of water is much larger than the stiffness of the sand skeleton. The amplitude of the wave pressure inside the seabed:

$$\hat{p}_{\text{wave;bed}} = p_0 e^{+kz} \quad \text{Eq. 5-6}$$

The above equation can also be obtained from Sleath's formula with $k_x = k_z$ and $d_s \rightarrow \infty$. Under natural conditions the horizontal permeability often isn't larger than 1,5 or 2 times the vertical permeability. Please note that here parameter 'k' represents the wave number in the seawater, while 'k' with a subscript represents the hydraulic permeability in the seabed! Normally, in both separate disciplines (fluid and soil mechanics) these parameters do not interfere.

Experiments done by Tsui et al. [1983] confirmed the ideas behind the expressions:

- 1) the pressure ratio p/p_0 decreases with increasing distance z into the sediment layer;
- 2) the pressure ratio p/p_0 is larger in loose sands than in dense sands due to higher permeability;
- 3) the pressure ratio increases with increasing period T.

The above conclusions are qualitatively in line with the theoretical expressions of Putnam and Liu, but there are also some striking differences. The measured values for loose sediment are more in agreement with both theories than the values for dense sediment. Putnam and Liu do not take sediment properties (other than permeability) into account and their formulas seem to be valid only for very permeable soils. Tsui et al. [1983] found an increasing difference between theoretical values and test results for decreasing permeability. So, wave-induced water pressures in loosely packed sand, which happens to be most sensitive to wave action, can well be represented by equation 5-6, but this equation may probably be not so applicable in dense sea beds. Summarizing, the wave-induced pressure on top of the bed is mainly influenced by the thickness of the sediment layer and hardly by permeability. The *attenuation* of wave pressures in the bed is strongly dependent on permeability.

To illustrate the impact of wave-induced fluctuating water pressures on the hydrostatic pressure, a sort of upper boundary is presented. Most unfavourable are shallow waters with high, long waves. The influence on the water pressures can be in the order of 25 % in case of depth-limited waves in shallow water. The value of 25% can theoretically be explained if one considers the ratio between the amplitude of the wave pressure and the hydrostatic water pressure at the surface of the sea bed. If the wavelength L increases to infinity and the wave height is limited to half of the water depth:

$$L \rightarrow \infty \Rightarrow k \rightarrow 0 \wedge H = \frac{1}{2}h : \frac{\text{wave_pressure}}{\text{hydrostatic_pressure}} = \frac{\frac{\rho_w g H}{2 \cosh(-kh)}}{\rho_w g h} \xrightarrow{k \downarrow 0} \frac{1}{4} \quad \text{Eq. 5-7}$$

The above approximation is realistic. For instance, if the water depth equals 5m, the wave height 2,5 m and the wave period 15 s, it means that the wavelength is 105 m and the ratio between wave pressure and hydrostatic pressure already becomes 0,239.

An important additional phenomenon is the occurring time lag. Although the wave pressures in the sediment also have a sinusoidal character, they lag behind the wave pressures on top of the sediment surface. Tsui et al. found that this time lag could grow as big as one-third of the wave period, see Figure 5-2.

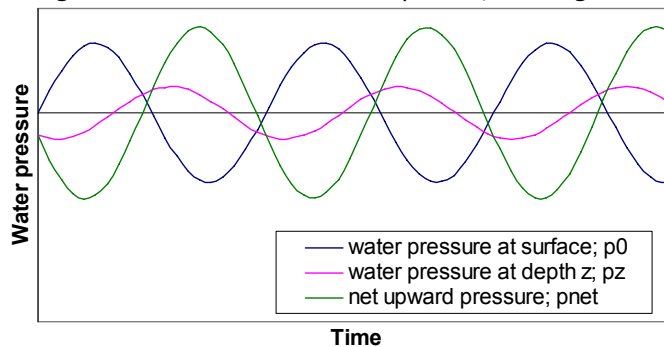


Figure 5-2: Wave-induced net upward pressure caused by time lag

Water pressures on top of the seabed and at a certain depth z inside the bed are plotted for the unfavourable time lag of one third of the wave period. The net upward pressure is represented by the green line and is largest under a wave trough.

Although some researchers warn against failure due to net upward pressures which may occur under a wave trough, the risk may be doubted. The time lag increases with increasing depth, but the effective overburden pressure (weight of the overlying soil) also increases. Demars [1983] considers the effect of this phase shift negligible for a flat seabed in practical purposes. However, in case of very long period waves, caution is recommended, especially because this phenomenon isn't fully understood. The risk of *slope failure* due to net upward pressures is much more likely, especially for steep slopes with micro-instability as normative failure mechanism.

5.1.4 Total and effective stresses inside seabed

Until now the development of cyclic pore water pressures has been discussed, but when considering slope stability problems, effective stresses are more important (cf slip circle analysis). The cyclic component of effective stress in the sea bed can be expressed as:

$$\Delta\sigma' = \Delta\sigma - \Delta p \quad \text{Eq. 5-8}$$

These cyclic effective stresses can be added to geostatic stresses if the seabed is sufficiently rigid (unlike the plastic deformation described in the next paragraph) and equilibrium is maintained. To obtain the total stress change, elastic theory on a homogeneous, isotropic bed can be used. The vertical total stress change becomes:

$$\Delta\sigma_z = p_0(e^{kz} - kze^{kz})\cos(kx) \quad \text{Eq. 5-9}$$

and the horizontal total stress change is expressed by:

$$\Delta\sigma_x = p_0(e^{kz} + kze^{kz})\cos(kx) \quad \text{Eq. 5-10}$$

while the change in shear stress in the x-z-plane becomes:

$$\Delta\tau_{xz} = p_0kze^{kz}\sin(kx) \quad \text{Eq. 5-11}$$

These total stress changes caused by waves are presented in Figure 5-3. They have been normalized by the wave pressure under a wave crest on top of the seabed. The z-axis represents the dimensionless depth '2z/L' (=kz/π), positive upward.

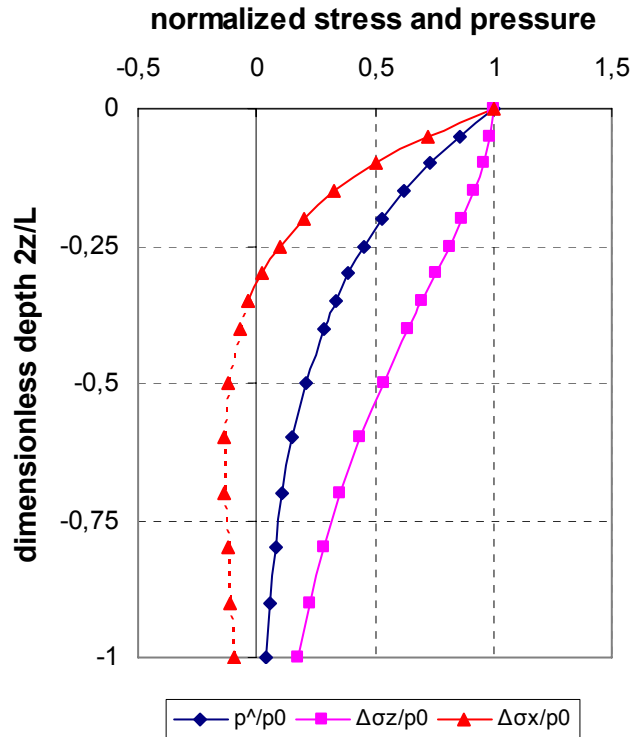


Figure 5-3: normalized sub-bottom stress and pressure profiles due to waves

The effective stress changes can be obtained by subtracting the pore-water pressure changes from the total stress changes. It appears that the horizontal and vertical effective stress changes have the same magnitude but a different sign:

$$\Delta\sigma'_z = -\Delta\sigma'_x = -p_0 k z e^{kz} \cos(kx)$$

Eq. 5-12

Under a wave crest, the vertical total and effective stress reach their maximum value, while under a wave trough the minimum value is reached. Under a wave node there are no changes in horizontal and vertical total and effective stresses. The expressions of the shear stress and effective stress are very similar; only the phase is different. Maximum shear stresses occur under wave nodes.

In Figure 5-4 the vertical and horizontal effective stress changes under a wave crest are presented; horizontal effective stress changes are negative under a wave crest and positive under a wave trough. Furthermore it can be seen, that maximum values occur at a depth of about one-sixth of the wavelength and that these stress changes are about $0,37 \cdot p_0$. It was already mentioned that this fluctuating water pressure p_0 cannot exceed 25% of the hydrostatic pressure, so this change in effective stress does not look very spectacular (less than 10% of the hydrostatic pressure). However, one should note that just below the seabed, effective stresses are still small. This is a first indication that wave-induced failure is a rather superficial phenomenon.

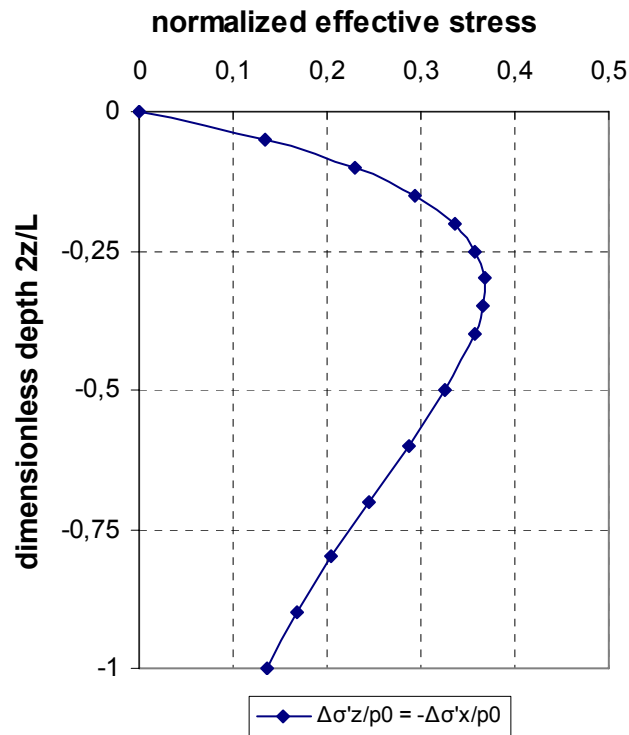


Figure 5-4: normalized effective stress under a wave crest

The effective stress state for any element determines stability. In absence of waves this stress state normally is represented by Mohr's circle, see Figure 5-5. If the bed is completely rigid, the excess pore pressures only generate from bed flow caused by waves and the stress at any moment is the sum of the geostatic stress and wave-induced increment.

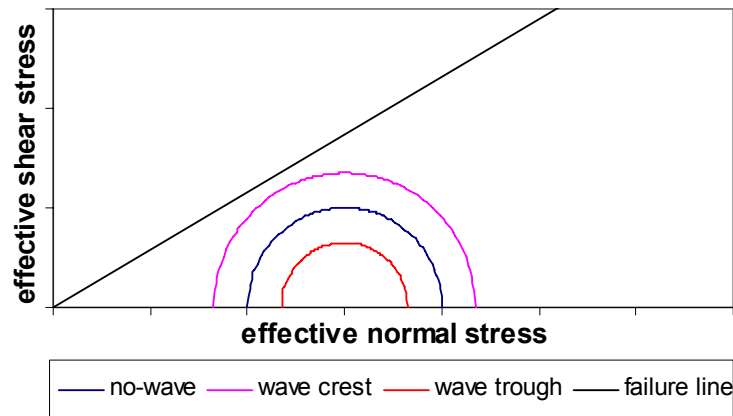


Figure 5-5: Mohr's Circle due to waves

In generally, the effective stress circle for the no-wave condition will increase concentrically in diameter when loaded by a wave crest ('purple' line) and decrease concentrically in diameter when loaded by a wave trough ('red' line), because the cyclic stress is such that $\Delta\sigma_z = -\Delta\sigma_x$. An element is at a state of failure if Mohr's circle for that element contacts the failure line.

Calculation example

Neglecting potential time lag effects, this example will show that failure of a flat seabed due to transient wave stresses is not very likely. Let's consider a seabed at 10 m depth of LFS with depth-limited waves ($H_s = 5$ m, $T = 10$ s, $L = 93$ m). At the seabed, the hydrostatic pressure will be 100 kPa and the cyclic wave pressure (Eq. 5-2) will be 20,2 kPa. The maximum change in effective stress can be approximated by $0,37 \cdot p_0 = 7,5$ kPa, but this maximum only occurs at about 15,5 m below the seabed and at that depth a significant (geostatic) effective stress already is reached.

The cyclic stress ratio (CSR = $\Delta\sigma'_z / \sigma'_z$), which can be obtained by dividing Figure 5-4 by the linear geostatic effective stress, is plotted in Figure 5-6. Please note that at the seabed ($z=0$) this CSR is not defined, because the effective stress is zero at the seabed:

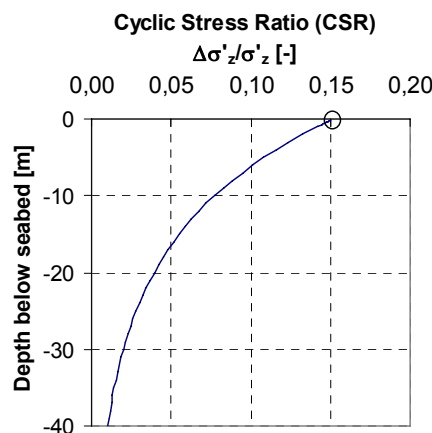


Figure 5-6: Cyclic Stress Ratio against depth below seabed; calculation example

In this example the maximum CSR occurs at the bed surface and is about 0,15. Theoretically, wave-induced bed failure can only occur if this ratio equals 1; this means that under a wave crest, the effective stress is reduced to zero. In practical situations, it can be shown that the CSR will never exceed 0,40 to 0,45. In Figure 5-7 the maximum CSR's for four different water depths h are plotted against wavelength. The solid part of a line represents the wavelengths belonging to wave periods of 5 to 15s according to linear wave theory. Wave heights are depth-limited ($H=0,5h$). So, maximum CSR's occur for larger wavelengths if the water depth increases. Please note that in more shallow water this depth-limited wave will be more frequent, because a larger part of the total incoming wave spectrum will be truncated down.

Unfavourable situations can occur when the saturated unit weight of the bed material decreases or another wave breaking criterion is used, for example the solitary wave breaking criterion ($H=0,78h$).

($\gamma_{\text{sat}} = 16 \text{ kN/m}^3$, $H=0,78h$, $\text{CSR} = 0,43$).

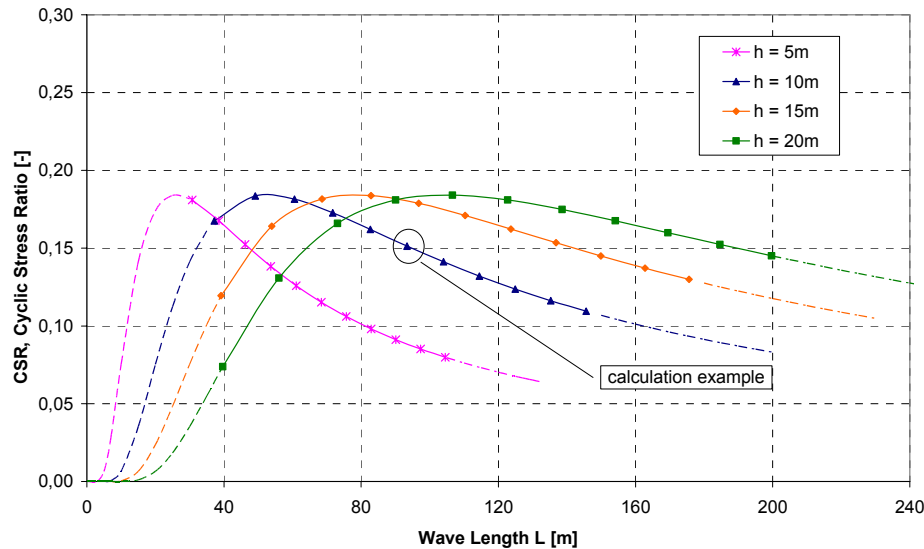


Figure 5-7: Maximum CSR's for different water depths against wavelengths

Demars [1983] showed that failure of the seabed can only occur if the permeability in vertical direction is much smaller than in horizontal direction: $k_x/k_z > 7$ or 8. Typical values for this permeability-ratio in a seabed are 1,5 or 2, so transient seepage failure is not very likely.

5.1.5 Slope stability under transient wave action

Until now, only the consequences of wave loading on water pressures and effective stresses in the seabed have been discussed and it was shown that failure of a seabed in the direct, elastic approach is not expected to occur. It is almost needless to state that slope stability can be drastically reduced.

Macro-stability

Two failure modes are often suggested in literature:

- I. Rotational sliding
- II. Infinite slope sliding

The first one is very similar to the slip circle analysis, described in Paragraph 4.1, although the calculations become much more complicated. Besides a number of slip circles with different radii and centre points, also a number of waves with different wavelength, wave height and location with respect to the slope, have to be considered. These waves affect the effective stresses and shear strengths; driving moments can be increased, resisting forces reduced. Besides iteration with centre point and radius of the slip circle, also the wavelength and position of wave crest, trough and node with respect to the slope have to be iterated. The wavelength, for which the maximum CSR occurs, does not necessarily have to be most unfavourable; it is also dependent on slope height and inclination angle. Because of the diminishing influence of waves with increasing depth, it can be expected that the normative slip circle is less deep and in some way related to the wavelength. Although the volumes that are involved in rotational sliding due to waves can be smaller compared to unloaded shear failure, this does not mean that the consequences are smaller. A sequence of such circular slides can result in progressive down slope movement.

The second failure mode occurs when at some depth the shear stresses exceed the shear strengths and sliding develops on a surface almost parallel to the seabed. This failure mode can be expected at small slope angles in very soft clays/muds, which have a strong stratification and therefore not the subject of this thesis.

Micro-stability

In Paragraph 4.4 submarine micro-stability proved to be dependent on the angle of internal friction only: submarine slopes could be as stable as subaerial slopes. As was mentioned before, the presence of a hydraulic gradient will influence stability. An inward-directed hydraulic gradient, like for instance under a wave crest, will allow for steeper slopes ($\beta > \phi$); an outward-directed hydraulic gradient, like under a wave trough, requires flatter slopes ($\beta < \phi$). In practice, one would never count on positive gradients in slope design, so only outward-directed hydraulic gradients will be considered.

In case of a slope completely situated below sea level, seepage flow will always be directed perpendicular to the slope, if the seabed is an equipotential line. Van Rhee and Bezuijen [1992] investigated two theories for the stability of the outer grains:

- continuum mode
- single-particle mode

The former theory is based on equilibrium of forces on a certain unit volume of soil at the outer slope and is most widely adopted. The latter theory is based on the stability of a single grain: the rolling criterion, determined by the total moment on the grain, controls stability. For outward flow their test results showed that the continuum mode determines stability, while the single-particle mode was normative in case of inward flow. So only the continuum theory will be presented. Because micro-stability is the normative failure mechanism for non-cohesive soils, only these soils will be under investigation ($c=0$).

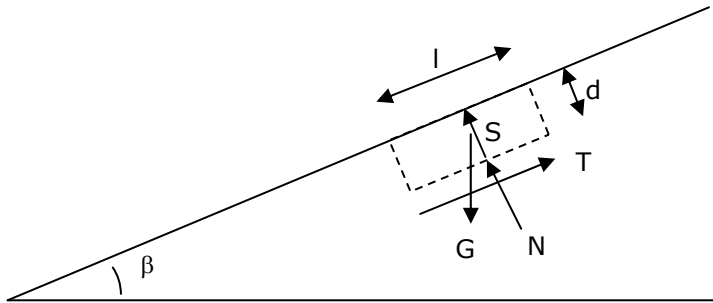


Figure 5-8: Schematization of forces

The small rectangular unit weight (Figure 5-8) is just stable, if a well-known friction law is fulfilled:

$$T = N \tan \phi \quad \text{Eq. 5-13}$$

Forces T and N can be deduced from equilibrium of forces parallel and perpendicular to the slope:

$$\begin{aligned} T &= G \sin \beta \\ N &= G \cos \beta - S \end{aligned} \quad \text{Eq. 5-14}$$

Force G is the weight under water and can be described as follows:

$$G = (\rho_{sat} - \rho_w) g l d = (1 - n)(\rho_s - \rho_w) g l d \quad \text{Eq. 5-15}$$

The flow force S is calculated according to Darcy's Law ($i = 1/\rho_w g \delta p / \delta x$):

$$S = i \rho_w g l d \quad \text{Eq. 5-16}$$

The hydraulic gradient is positive for outward-directed flow. Combining equations 5-13, 5-14, 5-15 and 5-16 yields an expression for the maximum hydraulic gradient:

$$i_{\max} = (1-n)\Delta \frac{\cos \beta \tan \phi - \sin \beta}{\tan \phi} = (1-n)\Delta \frac{\sin(\phi - \beta)}{\sin(\phi)} \quad \text{Eq. 5-17}$$

The maximum hydraulic gradients ($i_{\max}$) are plotted in Figure 5-9 for all non-cohesive reference soils. This hydraulic gradient is a rather trivial quantity when dealing with different piezometric levels, for instance during ebb in the tidal zone. The problem becomes a little more complicated in presence of transient waves: the hydraulic gradient will change in time and alter direction. The most vulnerable part of a slope to wave loads will be the top of the slope (unlike for example static liquefaction), because wave pressures diminish rapidly with depth. The hydraulic gradient due to waves will be approximated by derivation of expression 5-1.

$$i = \frac{1}{\rho_w g} \frac{\partial p_0}{\partial x} = -\frac{Hk}{2 \cosh kh} \sin(kx - \omega t) \quad \text{Eq. 5-18}$$

Substituting equation 5-17 into 5-18 yields for the maximum wave height:

$$H_{\max} = \frac{2(1-n)\Delta \sin(\phi - \beta) \cosh kh}{k \sin(\phi)} \quad \text{Eq. 5-19}$$

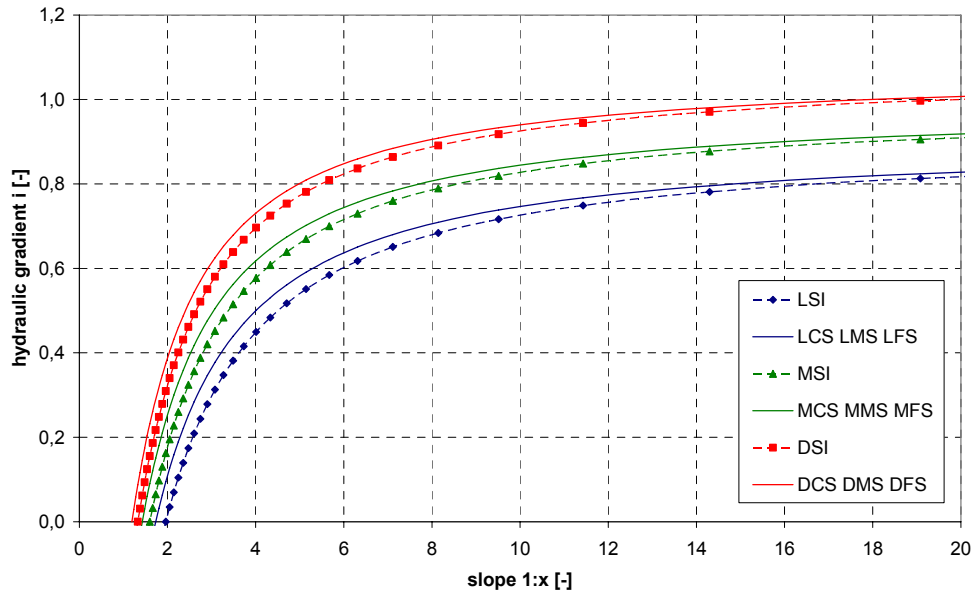


Figure 5-9: Maximum hydraulic gradients for all non-cohesive reference soils against slope

To gain insight in the impact of wave loads on micro-stability, the maximum waves according to equation 5-19 have been calculated for LSI and DCS/DMS/DFS for different water depths ($h=5, 10, 15, 20\text{m}$). It was already proved that the normative wavelength (and corresponding wave period) is dependent on the water depth. So, from all calculations the most unfavourable wavelength has been determined. The normative wave heights are presented in Figure 5-10: the solid lines represent the maximum wave heights for a certain slope in LSI (weakest reference soil) and DCS (=DMS, DFS; strongest reference soil); these lines convert into dashed lines at the depth-limited wave height. The most unfavourable wave periods for a certain water depth have been used to determine a sort of upper boundary.

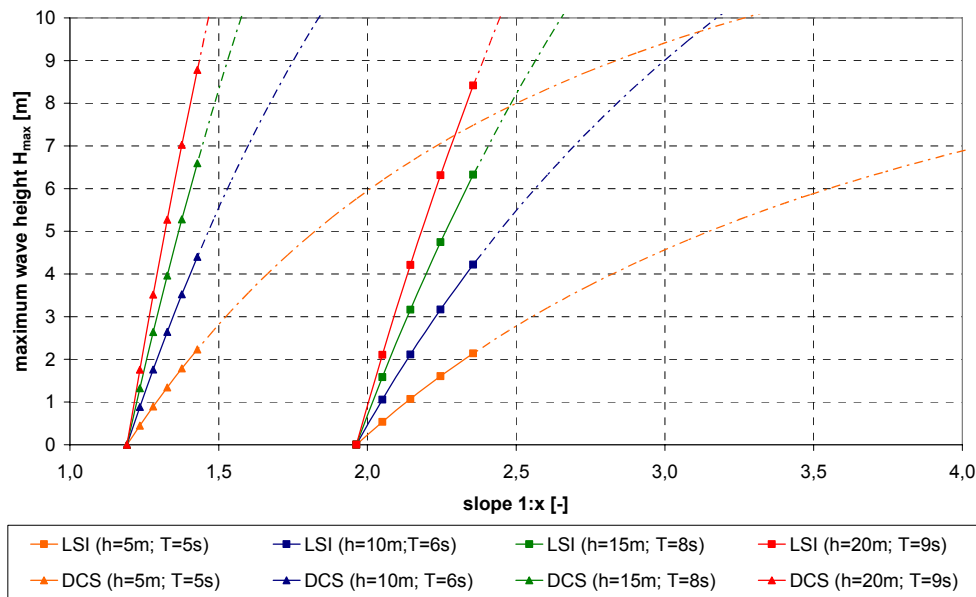


Figure 5-10: Maximum wave heights (upper boundaries) for weakest and strongest reference soil

The primary conclusion to be drawn from Figure 5-10 is that the impact on micro-stability of depth-limited waves does not exceed approximately 20%. This means that a just stable 'unloaded' slope in non-cohesive soil has to be flattened by about 20% to be able to resist the largest possible waves. The critical area of the slope is the upper part. Inside trenches and channels not only will the depth be larger, due to wave shoaling the wave impact will further decrease. Apparently the direct effect of waves remains rather small and is often implicitly accounted for in the safety factor.

5.2 Indirect Plastic Behaviour

5.2.1 Dynamic liquefaction by wave loads

Static liquefaction, as explained in Paragraph 4.2, was determined considering a sudden (relatively small) trigger. When this trigger force is a large, cyclic force (earthquakes, storm waves, tsunamis) another approach is advised. In this thesis, this case will be investigated for large storm waves. In Paragraph 5.1 the direct, elastic effect of waves on the sea bed was explained: waves instantaneously affect pore water pressures and total stresses, thus implicitly effective stresses. Although this reduction of strength can result in stability problems, these elastic changes of water pressures and effective stresses have no net effect over a full wave cycle. But as was mentioned before, there is also an indirect, plastic effect. Every wave load cycle will cause compaction of the soil due to shear stress variations, generating excess pore water pressures (EPP's). Depending on the consolidation properties of the soil these EPP's fade out after every wave cycle or cumulate during a storm.

In this thesis this process will be explained briefly and demonstrated with the help of the rather simple, but therefore very clear computer program *MCycle* by GeoDelft, which calculates the development of water overpressures in time under storm conditions. However, when these calculations show a large pressure built-up, more advanced analyses are recommended.

MCycle solves the 1-dimensional consolidation equation for the excess water pressure 'u':

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} - A \quad \text{Eq. 5-20}$$

in which "pumping term" A represents the undrained internal generation of water pressure ($A = (\delta u / \delta t)_{\text{undrained}}$) which is dependent on the cyclic properties of the soil, the wave characteristics (external load) and the extent to what these waves are felt inside the slope. This parameter will decrease with depth and vary in time. Parameter ' c_v ' represents the consolidation coefficient according to equation 3-5. Because both the consolidation properties and the pumping term vary with depth, the seabed is divided into a number of soil layers.

The hydraulic permeability k is kept constant during the compaction process, whereas the consolidation coefficient c_v is defined at a certain soil stress and is proportional to the square root of the effective stress. It is important to note that the consolidation coefficient c_v is not a constant, but varies with both the level of stress and degree of consolidation. For practical site settlement calculations, however, it is sufficiently accurate to measure c_v relative to the loading range applicable on site and then assume this value to be approximately constant for all degrees of consolidation (except for very low values).

The approach adopted in *MCycle* is based on the following assumptions:

-) transient waves in one direction;
-) the number of waves needed to build up the water overpressure is relatively large
-) no construction present on the seabed;
-) compressibility of the pore water is small compared to compressibility of the grain skeleton;

The first three assumptions are related to the 1-dimensional approach: the vertical consolidation is predominant. Because of the large number of waves (and therefore small influence of a single wave) the horizontal consolidation may be neglected. In case of only one or a few very steep waves, the problem becomes 2-dimensional and far more complicated. When considering the *direct, elastic* effect in the previous paragraph, the variations in pore water pressures and effective stresses under a single wave were determined. In that approach, no pressure generation effects were assumed to occur.

The prerequisite of 'a large number of waves' also has a more practical background: the wave climate has to be described as a regular wave field. In the *direct, elastic* approach one could suffice with the largest occurring wave; in this *indirect, plastic* approach a representative wave height has to be chosen: the significant wave height H_s . This wave height has to be chosen very carefully. If one takes a long storm duration and a lower value of the significant wave height, a small group of larger waves can increase excess pore pressures (EPP's) considerably and cause failure. But storms also must contain at least 50 waves to fulfil the above mentioned assumptions. A sound description of the complete storm may ask for multiple storms with various significant wave heights.

The negligible compressibility of the pore water means that volume reduction due to cyclic compaction under fully saturated conditions only occurs if pore water is pressed out. If this pore water cannot be pressed out (undrained conditions), excess pore water pressures arise: effective stresses are reduced. The tendency to compact will be larger for larger variations in the shear stresses. Important to note is that not the absolute, but the relative variation of these shear stresses is meant (like in all friction-based calculations): $\Delta\tau / \sigma_{v0}'$. Parameter σ_{v0}' represents the initial vertical effective stress. Because this effective stress decreases during a storm, the soil is subjected to an increasing 'load'. This behaviour is often mentioned as the 'explosive character' of cyclic liquefaction. Therefore in practice often an upper limit of the excess water pressure is defined:

$$u \leq 0,5\sigma_{v0}' \quad \text{Eq. 5-21}$$

although 'real' cyclic liquefaction occurs if effective stresses are reduced to zero. The sensitivity of the cyclic response of sand to water pressures of 40-50% of the effective stresses is presented in Figure 5-11 in which the excess pore pressure divided by the effective vertical stress is plotted against the number of waves divided by the total number of waves at which liquefaction will occur (N_{l0}). It can be observed that during the first number of waves N , the excess pore pressure increases rather fast to a value of about 30%, while quite a number of waves are needed to reach an excess pore pressure of 50%, but then only a few extra waves can cause liquefaction.

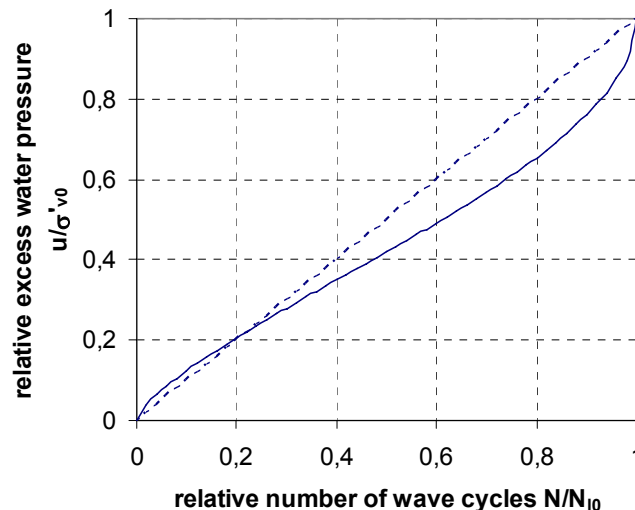


Figure 5-11: Relative excess water pressure against relative number of waves

Two special situations

When looking at the consolidation equation, one can distinguish two special situations:

- I) the consolidation coefficient is very small: no noticeable consolidation occurs and the increase in water pressure is equal to the generation term 'A': $\delta u / \delta t = A$
- II) water pressure is constant in time: $\delta u / \delta t = 0$ and the equation 5-20 becomes:

$$c_v \frac{\partial^2 u}{\partial z^2} = -A$$

Eq. 5-22

This represents the stationary situation that during every wave cycle the generated water pressure disappears due to consolidation, so no net pressure built-up occurs. Both situations are illustrated in Figure 5-12.

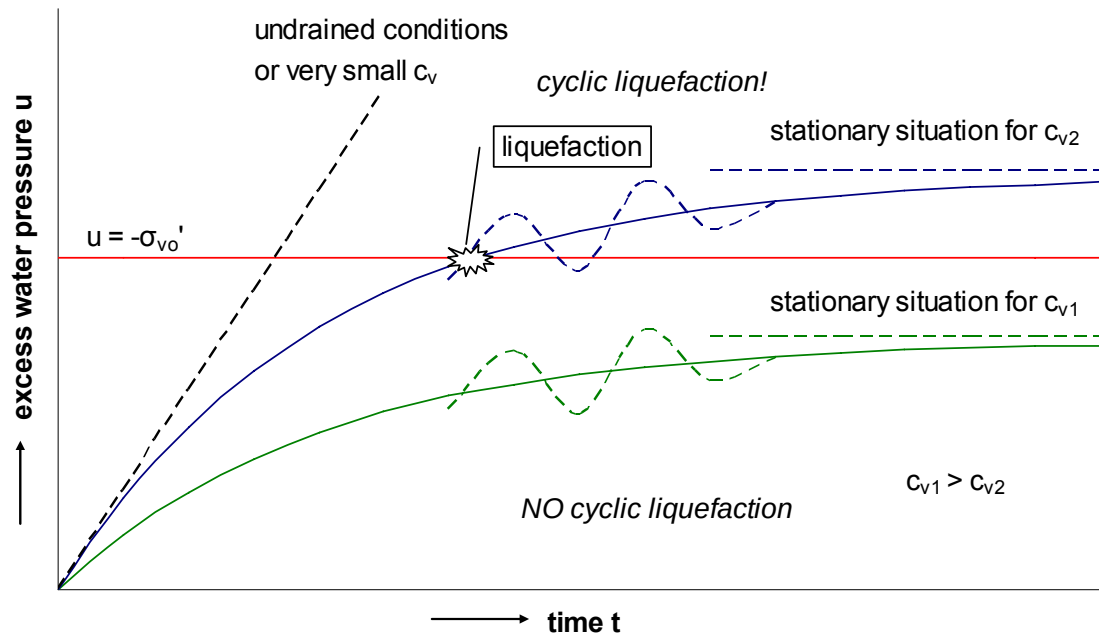


Figure 5-12: Excess pore pressure for two soils with different consolidation coefficients

Soil '1' (green line) will never reach the liquefaction criterion ($u = -\sigma'_{v0}$), whereas soil '2' cannot reach the stationary situation, because the EPP becomes larger than the effective stress and cyclic liquefaction occurs. Please note that in engineering practice often the above mentioned more conservative liquefaction criterion (equation 5-21) is applied due to the 'explosive character'.

The stationary situation can be explained as follows. Due to drainage, volume reduction occurs (increase of isotropic stresses) and this will result in a decreasing reduction of the vertical effective stress during each wave cycle. As a consequence the excess water pressure will decreasingly increase, until the stationary situation is reached. Now the soil behaviour can be interpreted as 'completely drained'.

However, under drained conditions the volume reduction per wave cycle will attenuate, which can also be observed in drained cyclic triaxial tests. The consolidation/drainage behaviour becomes predominant and the EPP starts to diminish, see Figure 5-13. So, in fact the stationary situation will in reality never be reached.

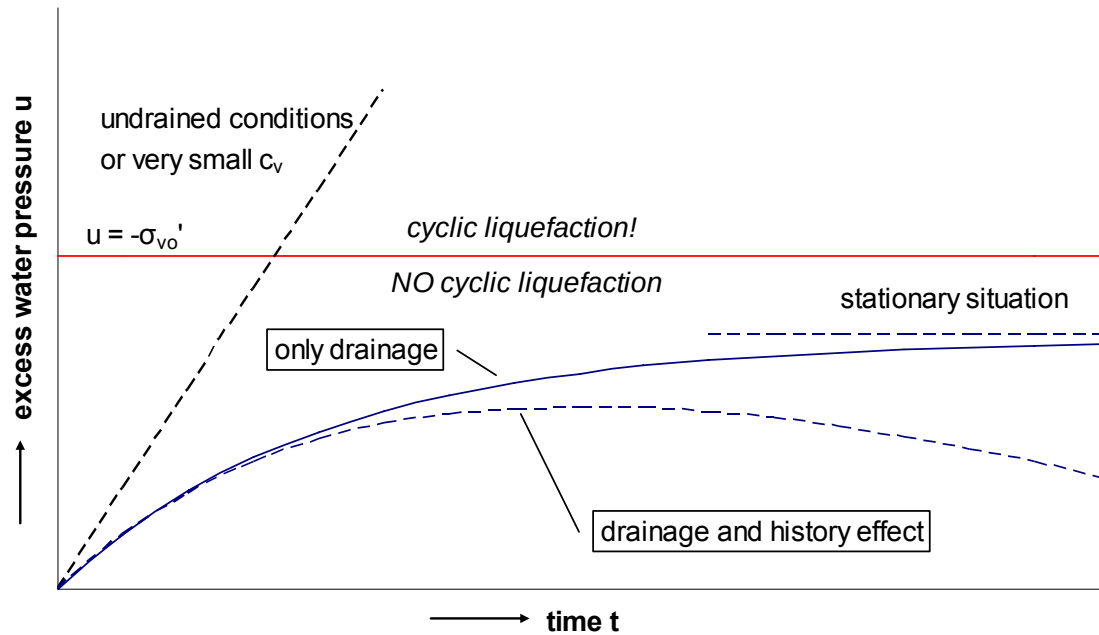


Figure 5-13: Excess pore water pressure under undrained conditions and drained conditions with and without history effect

The above described effect is often described as pre-shearing or 'history effect' (the extent to what the soil has been subjected to cyclic loading before). The decrease in porosity per wave cycle (Δn) is a good measure for this history effect. The fact that 'virgin' soils react different to storm waves than 'mature' soils can be explained by this history effect.

The above described, mainly qualitative description of the behaviour of soils subjected to wave loads has to be expressed in formulae. Therefore first the undrained cyclic behaviour is taken as a starting point.

Seed en Rahman (1976, 1978) deduced a formula for the number of load cycles to induce (undrained) cyclic liquefaction from a lot of laboratory tests:

$$\log N_{10} = 3 - \left(\frac{67,5}{RD} (\tau / \sigma_{v0}') - 12,25 \right)^{0,351} \quad \text{for } N_{10} \leq 1000$$

$$\log N_{10} = 3 + \left(12,25 - \frac{67,5}{RD} (\tau / \sigma_{v0}') \right)^{0,351} \quad \text{for } N_{10} > 1000 \quad \text{Eq. 5-23}$$

This formula has been plotted in Figure 5-14 for different relative densities. Waves that induce shear stresses of about 15 % of the vertical effective stress almost cause immediate liquefaction in loosely packed soil ($RD = 0,3$), after 9 cycles in medium packed soil ($RD = 0,5$) and after 50 cycles in densely packed soil.

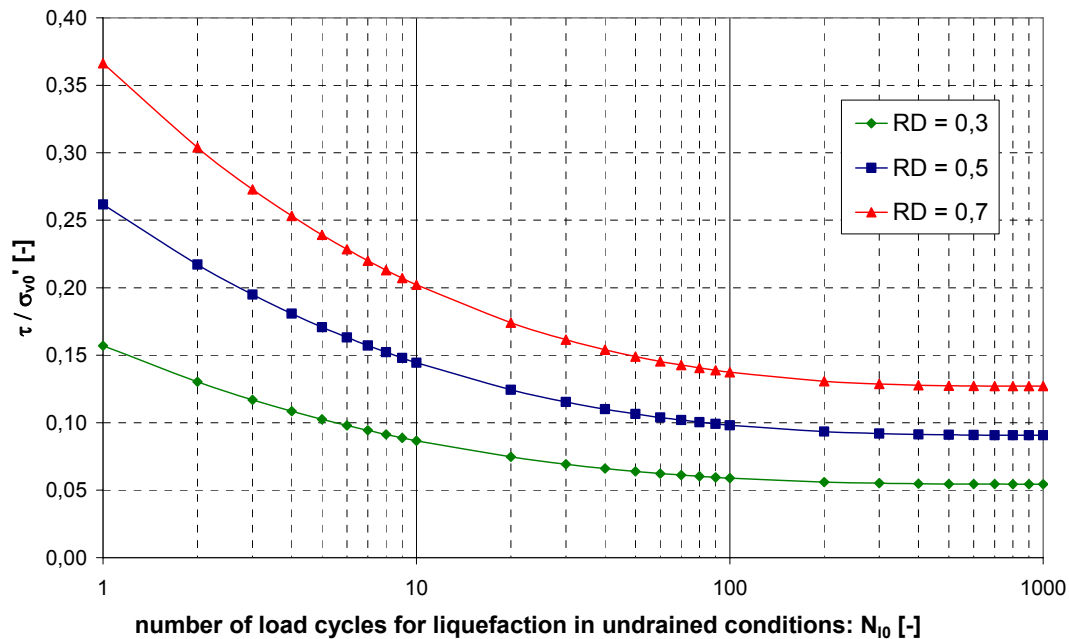


Figure 5-14: Development of relative shear stresses with number of load cycles for three different relative densities

But are these relative shear stresses realistic when it comes to the reference geometries and wave heights of this thesis? Therefore the relative shear stresses have been plotted for depth-limited waves in water depths of 5, 10 and 20 m and wave periods of 5, 10 and 15 s with the remark that the maximum wave pressures for a certain water depth occur for different wave periods (see Paragraph 5.1.4).

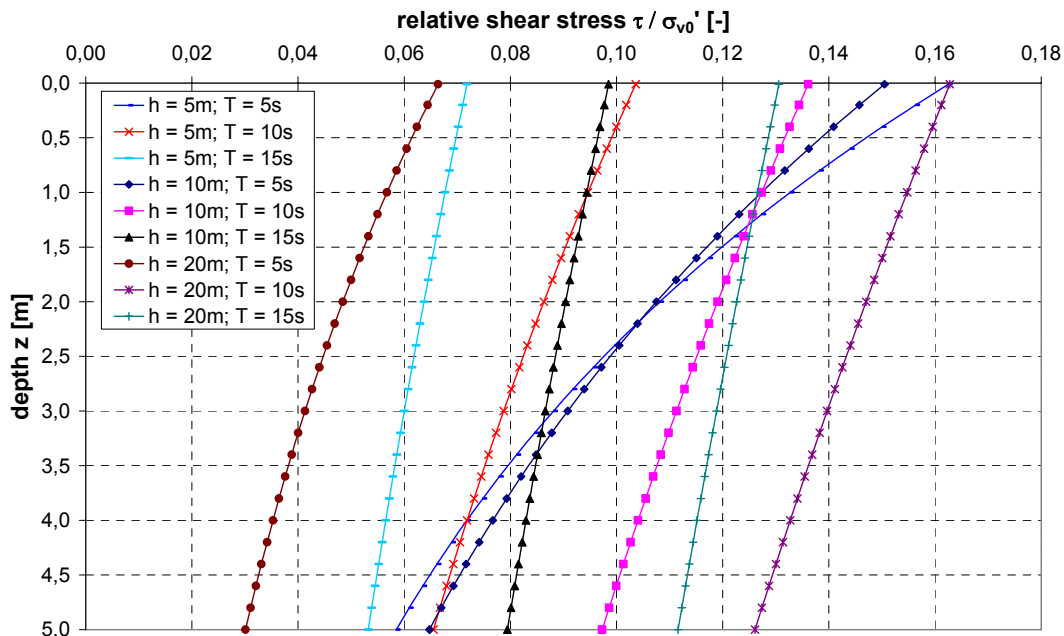


Figure 5-15: Development of wave-induced relative shear stresses as a function of sub-bottom depth for different water depths and wave periods

So especially loosely to medium packed sands are susceptible to cyclic liquefaction. Furthermore it can be observed that relative shear stresses faster diminish with depth for smaller wave periods.

The relative excess pore pressure shows strongly non-linear behaviour, as was mentioned before as the explosive character. Seed and Rahman delivered the following expression that was plotted in Figure 5-11:

$$u / \sigma'_{v0} = 0,64 \arcsin(N / N_{l0})^{0,714} \quad \text{Eq. 5-24}$$

in which N is the number of wave cycles at a certain moment.

Pre-shearing or history effect

From test results obtained during the design of the Eastern Scheldt Storm Surge Barrier Smits et al. [1978] introduced an expression for the number of wave cycles to induce cyclic liquefaction after compaction of the sand skeleton due to drainage:

$$N_{l0;hist} = 10^{\varepsilon_{hist} \Delta n} N_{l0} \quad \text{Eq. 5-25}$$

in which Δn is the reduction of the porosity due to cyclic loading; ε_{hist} is a constant representing the history effect. From the test results this constant could be estimated at $\varepsilon_{hist} = 830$, but most of the times more conservative values are used, like $\varepsilon_{hist} = 333$, because there is little knowledge of the 'real' behaviour in nature. There is however no theoretical base for this value.

The introduction of pre-shearing causes a reduced sensitivity to generation of excess water pressures. Because of the dependence on the reduction of porosity Δn this history effect remains small in case of almost undrained conditions (small c_v and large H_s), because compaction does not occur.

Equivalent wave height and period

The irregular wave field in reality is in many hydraulic applications described with a significant wave height H_s and wave period T_s . It is also desirable to be able to describe the relative shear stress changes caused by an irregular wave field with a certain wave height and wave period. To that purpose the impact of different wave groups has been studied. When the so-called equivalent wave height H_e is equal to the significant wave height H_s , it appeared that the equivalent wave period T_e should be 2,3 times the wave period belonging to the significant wave height T_s to produce the same pressure built up during a storm. This equivalent wave period T_e is only needed to calculate the total number of waves within a certain period; the relative shear stress changes are calculated with the 'normal' wave properties H_s and T_s .

This value of 2,3 (*MCycle* uses 2,0) can be explained by the fact that H_s is a rather large wave; only 13,5% of the waves will exceed the significant wave height in case of a Rayleigh-distributed narrow wave spectrum. So, the pressure built up will be governed by the more frequent smaller waves or, and this approach has been adopted, waves with a artificially increased wave period:

$$\begin{aligned} H_e &= H_s \\ T_e &= 2,3 T_s \end{aligned} \quad \text{Eq. 5-26}$$

Pumping term A

The pressure generation or "pumping" term A can be obtained from the expression for the relative excess pressure (equation 5-24) in which the quotient N/N_{l0} is substituted by a new expression, which combines the history effect and the efficient wave period:

$$\frac{N}{N_{l0;hist}} = \frac{t}{N_{l0;hist} T_e} \quad \text{Eq. 5-27}$$

The expression for the excess water pressure becomes:

$$u(t) = 0,64 \sigma'_{v0} \arcsin \left(\frac{t}{N_{l0;hist} T_e} \right)^{0,714} \quad \text{Eq. 5-28}$$

When this expression is differentiated to t, the undrained generation term A is obtained:

$$A = \left(\frac{\partial u}{\partial t} \right)_{undrained}$$

Eq. 5-29

Calculation method

The calculation method of *MCycle* by GeoDelft is roughly as follows:

- with the storm and wave input the wave pressures on top of the seabed and the relative shear stress changes as a function of sub-bottom depth are calculated according to the 2-dimensional direct, elastic approach of Paragraph 5.1 (Eq. 5-11);
- with this relative shear stresses and the relative density the number of load cycles is determined according to Eq. 5-23;
- together with the wave period, the undrained pumping term 'A' can be calculated as a function of depth according to Eq 5-28 and 5-29;
- for every depth the 1-dimensional consolidation equation is solved, resulting in the EPP as a function of depth and time.

The 1-dimensional approach is widely adopted, but there is some uncertainty about the correctness of the uncoupled analysis of the 2-dimensional wave penetration and 1-dimensional generation of EPP's.

The computer model *MCycle* can calculate three different situations:

- water overpressure in a horizontal sea bed;
- water overpressure in an infinite under water slope that is entirely located below sea level outside the breaker zone;
- water overpressure under a construction on the sea bed, assuming the length of this construction is much larger than the width.

The second situation is solved just like the situation of a horizontal sea bed with pore water flow perpendicular to the slope. Additionally, the stability of the slope will be checked.

Restrictions

Some other restrictions of *MCycle* (some were already mentioned) are:

- this program is 1-dimensional. 2-dimensional effects are only accounted for in a schematized way;
- the permeability of the soil is kept constant during compaction by waves;
- only five soil layers can be modelled;
- the number of waves needed to increase the water pressure is relatively large; in other words generation term 'A' is very small;
- three storms with wave heights only varying in time can be prescribed;
- the compressibility of the sand skeleton is much larger than the compressibility of the pore water

Numerical solution procedure

For the numerical solution of the consolidation equation a forward explicit integration scheme is applied. To guarantee stability, the time step is restricted to:

$$\Delta t \leq \frac{(\Delta z)^2}{2c_v}$$

This means that the maximum time step will be determined by the upper sublayer (smallest thickness Δz) and the initial situation ($t=0$), because the consolidation coefficient c_v will decrease in time when water pressures increase and effective stresses decrease (c_v is related to root of the effective stress).

Stability under wave loads of an infinite long slope

The above mentioned approach for a flat seabed is adopted for wave loads on a slope. At first the actual slope of the seabed is ignored: the consolidation equation is applied at a line perpendicular to the seabed; the water depth (and the corresponding wave loading) is of course dependent on the location on the slope. When the bottom is

sloping, in principal multiple calculations (for every point along the slope) with various water depths and wave heights (which are depth-related) have to be done.

In the case of trenches and channels, which are in fact local 'dips' in the surrounding seafloor, the surrounding water depth h_0 and depth-limited wave height are normative. Failure therefore means: a small, relatively shallow sliding of the upper part of the slope, which can result in a large mass flow, downward the slope. At the beginning, failure due to wave loads contains small amounts of sediment, but can eventually become just as dangerous as slip circle failure or static liquefaction.

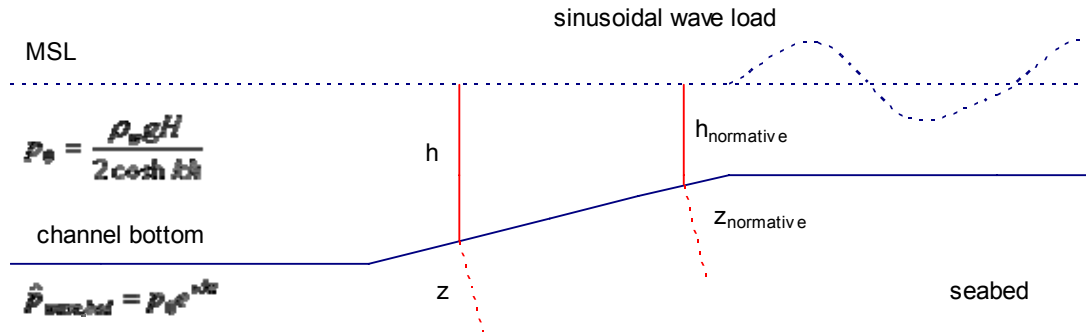


Figure 5-16: Schematization of calculations at two locations on the sloping bottom

After the consolidation equation in *MCycle* is solved numerically, the actual slope is taken into account and the 'stability check' will be performed. The safety factor, under the influence of wave loads, can be calculated, based on a simple schematization of force equilibrium that is very similar to the approach of Paragraph 5.1.5. Instead of the seepage force on the outer grains, now the difference in wave pressure ($p_0 - p_1$) over a certain depth is calculated. The maximum amplitude of this wave pressure is represented by the variation of the effective stress (equation 5-12). The fact that the z-axis isn't really perpendicular to the slope surface is neglected. Another difference with the 'seepage approach' is the addition of a term representing the excess water pressure 'u'.

Force Equilibrium
-perpendicular to slope:

$$p_0 + \rho_w g h + \rho_{sat} g z \cos \beta = p_1 + \rho_w g (h + z \cos \beta) + u + F_g \quad \text{Eq. 5-30}$$

in which F_g represents the total pressure by the grains from inside the slope.

-parallel to slope:

$$F_f = (\rho_{sat} - \rho_w) g z \sin \beta \quad \text{Eq. 5-31}$$

in which F_f is the friction force along the shear surface. The slope will be stable, if:

$$F_f < F_g \tan \phi \quad \text{Eq. 5-32}$$

Combining these three equations yields an expression for the excess water pressure, which can be interpreted as the slope stability check:

$$u < (p_0 - p_1) + (1 - n)(\rho_s - \rho_w) z \frac{\sin(\phi - \beta)}{\sin(\phi)} \quad \text{Eq. 5-33}$$

in which $(p_0 - p_1)$ is represented by:

$$p_0 - p_1 = \Delta \sigma'_z = \frac{\rho_w g H}{2 \cosh kh} k z e^{kz} \quad \text{Eq. 5-34}$$

If anytime during the calculation procedure instability occurs, the computer program stops the simulation and the input data has to be adjusted.

5.2.2 Computations with *MCycle* for different slopes

Most important issue is of course the influence of the slope to the cyclic liquefaction potential: how effective is flattening of the slope in increasing slope stability? As was concluded before, see Paragraph 4.2.2, this was a very powerful measure when dealing with static liquefaction.

A number of 36 simulations were done for all non-cohesive reference soils (see Appendix C for the soil properties) for slopes (12 in total) of 1:3, 1:5 and 1:7 in a surrounding water depth of 10m. The significant wave height was varied in steps of 0,25 m to find the maximum wave height which just does not cause slope failure due to cyclic liquefaction with the restriction that the maximum wave height is depth limited to half of the water depth: $H_s = 0,5h_0$.

In all simulations the same storm pattern is used. The total storm duration is 3 hours according to a trapezoidal profile; see the little graph at the right bottom in the next figures. The soil is again considered to be completely homogeneous, represented as a rather thick soil layer of 20 m, divided into 40 sub layers. Some representative results are presented, but because of the similar appearance of many of the graphs, not all graphs will be drawn.

In Figure 5-17 for MFS can be seen that the excess pore pressures in the upper layers increase rather fast (*pink* line), but when the storm progresses these pore pressures diminish and spread out over the depth. The critical period is just after the storm reached its maximum wave height. When the storm has ended, excess pore pressures reduce very fast. If we consider the same seabed, but now in loosely packed fine sand (LFS), see Figure 5-18, it becomes clear that the significant wave height is much smaller (4 m against 1,44 m).

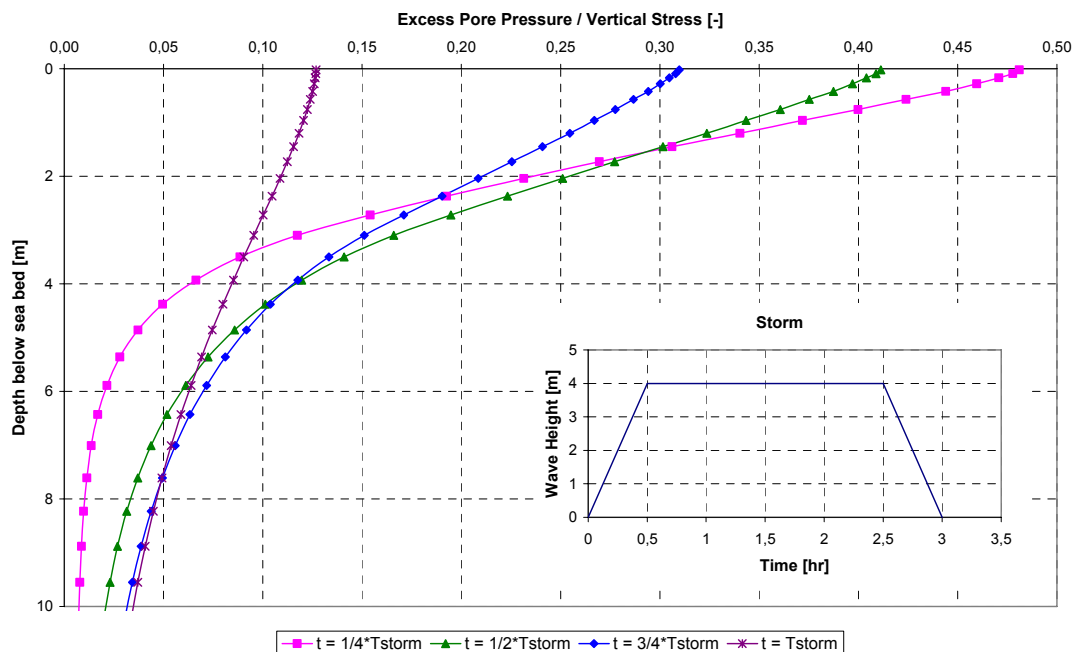


Figure 5-17: Excess pore pressure development in a slope of 1:5 in MFS

This is caused by the most important difference between LFS and MFS, the relative density, which appears to be the most influential parameter: compare $RD_{LFS}=16\%$ and $RD_{MFS}=43\%$. Although the normative storm is much smaller, the excess pore pressures pattern is almost similar for both soils, because the other important soil properties, hydraulic permeability and consolidation coefficient, only slightly differ from each other. It appears however that very loose soils increase their resistance to cyclic liquefaction faster than denser soils. In fact every wave cycle compacts the soil, thereby increasing this resistance. This can be observed from the somewhat lower excess pore pressures in MLS for $\frac{1}{2}$ and $\frac{3}{4}$ of the total storm period.

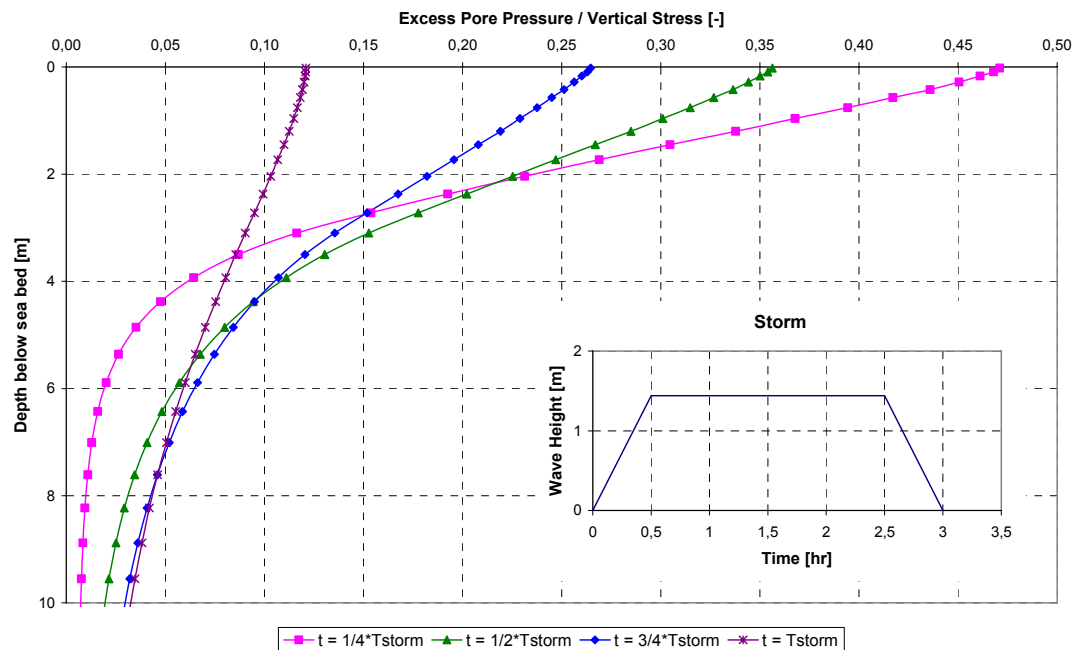


Figure 5-18: Excess pore pressure development in a slope of 1:5 in LFS

From the above it can be concluded that soils with low relative densities in moderate wave climates become more compacted, either by the often passing waves or already during dredging activities. The before mentioned history effect, although difficult to account for, is important. It seems relevant to neglect failure caused by waves that are common in the area under investigation (i.e. small waves), because the soil will automatically have adapted to this wave climate.

In DFS the slope even remains stable when subjected to depth-limited wave heights. This situation is considered 'unconditionally stable'. Dense fine sand (DFS) is stable for all slopes and wave heights, because of its combination of a large relative density, hydraulic permeability and consolidation coefficient, see Figure 5-19. Even waves of 5 m are hardly capable of causing excess pore pressures. Another remarkable thing is the fact that pore pressures keep increasing during the storm, while loosely packed soils were mainly susceptible to cyclic liquefaction in the beginning of a storm period. This is probably caused by the large number of wave cycles to induce liquefaction because of the larger relative density.

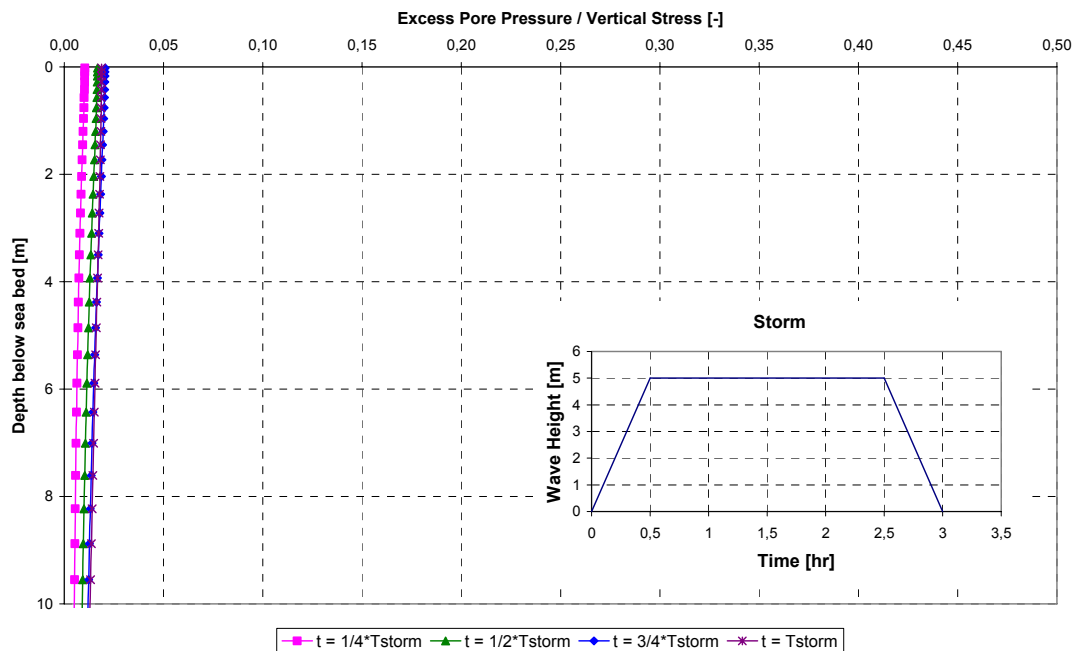


Figure 5-19: Excess pore pressure development in a slope of 1:5 in DFS

Until now, excess pore pressures were presented relative to vertical stresses. It is however interesting to see where maximum absolute excess pore pressures occur to demonstrate the differences between medium and densely packed soils, see Figure 5-20 and Figure 5-21.

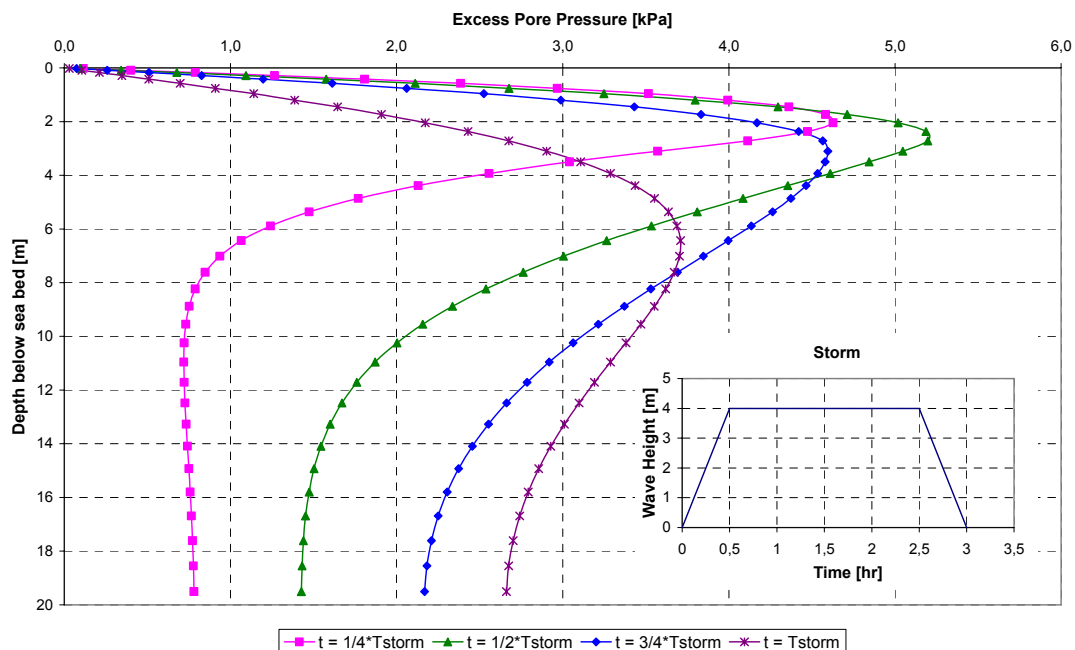


Figure 5-20: Absolute excess pore pressures in a slope of 1:5 in MFS

In medium packed sand, maximum absolute excess pore pressures occur a few meters below the seabed and halfway the storm period. Pore pressures tend to drain off to deeper layers, where they can do less harm, because of the larger geostatic stresses. In soils of higher density, excess pore pressures keep increasing in time and in depth below the sea bed. Because of this large depth, these pore pressures are not very dangerous. Even large long-lasting storms will not cause problems.

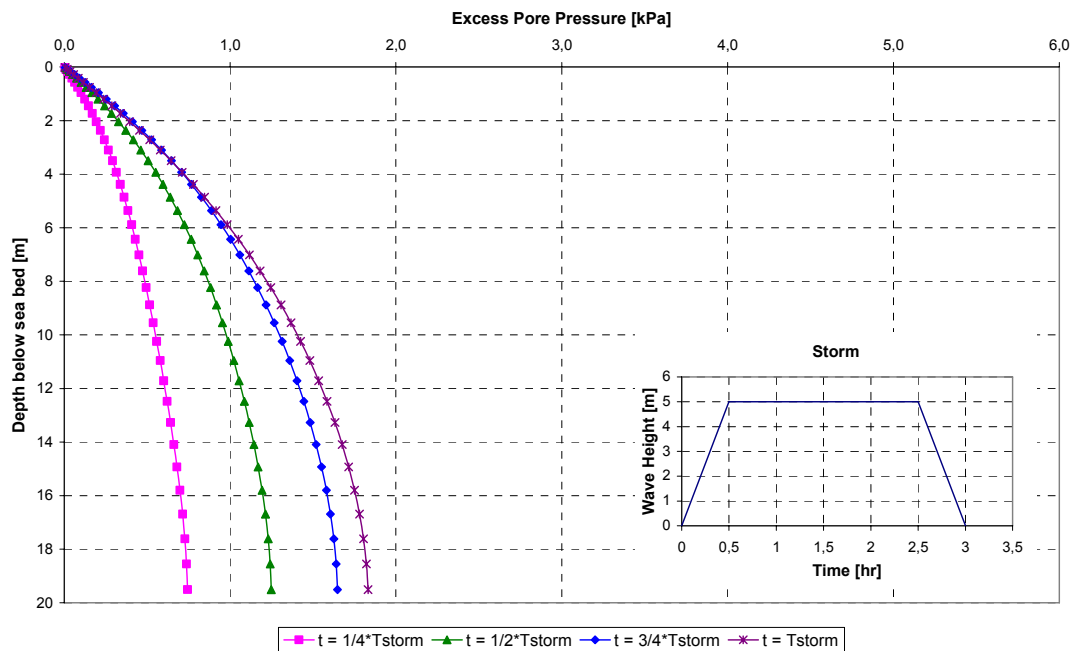


Figure 5-21: Absolute excess pore pressures in a slope of 1:5 in DFS

All 36 computations are summarized in Figure 5-22 with the remark that the densest packed soils (DCS, DMS, DFS, DSI) are neglected, because no instability occurred for maximum depth-limited waves. On the other hand, the loosest soils fail when subjected to relatively small waves, which is an indication that such densities are not very likely in a moderate wave climate.

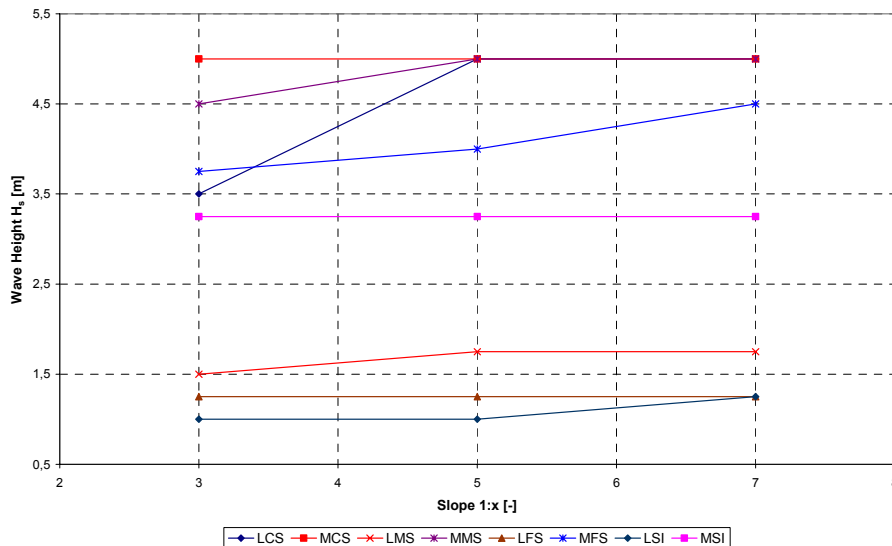


Figure 5-22: Maximum wave heights for just stable slopes

The common medium packed soils are sensitive to storm waves, with the exception of MCS, which is unconditionally stable ($H_s > 5$ m):

MSI: $H_s \approx 3,25$ m

MFS: $H_s \approx 4,0$ m

MMS: $H_s \approx 4,75$ m

Please note that the abrupt bends in the lines are caused by the discrete steps in wave height of 0,25m. The remarkable line of LCS has a steep part due to the good consolidation properties and is then truncated down at the depth-limited wave height. It becomes clear that due to the superficial influence of waves, the slope steepness isn't that important. Coarser sediments show some increase of stability if the slope is

flattened, but fine sediments, like silts, seem to be insensitive to slope flattening, see also Figure 5-23. Clearly can be observed that maximum wave heights increase for larger grains (read implicitly: better consolidation properties). This behaviour is less pronounced with small waves, which can be understood when looking at Figure 5-14. At very small relative shear stresses, the lines are almost flat around 300 load cycles ($t/2,3T_s = 7200/(2,3*10)=313$ load cycles).

Especially medium packed fine and loosely packed coarse soils show some slope dependency. One should however keep in mind that once a flow slide has been initiated by large storm waves, the consequences are much larger for steeper slopes: the resulting sediment gravity flow will be much more erosive on these slopes. This computer program does not give any predictions about this phenomenon. One can imagine that erosion of the toe of a 1:3-slope by such a gravity flow truly affects slope stability; for example a circular slip circle may occur after cyclic liquefaction followed by a turbidity current.

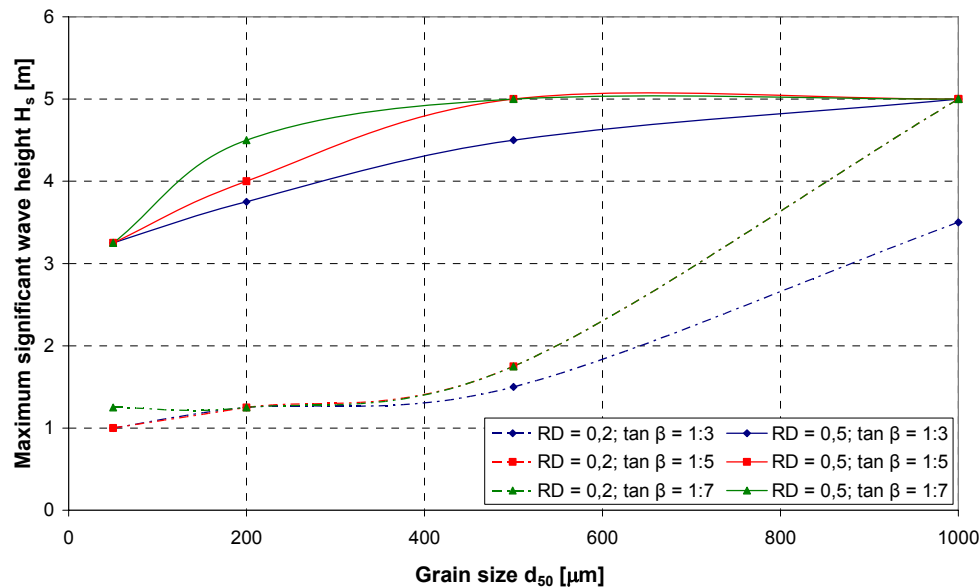


Figure 5-23: Maximum wave heights against grain sizes for different relative densities

We have seen that normative wave heights are quite different for the various reference soils, but is there also a difference in the so-called 'depth of influence'. In this thesis this 'depth of influence' is defined as the depth at which the excess pore pressures become smaller than 10% of the total vertical stress. And what about the storm duration?

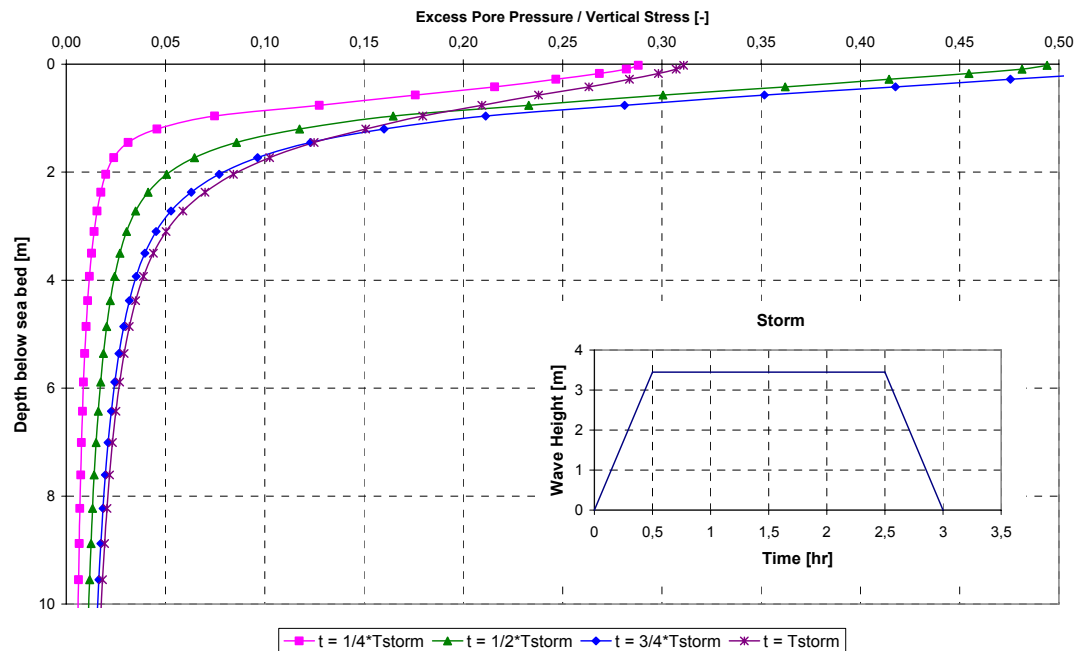


Figure 5-24: Excess pore pressure development in a slope of 1:5 in MSI

When comparing Figure 5-24 (MSI) with Figure 5-17 (MFS), one can notice two phenomena:

- the 'depth of influence' of MFS is about 5 m, whereas in MSI this depth reduces to 2 m;
- in MFS the critical time is just after the wave height has reached its maximum; after that time the excess pore pressures slowly diminish. MSI shows the opposite behaviour: excess pore pressures keep increasing and only start to fade when the wave height is decreasing.

These trends can also be observed when looking at MMS, Figure 5-25:

- the 'depth of influence' stretches to 8 m;
- the critical time is also in the beginning of the storm after which the excess pore pressures decrease very fast.

Both observations can be explained by the variation in hydraulic permeability ' k ' and vertical compressibility ' m_v ', which are together determining the consolidation coefficient ' c_v ', remember Eq. 3-5, in which the compressibility of the fluid (β) was neglected:

$$c_v = \frac{k}{\gamma_w (m_v + n\beta)} \quad \text{Eq. 5-35}$$

- a larger hydraulic permeability means that excess pore water pressures can easily dissipate to deeper layers, thereby increasing 'the depth of influence'. Because pore pressures are spread out over a larger area, bigger waves can be withstood;
- the vertical compressibility (m_v) determines the time in which compaction of the grain skeleton and the matching decrease of pore volume takes place. If this consolidation process progresses fast, the critical period will be at the beginning of the storm. So, although compression results in a stiffer soil skeleton and therefore a smaller susceptibility to cyclic liquefaction, too fast compression is dangerous.

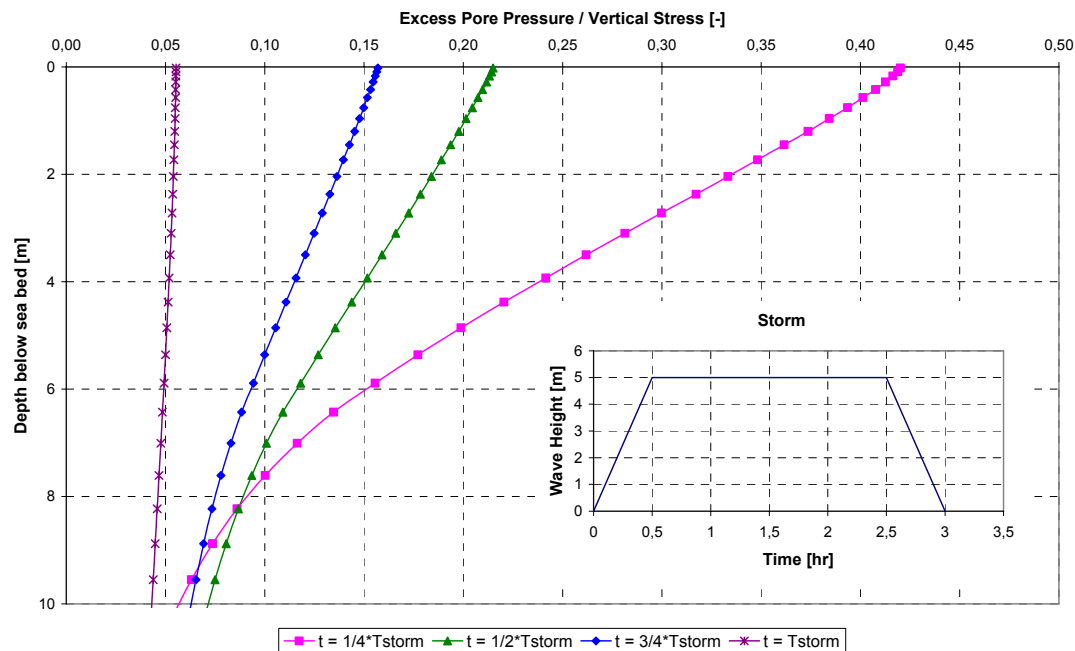


Figure 5-25: Excess pore pressure development in a slope of 1:5 in MMS

What can be said with respect to the reference soils? Coarser soils generally have a larger hydraulic permeability than finer soils. More loosely packed soils also tend to have a larger permeability, but that is often accompanied by a larger compressibility, so the consolidation coefficient for soils with equal grain sizes, but different porosities usually is of the same order. However, the unfavourable small relative densities of loosely packed soils result in a small number of load cycles to induce liquefaction (Figure 5-14) and therefore in a large pumping term (Eq 5-28 and 5-29), or in this approach, where the maximum wave height is investigated, a small relative shear stress, because the number of load cycles is determined by the storm duration.

Returning to the observed phenomena of *depth of influence* and *critical time*, it can be explained that MFS (Figure 5-17) starts to benefit from its better consolidation properties much faster than the equal dense MSI (Figure 5-24). Because EPP's in MFS faster drain off to deeper layers, the critical time is in the beginning of the storm. In MSI with the poorer consolidation properties, every new wave will add a small contribution to the EPP's, because hardly any drainage occurs. Therefore the depth of influence will be smaller and the critical time is at the end of a storm. Consequently the maximum resistible wave height will be somewhat smaller.

When soils with comparable consolidation properties, but different relative densities are compared (like MFS and LFS, Figure 5-18), the EPP development is rather similar, but the main difference is the maximum resistible wave height: the number of load cycles is determined by storm duration, but the relative shear stress has to be much smaller.

Please note that the unfavourable consolidation properties of pure silts are seldom found, because in nature often mixtures occur. For instance, silty clays are very impermeable, but have cohesive properties, which increase the resistance against liquefaction.

5.2.3 Computations with *MCycle* for different storms

To consider the influence of different storm patterns, the following situations have been simulated, all for 1:5-slopes in MFS:

- 1 'trapezoidal' storm with a wave height of 4,0 m and a duration of 3 h, similar to Figure 5-17;
- 1 'rectangular' storm with the same wave height and duration;

- C. 1 triangular storm with the same wave height and duration;
- D. 1 smaller storm with a wave height of 2,5 m and a duration of 2 h, followed after 1 h by storm a.

All storms are presented in the bottom right corner of the following graphs.

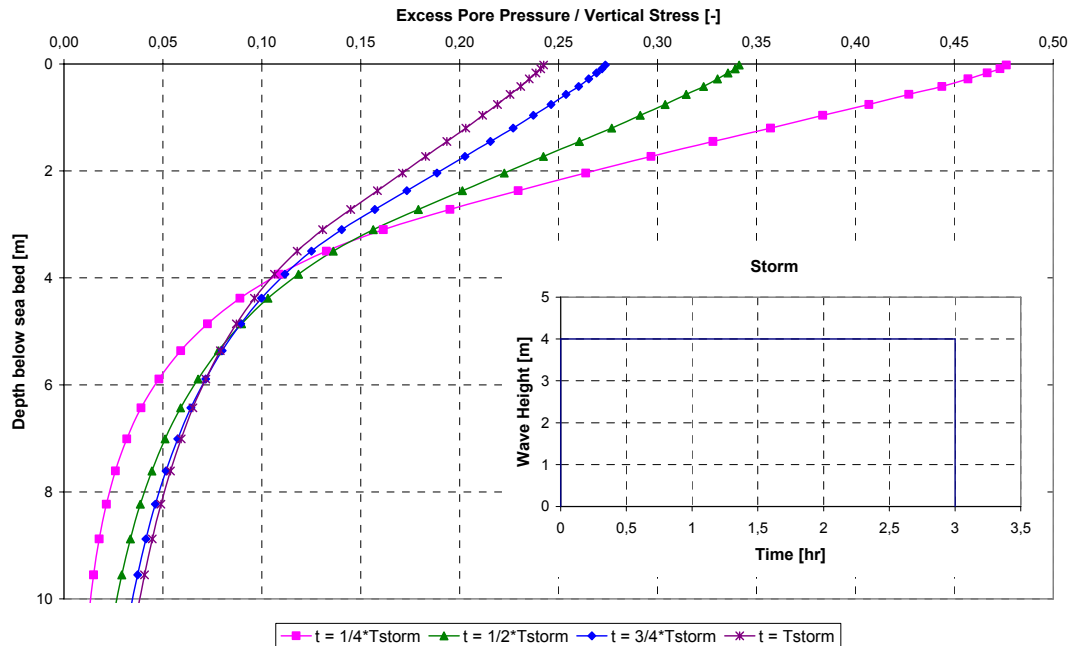


Figure 5-26: Excess pore pressure development in a slope of 1:5 in MFS during storm B

It is based on Figure 5-26 and Figure 5-17 not easy to compare the influence of the storm pattern, because of the presentation on certain moments. It is likely that the 'pink line' in Figure 5-26 does not represent the maximum excess pore pressures. The fact that the 'green and blue line' are somewhat more positioned to the right in Figure 5-17 can be explained by the fact that the consolidation process has less far developed. Profound data analysis shows that maximum absolute excess pore pressures are approximately the same (5,3 kPa), located at the same depth (2,37 m below the sea bed). These maximum *absolute* EPP's occur after 4557 s for storm A and after 2843 s for storm B, which shows that storm A has a delay on storm B of approximately a half hour.

Maximum *relative* EPP's (0,5) are found just below the sea bed. For storm A this maximum occurs at 3560 s, while for storm B this happens to be at 1827 s. In both cases maximum EPP's appear to occur about half an hour after the wave height has reached its maximum, which is consistent with the earlier conclusion that in medium fine sands the situation in the beginning of a storm is normative.

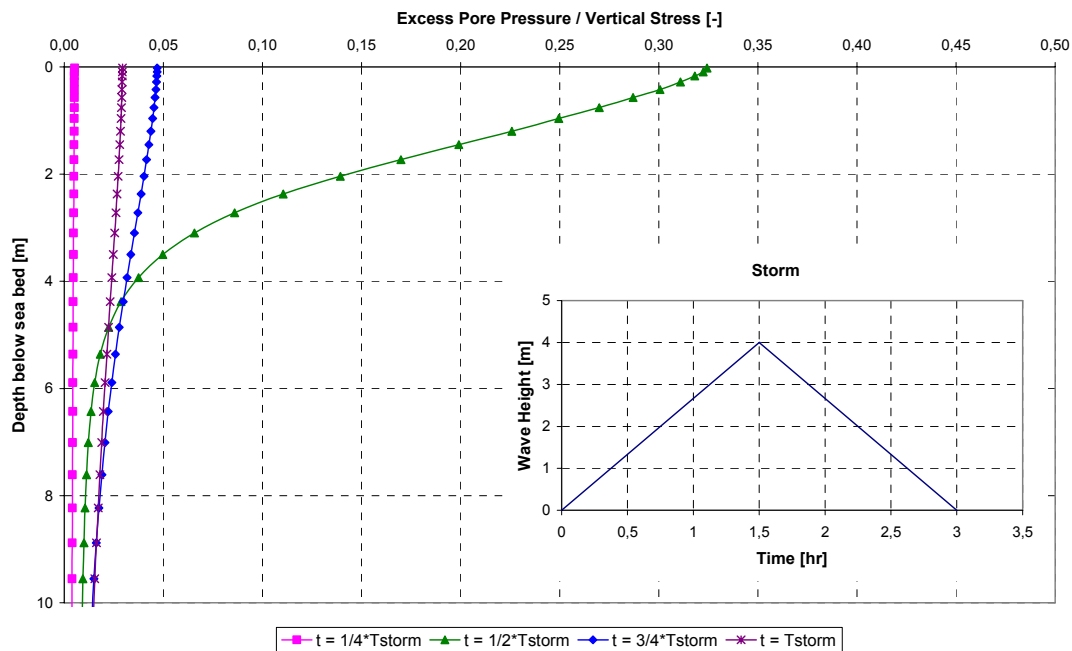


Figure 5-27: Excess pore pressure development in a slope of 1:5 in MFS during storm C

Storm c is less risky, because of the very slow increase of wave height. This is found in the maximum *absolute* EPP (2,95 kPa at a depth of 1,73 m: less time for the waves to penetrate into the seabed). Maximum *relative* EPP is 0,32. Both maxima occur just after the peak wave height.

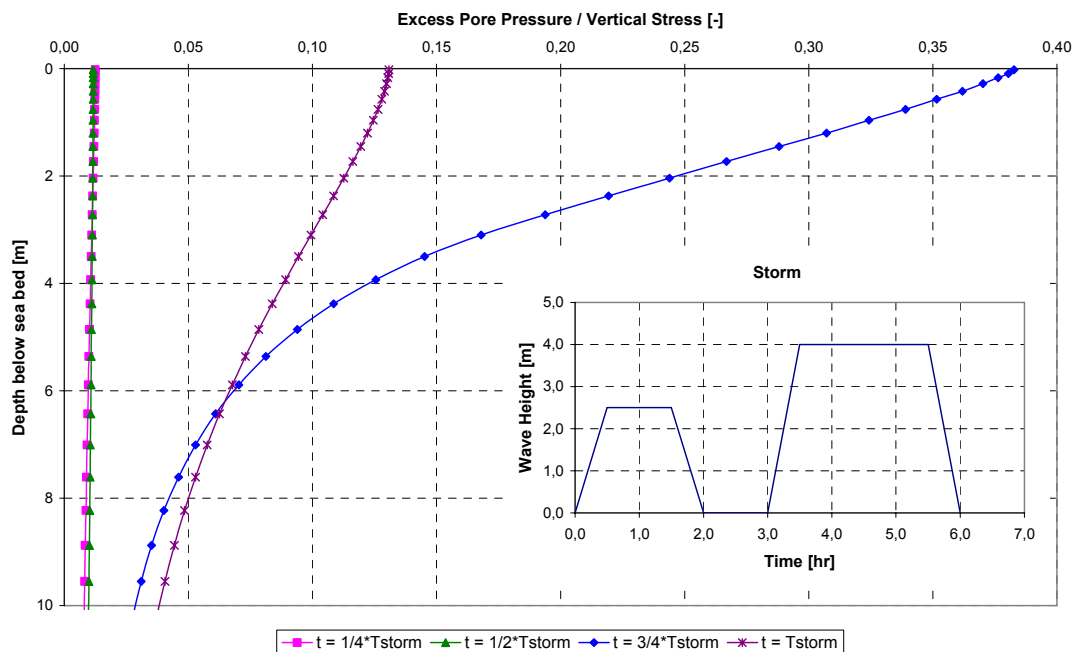


Figure 5-28: Excess pore pressure development in a slope of 1:5 in MFS during storm D

One often assumes that smaller preceding storms have a favourable influence on the stability of slopes. Although Figure 5-28 seems to confirm this assumption, it is hardly true. Compare the blue line to the green line of Figure 5-17 (also halfway the main storm) and there is no favourable effect noticeable. Although the EPP's after the first small storm have almost faded out, during the second storm EPP's are reached similar to storm A: maximum *absolute* EPP is 5,48 kPa and maximum *relative* EPP is 0,5. Although not expected, the influence of smaller preceding storms appeared to be very limited, at least for this combination of relative density and soil type.

It is however very well possible that longer and higher preceding storms (with only a slightly smaller wave height than the main storm) have a more favourable effect.

Studying the development of the relative density in time revealed that this soil with a porosity of 0,40 ($RD=0,5$) is only slightly compacted by waves. This is an indication that preceding storms will have very limited effect. Therefore the same comparison is made for a porosity of 0,42 ($RD=0,4$), see Figure 5-29 and Figure 5-30.

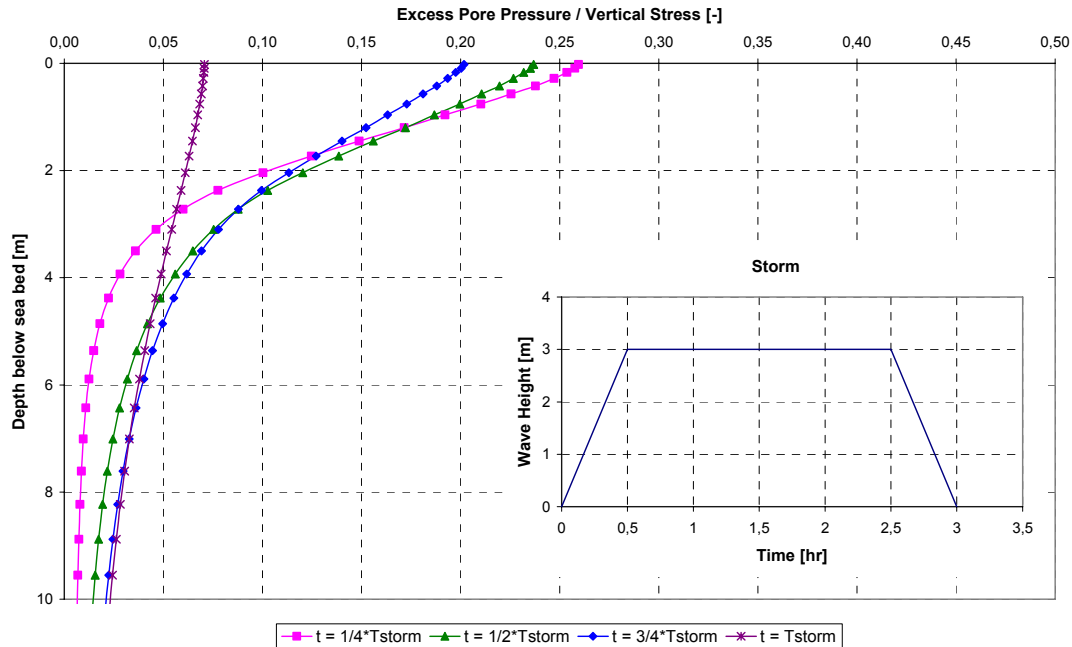


Figure 5-29: EPP-development in a slope of 1:5 in MFS with $n=0,42$ during storm type A

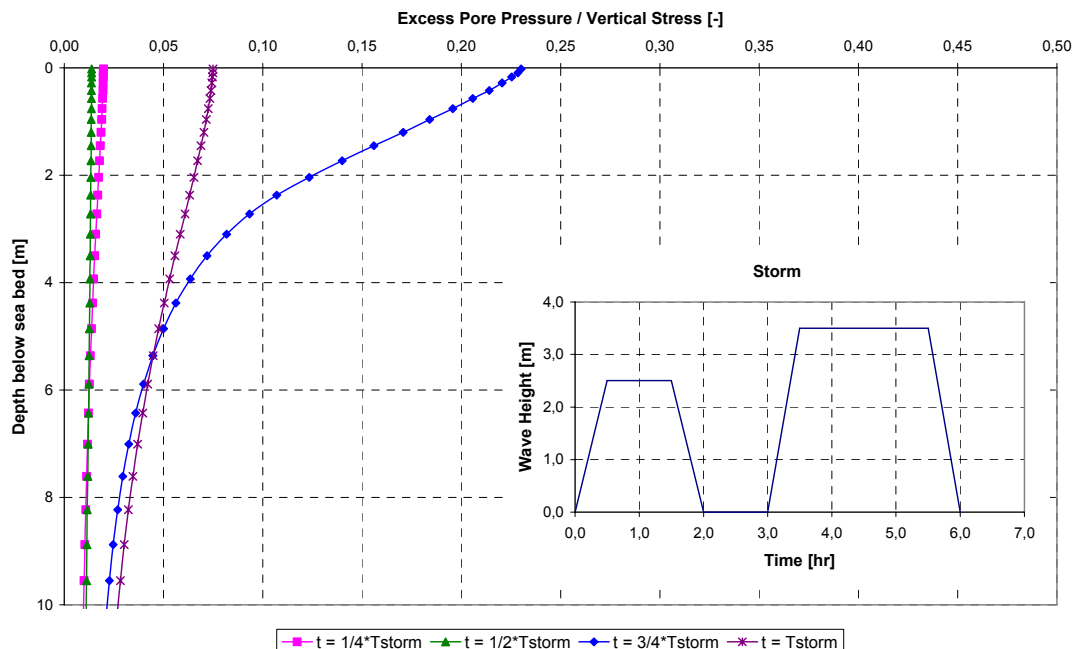


Figure 5-30: EPP-development in a slope of 1:5 in MFS with $n=0,42$ during storm type D

It can be seen that the normative wave height is increased from 3 to 3,5 m when this storm is preceded by a smaller storm. Maximum *relative* EPP's are both 0,26 and occur at nearly the same time, which is a half hour after the wave height has reached its maximum.

It can be concluded that preceding storms are particularly favourable in case of loosely packed soils, which can undergo a significant compaction (Δn). According to the same line of thought, suddenly rising storms are far more dangerous than storms with gradually increasing wave heights, especially for loosely packed soils.

5.2.4 Computations with *MCycle* for multiple layers

Another common phenomenon is the presence of multiple layers in the seabed. Freshly deposited sediments are often less densely packed and therefore more susceptible to liquefaction than older soil layers, but what is the effect on the normative storm wave height?

Therefore two extra cases are investigated, which will be compared with a seabed of homogeneous sand:

- I a 5 m thick layer of loosely packed fine sand (MFS but with $n=0,42$) on top of 15m of sand with ten times smaller hydraulic permeability k ($1,42 \cdot 10^{-7}$ m/s) and consolidation coefficient c_v ($0,0005$ m²/s) and a relative density of 0,8;
- II a 5 m thick layer of loosely packed medium sand (MMS but with $n=0,42$) on top of 15 m of sand with ten times smaller hydraulic permeability k ($8,8 \cdot 10^{-7}$ m/s) and consolidation coefficient c_v ($0,0031$ m²/s) and a relative density of 0,8;

The excess pore pressure development of case I is presented in Figure 5-31 with the remark that the y-axis depicts only the upper 10 m of the sea bed. The pore pressures have difficulties in penetrating the denser layers. When case I is compared to Figure 5-29 (the same conditions but homogeneous MFS ($n=0,42$) over the entire depth), it appears that such an impermeable lower layer does not affect the pore pressures of the upper layer much. Because of the superficial wave-impact, the reduction of pore pressure build-up in lower layers will not have any effect on slope stability: very deep failures are not likely and the probability that a shallow failure occurs remains the same.

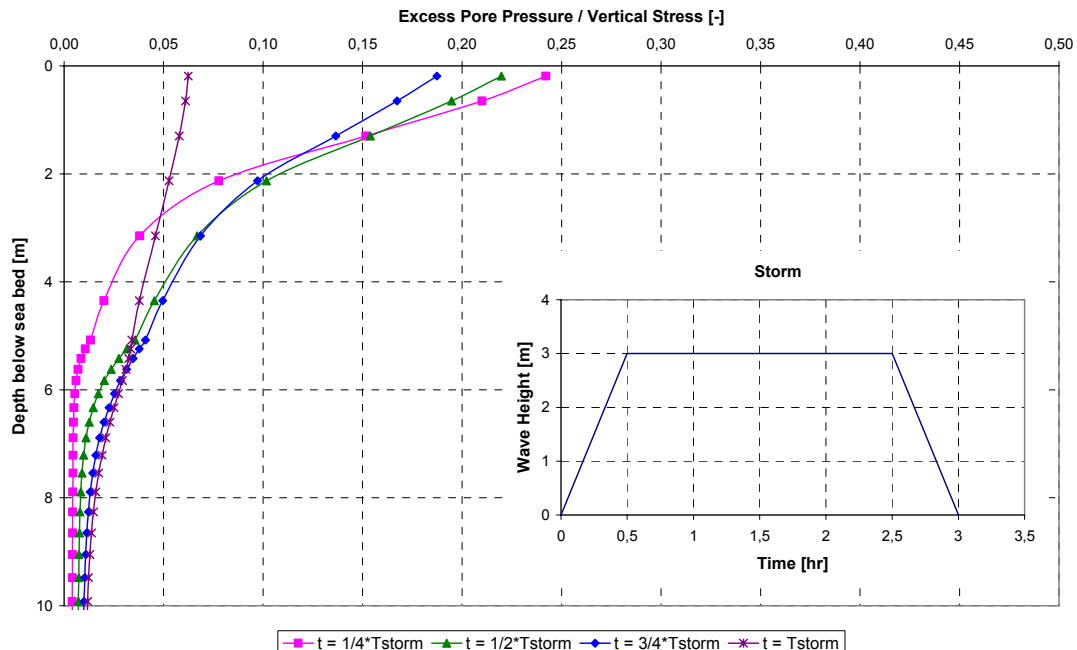


Figure 5-31: EPP development in a slope 1:5 with 2 layers: upper layer of MFS (thickness 5m) and lower layer with ten times smaller k and c_v (thickness 15m)

In Case II in the beginning of the storm similar behaviour can be observed: EPP's are less penetrating in the denser soil layer. During the storm another phenomenon can be noticed, which was already mentioned at Figure 5-24. Excess pore pressures keep increasing in the deeper parts of dense soils, because consolidation is a very slow process.

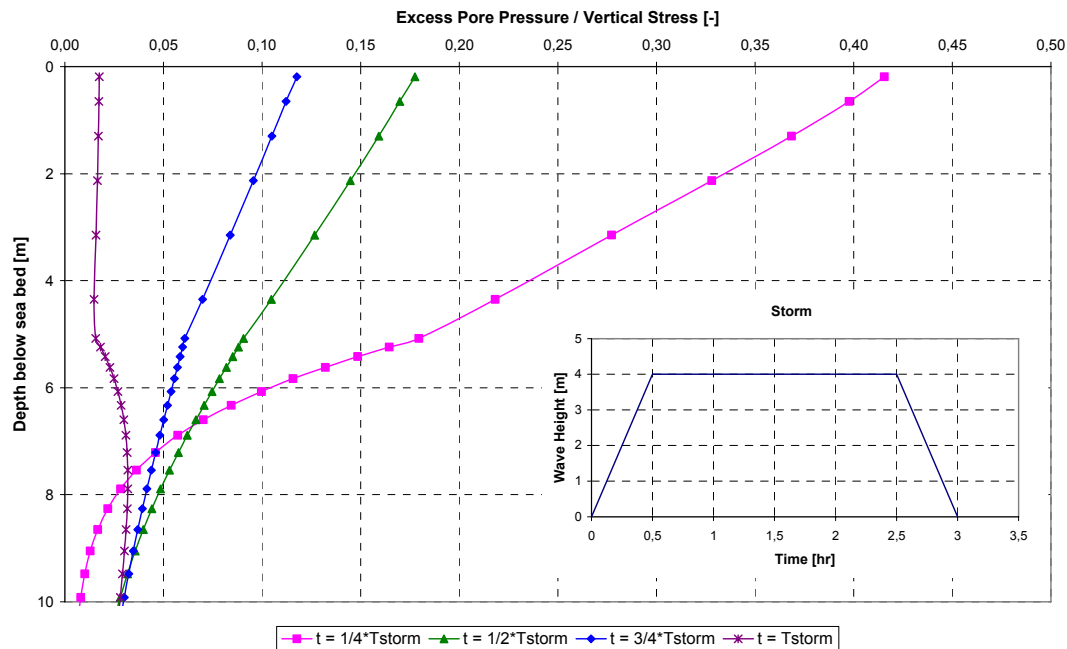


Figure 5-32: EPP development in a slope 1:5 with 2 layers: upper layer of MMS (thickness 5m) and lower layer with ten times smaller k and c_v (thickness 15m)

It could well be possible that thinner layers of loose material (about 1-2m) have a larger unfavourable effect. The consequences of various combinations of different layers have not been studied.

It is advised that, as long as precise quantities of soil that will start to flow due to cyclic liquefaction are still very hard to predict, acceptable probabilities of failure should be based on thickness of susceptible soil layers, height of the slope and the slope steepness, wave climate throughout the year from the moment the dredging works have started and the possible (economic) damage.

Summarizing all of the above mentioned calculations, it can be stated that:

- an increase of the hydraulic permeability of the soil (k) results in a smaller pressure build-up and therefore a reduction of the risk of cyclic liquefaction;
- a larger consolidation coefficient (c_v) means faster diminishing excess pore water pressures. Soils with a larger hydraulic permeability, like coarse sands, or a smaller compressibility, like dense sands, therefore are far less susceptible to liquefaction due to cyclic loading;
- although every wave cycle causes a certain compaction, soils with large relative densities show less compressibility each wave cycle than soils with small relative densities;
- the influence of waves fades out fast with increasing water depth, so the shallowest part of a slope will be normative for the risk of cyclic liquefaction;
- due to the earlier mentioned large sensitivity of EPP's between 0,5 and $1,0\sigma'_{v0}$ and consequently very fast decrease of effective stresses around failure, the actual slope steepness has little influence. Because cyclic liquefaction due to waves is caused by a large number of waves, which all have a small contribution, slope failure is very sensitive to the wave height; this means that steeper slopes in equal soils only can withstand slightly higher waves. Flattening the side slopes is therefore not a quite effective measure to increase slope stability. However, consequences of gravity flow caused by this liquefaction are greater on a steeper slope;
- small preceding storms can increase the normative wave height of the main storm by about 0,5 m, but this effect is only significant for loosely packed soils. Medium to densely packed soils are not much influenced by preceding storms;
- although not mentioned before, compaction (increase of RD) that occurred during the described storms was often in the order of 1 or 2% for loosely and medium packed soils.

Measures to reduce the risk (and consequences) of cyclic liquefaction caused by waves are:

-) adaptation of the channel geometry or alignment such that large sedimentation under calm hydrodynamic conditions is prevented or is forced to certain predestined places, like sediment pocket holes. This should prevent fast accretion of the side slopes, resulting in loosely packed sediment;
-) selection of a 'rough' dredging method which triggers regulated liquefaction flow slides or, if this does not occur, causes cyclic compaction of the soil, thereby increasing safety against liquefaction;
-) construction of discrete zones with stable material like gravel;
-) (partial) (vibro-)compaction of the soil;
-) soil improvement.

All mentioned measures are aimed at improving soil properties (hydraulic permeability k , consolidation coefficient c_v and relative density RD) of the 'danger zones'. Because of the failure mechanism of wave-induced liquefaction increasing the relative density or providing the side slopes with some drainage facilities are far better measures than reducing the slope steepness.

Finally, the difference between static and cyclic wave-induced liquefaction will be emphasized. While static liquefaction usually initiates at a location somewhere in the inner lower part of a slope, wave-induced liquefaction takes place at a shallow location at the top of the slope. The former is strongly related to the slope height, while the latter is not; only the consequences of the generated turbidity current will be larger for higher slopes, because the erosive capacities will increase when this turbidity current is accelerated. While slope flattening is an effective measure to resist static liquefaction, other measures are needed to deal with wave-induced liquefaction, aimed at improving soil properties of the upper layer.

It is therefore very well possible that a certain slope is stable when considering static liquefaction, but susceptible to wave-induced liquefaction. On the other hand, another slope may be able to resist local storm waves, but is susceptible to static liquefaction. Still, a large wave can of course act as a trigger mechanism.

5.3 Other loads

Already in Paragraph 2.1 earthquakes, tectonics and man-made loads, like dumping of waste deposits, anchor forces and thrust produced by a ship propeller, were mentioned as trigger mechanisms. None of these loads are unlikely in a navigation channel, but they are not treated elaborately in this thesis. Some loads, like anchor or propeller forces, are relatively small, but can however be just large enough to act as a releasing mechanism for liquefaction flow slides. The design approach in this thesis was aimed at avoiding slope geometries that are susceptible to such small forces under the assumption that a certain trigger mechanism will always occur. Other loads occur at totally irrelevant time scales, like tectonic movement.

Earthquakes however should not be neglected beforehand. Therefore some points of special interest are mentioned briefly. Seismically induced liquefaction is similar in some respects to the phenomenon of wave-induced liquefaction [Nataraja, 1983]. One can however distinguish three differences:

1. ocean wave periods are longer than the periods of earthquake cycles of shaking. Typical wave frequencies are about 0,1 Hz, whereas earthquake frequencies are in the order of 1 Hz;
2. ocean wave storm durations (a few hours) are considerably longer than the seismic shaking duration (about 1 minute);
3. ocean wave loadings are imposed at the surfaces of the seafloor rather than at some lower boundary conditions as in earthquakes.

When designing the slope of a navigation channel, it depends very much on the location whether one should take earthquakes into account. Because of the short timescale of one year in this thesis, it does not seem relevant to consider heavy or even very heavy earthquakes.

Probability of occurrence will be low and risk of human losses due to failure of the side slopes of a navigation channel is almost negligible. In that case we'll accept to do some repair works.

Therefore it is advised to make calculations based on two reference earthquakes:

- 1) a light earthquake (4,0 on the Richter Scale) with an accelerations of $0,1 \text{ m/s}^2$ of which a total of about 6200 take place annually on a global scale.
- 2) a moderate earthquake (5,5 on the Richter Scale) with an acceleration of $0,681 \text{ m/s}^2$ of which throughout the world about 800 earthquakes take place every year.

There are several ways to simulate earthquakes on slopes. In MStab earthquakes can be modelled by introducing a constant horizontal or vertical acceleration (a constant times g). This approach only accounts for the direct, elastic effect (cf wave loads). The indirect, plastic effect asks for an approach which takes compaction and consolidation into account. A free software package is Cyclic1D by Elgamal, Yang, Parra and Ragheb, which can be downloaded from the internet (<http://cyclic.ucsd.edu>). Various earthquakes from the past or a theoretical sinusoidal motion can be selected. A more complete and expensive software package is for instance Liquefy Pro by CivilTech Software which evaluates liquefaction potential and earthquake-induced settlement in saturated soils (sand, silt and gravel).

Both programs give insight in the earthquake behaviour of the side slopes and a quick investigation is therefore recommended in seismic areas.

In the past a lot of research has been done on earthquake-induced liquefaction, especially in areas with frequent earthquakes like Japan and the United States of America. Research in these countries usually is fundamentally different from the approach of static liquefaction. Due to the frequent occurrence of earthquakes, cyclic earthquake-induced liquefaction gets a lot of attention. At different void ratios the steady state is determined (the bend in line C of Figure 4-13). When the void ratios of different tests are plotted against the 'steady state' isotropic stresses, a logarithmic relationship appears, see Figure 5-33 for an example. In-situ stresses and void ratios are compared with this steady-state line to get an estimation of the susceptibility to liquefaction. If in-situ points are above the steady-state line, there is a risk of liquefaction.

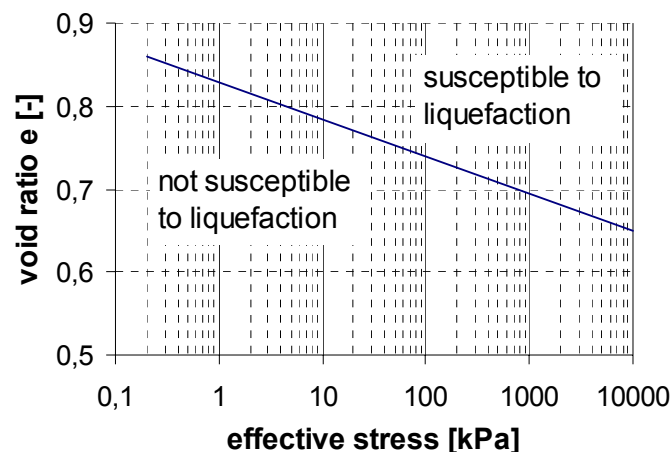


Figure 5-33: Example of a steady-state line

From definitions can be deduced that steady state values are between dry and wet critical densities, although this steady state is often confused with the wet critical density, which isn't well established abroad. An important advantage of this steady-state line over the wet critical density is the fact that tests are done with multiple isotropic stresses. Initial liquefaction is also often defined as the state in which either:

- the pore water pressure builds up to a value equal to the initially applied confining pressure or
- axial strains of about 5% in amplitude occur.

6 Hydrodynamics in trenches and channels

6.1 Currents

When currents pass over a pipeline trench or navigation channel, there are two ways to describe the flow velocity profiles, primarily depending on the bed level gradients. In this thesis a flat surrounding seabed is assumed, where the flow is stationary and friction dominated ('wall flow'), so the channel geometry itself determines which mathematical model of the flow velocity profiles inside the channel has to be applied. The above mentioned bed level gradients are therefore represented by the side slopes and the two mathematical models are:

- gradually varying flows: the flow velocity profiles can be represented sufficiently well by logarithmic profiles;
- complicated flows: flow velocity profiles strongly deviate from the logarithmic profiles and even flow reversal may occur on steep side slopes.

The properties of both models will be examined in the next two paragraphs.

6.1.1 Static wall flow

First, the situation of static wall flow will be considered. The flow velocities are dominated by friction (Formula of Chézy), the velocity profile is logarithmic and there is no flow separation. This situation is representative for slowly varying currents over gentle slopes (approximately not steeper than 1:5-7, depending on the flow conditions and slope height). The logarithmic velocity profile can be represented by:

$$u(z) = u_{h,e} \frac{\ln\left(\frac{z}{z_0}\right)}{\ln\left(\frac{h}{z_0}\right)} \quad \text{Eq. 6-1}$$

in which

z_0 = zero-velocity level = $0,03 k_r$ (Nikuradse, hydraulic rough bottom);

k_r = equivalent bed roughness;

$u_{h,e}$ = equilibrium flow velocity at water surface ($z=h$).

This equilibrium flow velocity $u_{h,e}$ is related to the depth-averaged flow velocity, which is equal to the flow velocity at about 0,4 times the water depth:

$$u_{h,e} = \frac{\ln\left(\frac{h}{z_0}\right)}{-1 + \ln\left(\frac{h}{z_0}\right)} \bar{u} \quad \text{Eq. 6-2}$$

The bed-shear velocity (u_*) is determined from the flow velocity computed at a small height above the seabed (SUTRENCH: $0,05h$), assuming a logarithmic profile in the near-bed layer.

$$u_{*e} = \frac{\kappa}{-1 + \ln\left(\frac{h}{z_0}\right)} \bar{u} \approx \frac{\sqrt{g}}{C} \bar{u} \quad \text{Eq. 6-3}$$

in which

κ = constant of Von Karman $\approx 0,4$

$$C = \text{Coefficient of Chezy} = 18 \log \left(\frac{12h}{k_r} \right)$$

When a current enters a channel, current velocities, directions and bed shear stresses will change, depending on the alignment of the channel with respect to the current direction. Subsequently a perpendicular, parallel and oblique incoming current will be discussed.

Perpendicular current

In many cases, a current will be more or less perpendicular to the channel axis (e.g. navigation channel perpendicular to a tidal current along the coast). If the water depth increases, the flow will slow down, because of continuity (subscript '0' means 'at surrounding bed', while '1' means 'inside channel'):

$$\bar{u}_1 h_1 = \bar{u}_0 h_0 \Rightarrow \bar{u}_1 = \frac{h_0}{h_1} \bar{u}_0 \quad \text{Eq. 6-4}$$

Parallel current

The opposite will occur with a current parallel to the channel axis. Because of the equal water surface slopes 'i' inside and outside the channel, the driving force ($= \rho_w g h \partial h / \partial x = \rho_w g h i$) is the same. Because this driving force is balanced in the equilibrium situation by the bottom friction force ($\rho_w g v |v| / C^2$), the following set of equations can be obtained:

$$\left. \begin{aligned} \tau_{b;0} = \rho_w g h_0 i_0 = \rho_w g \frac{v_0 |v_0|}{C_0^2} &\Rightarrow i_0 = \frac{v_0 |v_0|}{h_0 C_0^2} \\ \tau_{b;1} = \rho_w g h_1 i_1 = \rho_w g \frac{v_1 |v_1|}{C_1^2} &\Rightarrow i_1 = \frac{v_1 |v_1|}{h_1 C_1^2} \end{aligned} \right\} i_0 = i_1 \quad \text{Eq. 6-5}$$

(note: flow velocity 'v' is directed along y-axis, parallel to channel axis)

If the coefficient of Chézy is considered depth-dependent, the horizontal flow velocity can be described as:

$$\bar{v}_1 = \sqrt{\frac{h_1}{h_0}} \frac{C_1}{C_0} \bar{v}_0 = \sqrt{\frac{h_1}{h_0} \frac{\ln \frac{12h_1}{k_r}}{\ln \frac{12h_0}{k_r}}} \bar{v}_0 \quad \text{Eq. 6-6}$$

A first approximation, assuming a constant coefficient of Chézy, results in:

$$\frac{\bar{v}_1}{\bar{v}_0} = \sqrt{\frac{h_1}{h_0}} \quad \text{Eq. 6-7}$$

However, this coefficient of Chézy will increase inside the channel: at a larger depth, the bed will be less 'felt', resulting in a larger coefficient of Chézy, representing a smoother bottom. The above equation can well be approximated by a power law function like:

$$\frac{\bar{v}_1}{\bar{v}_0} \approx \left(\frac{h_1}{h_0} \right)^p \quad \text{Eq. 6-8}$$

in which p is not longer 1/2 (Eq. 6-7), but a constant between 0,61 and 0,65 for water depths h_0 between 5 and 20 m and bed roughness k_r between 0,01 and 0,1 m. In Figure 6-1, this coefficient 'p' is 0,63 for $h_0 = 10$ m and $k_r = 0,03$ m.

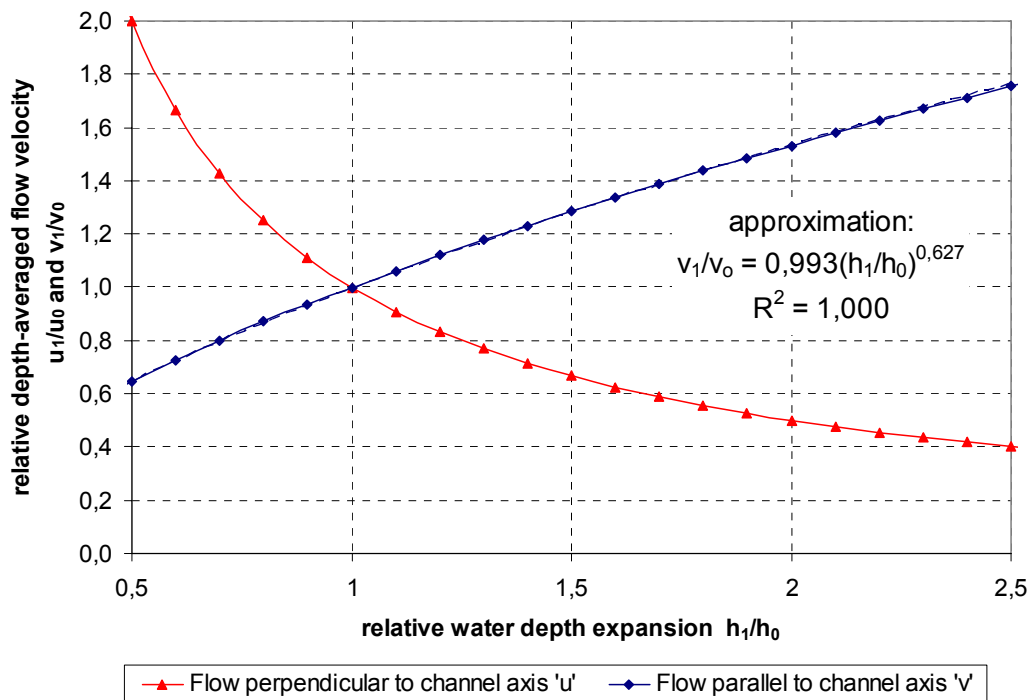


Figure 6-1: relative flow velocities of perpendicular and parallel currents against relative water depths

Due to the increasing flow velocity and, as will be explained later on, the sediment transport capacity, parallel currents are theoretically capable of 'self-cleansing' of the trench, but because of gravity effects, the side slopes will flatten and particles will settle on the bottom of the channel. Under normal conditions self-cleansing will therefore not be observed in nature.

Oblique current

Currents that are oblique to the channel axis give a more complex situation. The perpendicular component is reduced due to continuity, the parallel component is increased because of the equal water surface slope (or driving force) inside and outside the channel. If both components are added as vectors, it will become clear that the current will refract inside the channel. Because of continuity between two (refracted) streamlines, the following relation holds:

$$\frac{\bar{V}_1}{\bar{V}_0} = \frac{b_0}{b_1} \frac{h_0}{h_1} \quad \text{Eq. 6-9}$$

This equation tells that if the depth ratio h_1/h_0 is larger than the ratio of the width between two streamlines, b_0/b_1 , the flow will slow down, otherwise the flow will increase.

Roughly speaking, there are two fundamentally different methods to calculate the current velocity inside the channel. The first method, which is applicable for large angles α_0 , assumes a constant component of the current velocity parallel to the channel axis, based on the idea that large velocity differences cannot exist over very short distances. It can be deduced that:

$$\text{Method I:} \quad \bar{V}_1 = \bar{V}_0 \sqrt{\cos^2 \alpha_0 + \left(\frac{h_0}{h_1} \right)^2 \sin^2 \alpha_0} \quad \text{in which } \bar{V} = \sqrt{\bar{u}^2 + \bar{v}^2} \quad \text{Eq. 6-10}$$

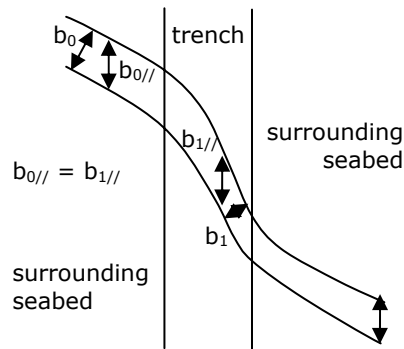


Figure 6-2: Schematization of refracted streamlines when crossing a trench

The second method which is applicable for small approach angles to the channel axis assumes a constant water surface slope in the direction of the channel axis (longitudinal component of current). The current velocity in the channel now becomes:

$$\text{Method II: } \bar{V}_1 = \bar{V}_0 \sqrt{\left(\frac{\ln \frac{12h_1}{k_r}}{\ln \frac{12h_0}{k_r}} \right)^2 \left(\frac{h_1}{h_0} \right) \cos^2 \alpha_0 + \left(\frac{h_0}{h_1} \right)^2 \sin^2 \alpha_0} \quad \text{Eq. 6-11}$$

Method II yields higher values for the current velocity than Method I, especially for small approach angles, but these angles actually are outside the range of Method I, see Figure 6-3. With increasing approach angles both methods more and more coincide with each other: they predict equal velocities in case of a perpendicular current. Approach angles of 45-90° are best calculated by Method I. Experiments of Wallingford (1973) and WL|Delft Hydraulics (1983) [Van de Graaff, 2003] confirmed the applicability of Method I for large approach angles and Method II for small approach angles. The transition between the two methods can not (yet) be described analytically and is just bounded by the upper limit (Method II) and lower limit (Method I).

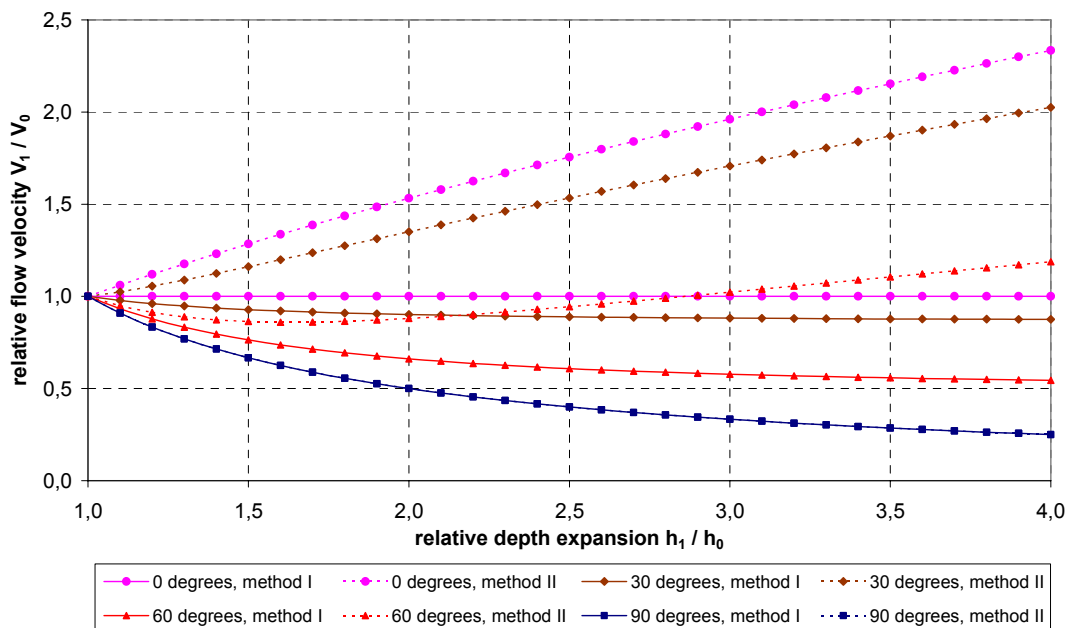


Figure 6-3: Current velocity inside channel against channel depth

In Jensen et al. [1999], a somewhat other approach is used. Also the different development of the cross-channel and longitudinal component is considered. When a current enters a channel, the depth-averaged velocity will be refracted to an equilibrium velocity and flow angle. This refraction is caused by:

- an instantaneous decrease in the perpendicular 'u'-component with increasing depth
- a gradual increase in the parallel 'v'-component

The slow development of the longitudinal velocity component is dependent on the angle of the incoming current α_0 and the relative depth increase h_1/h_0 . Especially for large flow angles the time to cross the channel is so small that the flow cannot adapt before reaching the downstream side slope.

Not the assumption of equal longitudinal flow velocities or equal water surface slopes inside and outside the channel is taken as a starting point, but equal driving forces ($\delta p/\delta y$) in longitudinal direction outside the channel and inside the channel in the equilibrium situation (fully adapted and refracted flow).

$$\left. \begin{aligned} \tau_{by0} &= \tau_{b0} \cos(\alpha_0) = -h_0 \frac{\delta p}{\delta y} \\ \tau_{bye} &= \tau_{beq} \cos(\alpha_{eq}) = -h_1 \frac{\delta p}{\delta y} \end{aligned} \right\} \begin{aligned} \tau_{beq} \frac{\cos(\alpha_{eq})}{\cos(\alpha_0)} &= \frac{h_1}{h_0} \end{aligned} \quad \text{Eq. 6-12}$$

In the equilibrium situation with fully developed flow, the continuity equation in cross-channel direction still holds:

$$V_0 \sin(\alpha_0) h_0 = V_{eq} \sin(\alpha_{eq}) h_1 \quad \text{Eq. 6-13}$$

When equations 6-12 and 6-13 are combined with the expression for the bed shear stress, according to Chézy's Law ($\tau = \rho_w g V^2 / C^2$), the following analytical expression for the equilibrium angle can be found, which can be substituted into Eq. 6-13 to obtain the equilibrium flow velocity.

$$\cos(\alpha_{eq}) = \sqrt{\left(\frac{1}{2} \left(\frac{C_0}{C_1} \right)^2 \left(\frac{h_0}{h_1} \right)^3 \frac{\sin^2(\alpha_0)}{\cos(\alpha_0)} \right)^2 + 1} - \frac{1}{2} \left(\frac{C_0}{C_1} \right)^2 \left(\frac{h_0}{h_1} \right)^3 \frac{\sin^2(\alpha_0)}{\cos(\alpha_0)} \quad \text{Eq. 6-14}$$

The results of Eq. 6-14 are plotted for different incoming flow angles against the relative depth expansion in Figure 6-4. It can be seen that currents with incoming flow angles smaller than 60° eventually result in an increase of the flow velocity. For larger incoming flow angles this depends on the relative depth expansion. A larger depth expansion reduces the cross-channel component and increases the longitudinal component.

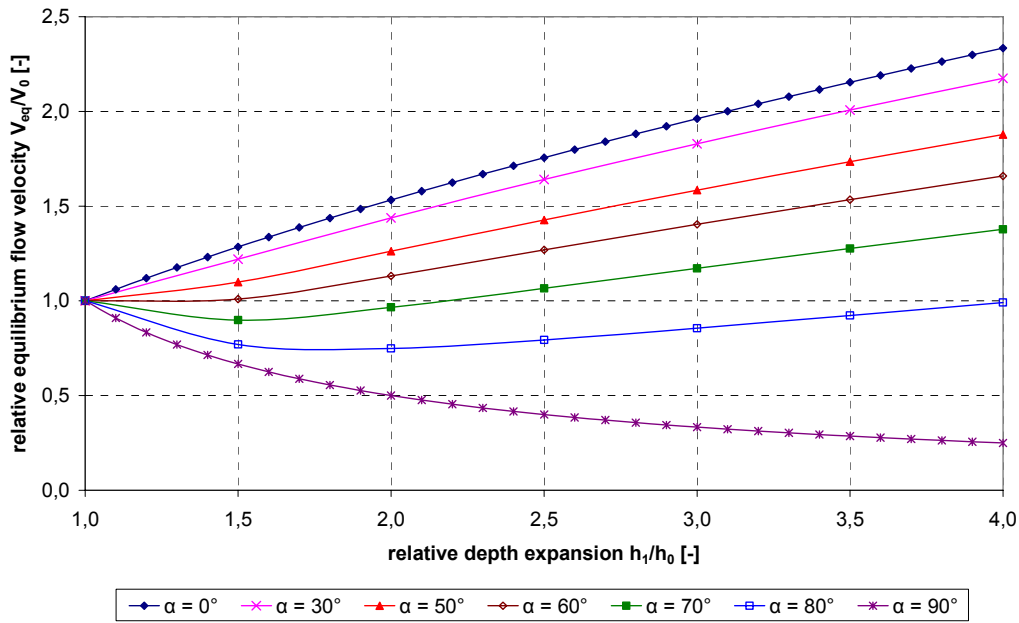


Figure 6-4: The relative equilibrium flow velocity against incoming flow angles

The above approach yields an analytical solution, with only the assumption of the empirical coefficient of Chézy. However, the acceleration of the longitudinal velocity component proceeds so slow, that in most practical situations one of the earlier mentioned phenomena will dominate:

- for large incoming flow angles the total depth-averaged flow velocity will decrease due to continuity;
- for small incoming flow angles the total depth-averaged flow velocity will increase;

To get an impression of the relevant time and space scales, the dimensionless timescale for the adaptation process is presented without the full mathematical derivation [Jensen et al., 1999]:

$$t = \frac{h_0}{V_0} \frac{1}{2\sqrt{ae}} \ln \left(\frac{\sqrt{\frac{a}{e} + \frac{v}{V_0}}}{\sqrt{\frac{a}{e} - \frac{v}{V_0}}} \right) \quad \text{Eq. 6-15}$$

in which

$$a = \frac{g \cos(\alpha_0)}{C_0^2}$$

$$e = \frac{gh_0}{h_1 C_1^2 \sqrt{1 + \tan^2(\alpha)}}$$

The acceleration of the longitudinal component v is in fact an infinite process, but if we are interested in the time it takes to reach, let's say, 95% of the equilibrium value v_{eq} , the expression becomes:

$$\Delta t = t(0,95 * v_{eq}) - t(v_0) \quad \text{Eq. 6-16}$$

Expression 6-15 is a function of longitudinal component v and related flow angle α . This flow angle α changes throughout the flow refraction process. The upper estimate of the timescale can be found, when angle α is taken just inside the navigation channel (continuity taken into account ($u_1 = u_0 h_0 / h_1$), but before longitudinal adaptation); a lower estimate can be found when α_{eq} is used. The 'real' timescale lies somewhere in between, so the average of both angles will be used. The time Δt has been calculated for different incoming flow angles (constant bed roughness $k_r = 0,03$, surrounding water depth $h_0 = 10\text{m}$). The results are very much in agreement with values reported by Jensen et al. [1999], although they used the Colebrook-White friction formula and a depth-independent bed roughness.

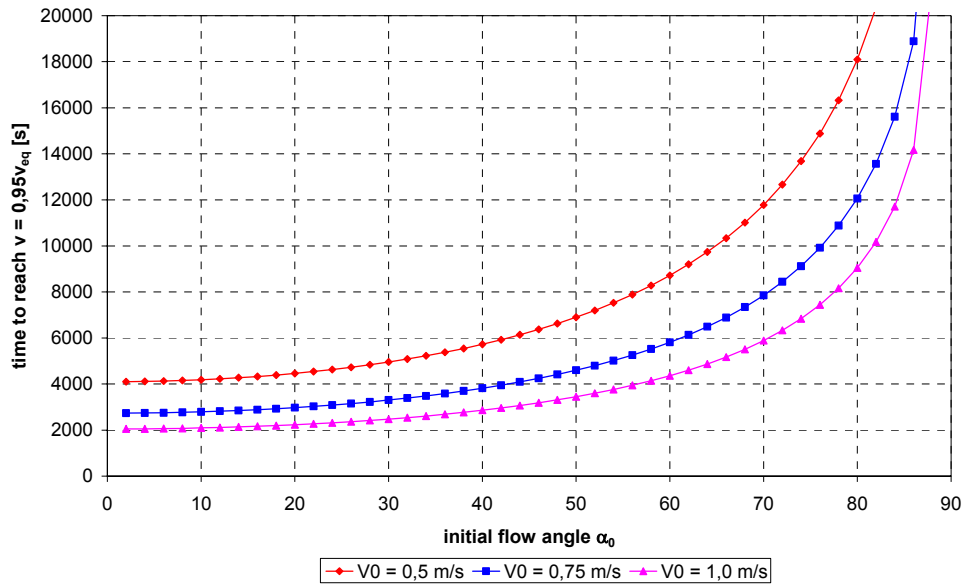


Figure 6-5: Time to reach $0,95v_{eq}$ plotted against incoming flow angle; $h_1/h_0 = 2$

In Figure 6-5 can be observed that, first of all, the longitudinal acceleration is a very slow process and, second, that this adaptation time increases very fast for incoming flow angles larger than 60° . When this graph is converted to the cross-channel distance (Figure 6-6), it appears that the magnitude of the current velocity does not affect this cross-channel distance. Therefore now, three relative depth expansions have been plotted instead of different flow velocities. The influence of the relative depth expansion however is minor.

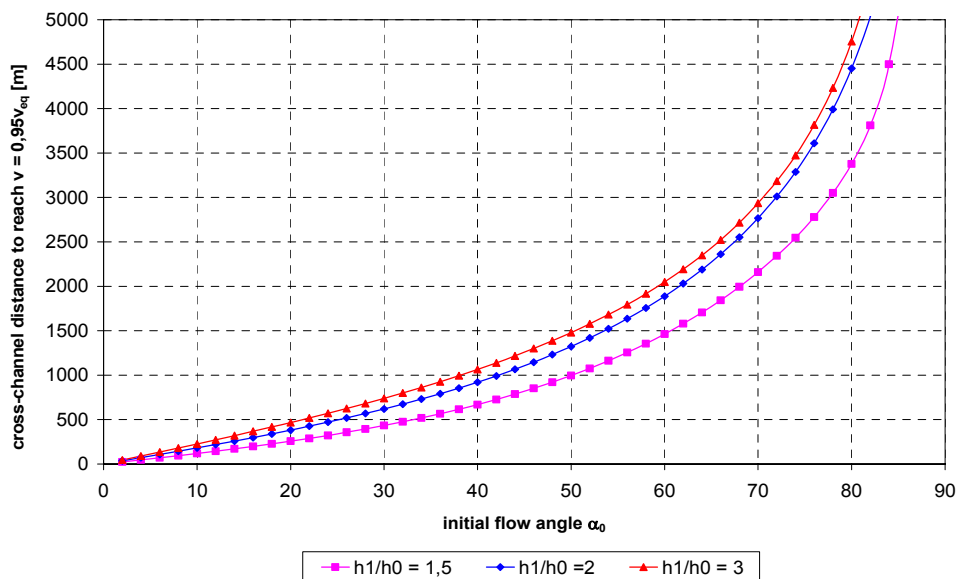


Figure 6-6: Cross-channel distance to reach $0,95v_{eq}$ plotted against incoming flow angle

These charts again demonstrate that currents with a large angle to the channel axis are dominated by a decrease of velocity due to continuity, because the width of navigation channels will be in the order of a few hundred meters. Currents with small incoming flow angles, say 30° , will almost fully adapt to the equilibrium flow velocity and angle.

6.1.2 Reynolds-averaged Navier-Stokes equations

If side slopes become steeper (roughly 1:5) or one is particularly interested in the detailed modelling of refraction of oblique currents, the full 3D flow problem needs to be solved. The well-known computer model *Delft3D* developed by Delft Hydraulics solves this problem under the shallow water assumption, which means that the vertical momentum equation is reduced to a hydrostatic pressure equation. Sudden variations in the bottom topography, like very steep side slopes, can still not be taken into account correctly. Nevertheless, this model is much more capable to deal with non-stationary problems and non-logarithmic velocity profiles. The fact that more complex velocity profiles can be calculated does not necessarily result in a more accurate prediction of sediment transport and morphological development, because sediment transport formulae often are calibrated to logarithmic velocity profiles. Only when the complete distribution of suspended sediment over the entire vertical profile can be obtained with good resemblance to field measurements, this better description of the velocity profile indeed yields better results in predicting sediment transport.

Nevertheless a very nice attempt to solve this problem has been made by several PhD-students of the *Institute of Hydrodynamics and Water Resources, Technical University of Denmark* [Jensen, 1999]. They solved the 3D-Reynolds-averaged Navier-Stokes equations using a K- ϵ turbulence closure model (representing the transport equations for turbulence kinetic energy K and dissipation rate ϵ) in a curvilinear coordinate system (without the restriction of hydrostatic pressure), so they were able to describe the two important flow features, when crossing a navigation channel obliquely:

- the flow will be refracted in the direction of the channel alignment, which can be described by depth-averaged models;
- a secondary flow will develop caused by shear in the velocity profile.

Without discussing the mathematical equations some of the described, instructive phenomena are presented. The above mentioned secondary flow is caused by non-equilibrium between the pressure gradient and the centrifugal forces. These centrifugal forces are unevenly distributed over the vertical velocity profile. The same phenomenon is observed in river bends (helical motions, see *River Dynamics*).

With very steep side slopes of $\pi/6:1$ (about 1:1,9) the flow is represented in Figure 6-7 for incoming flow angles of 30° and 60° . Three streamlines, representing paths of three fluid particles, are plotted: one just above the bed, one at mid-depth and one near the water surface. Because of the formation of a separation bubble, also a fluid particle within this separation zone is followed. Because this particle is trapped within this bubble, it is carried downstream by the longitudinal velocity component in a corkscrew pattern. Also it can be observed that the near-bed particles experience the largest refraction.

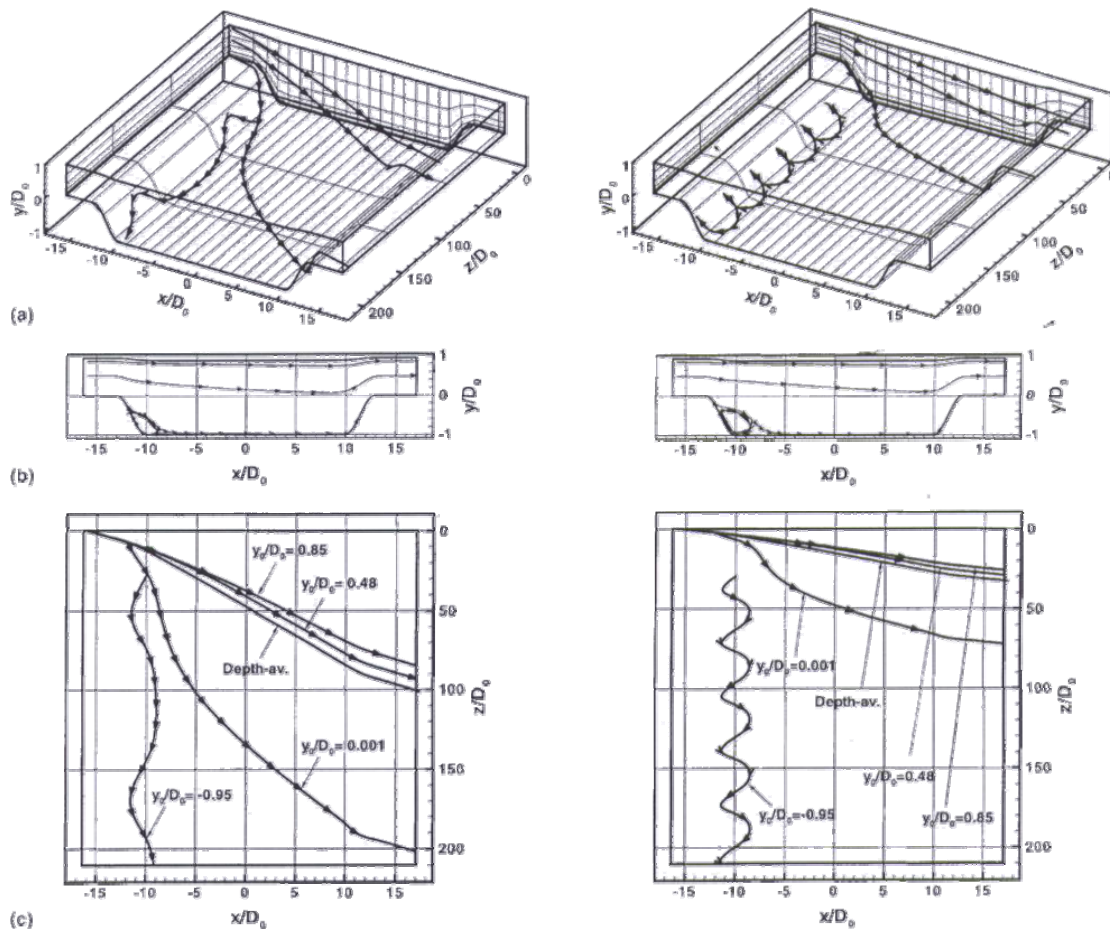


Figure 6-7: Visualisation of refracted streamlines at three different depths and inside the separation zone [source: Jensen, 1999]

When a current enters a navigation channel at the upstream slope, the near-bed fluid particles experience a larger deceleration than near-surface particles due to the positive pressure gradient on the upstream slope affecting low inertia particles more strongly (note the logarithmic velocity profile in Figure 6-8) resulting in a larger refraction (smaller flow angle) of the near-bed flow. When the current crosses the flat channel bottom, this difference in flow angles is reduced due to shear in the velocity profile. On the downstream slope, the current accelerates again (large negative $\delta p/\delta x$) and the near-bed particles undergo a larger velocity increase than the near-surface particles: now the current angle of the upper layers will be larger than the depth-averaged flow angle.

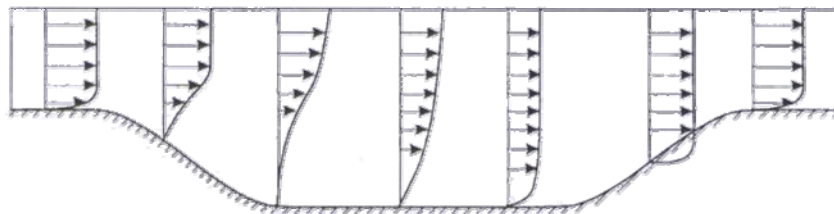


Figure 6-8: Velocity profiles in trenches with steep side slopes

These observations are demonstrated in Figure 6-9 for two channel widths $W=20h_0$ and $W=100h_0$. The variation of the current refraction starts at the side slopes and is reduced at the channel bottom due to shear in the velocity profile. If the channel becomes wider, the flow angle difference over the vertical will reduce to zero. Please note, firstly, the flow angle's tendency to decrease inside the channel due to adaptation of the longitudinal velocity component and, secondly, the somewhat smaller flow angle (43° and 40°) after crossing the channel.

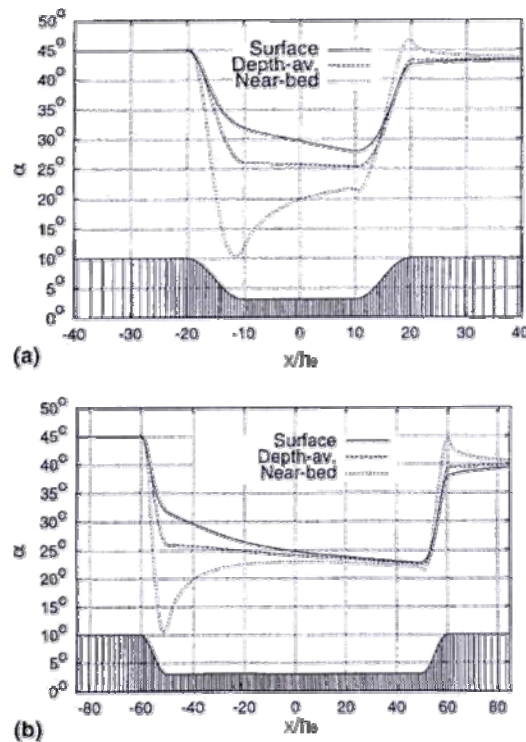


Figure 6-9: Flow angle development against cross-channel dimensionless distance for $\alpha_0=45^\circ$, $h_1/h_0=2$, side slopes 1:6, (a) $W=20h_0$ and (b) $W=100h_0$;

This variation of the current refraction over the vertical is considered as a secondary motion on top of the depth-averaged flow. The difference between angles of bed-shear stress and depth-averaged flow angle ($\Delta\alpha$) may become as large as 20-30°, see Figure 6-10. In general, this difference is dependent on:

- steepness of side slopes, $1/\tan(\beta)$
- expansion in depth, h_1/h_0
- incoming flow angle, α

An increase of the above parameters means a larger secondary motion, with the remark that currents with a flow angle of 80-90° however can experience separation bubbles, but will show little secondary motion because of the small longitudinal velocity component. Increasing the incoming flow angle, in fact, results in a steeper slope from the point of view of the current. A steeper slope means the distance over which the streamlines must bend will be shortened due to the larger pressure gradient $\delta p/\delta x$.

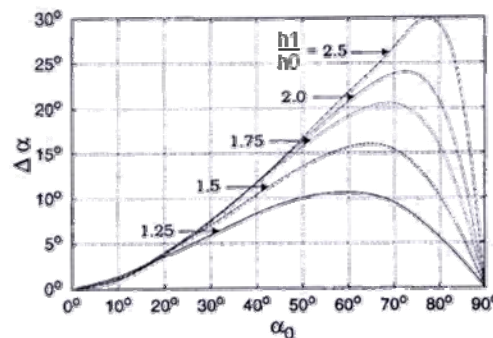


Figure 6-10: Difference between angles of bed shear stress and depth-averaged flow angle for $W=20h_0$ and side slopes 1:6

Furthermore, it can be concluded from Jensen [1999] that for channels with a width of about $20h_0$ and side slopes of 1:6 an increase of the depth-averaged flow velocity only occurs for incoming flow angles smaller than 30°. Otherwise the width is simply too small to adjust to the longitudinal velocity component.

The above mentioned phenomena are typical for large-scale situations. Although not much research has been done on very small channels, like pipeline trenches, some things can be learned from the 'larger-scale theory'. Currents crossing small trenches cannot adapt to the longitudinal velocity component. If the width becomes too small and the side slopes are very steep, the streamlines aren't able to 'reattach' to the seabed, because of the presence of a very turbulent recirculation zone. In this case the current will be 'blown' over the trench and there will be hardly any sediment exchange between the upper layer and the recirculation zone.

6.2 Waves

Waves were already mentioned as a trigger mechanism of static liquefaction flow slides and as a cyclic load, or in fact as a reduction of strength, inducing dynamic liquefaction flow slides. It was assumed that the largest occurring waves are depth-limited. This means that the irregular wave field has become more complicated because the highest waves are truncated down. On the basis of two dimensionless parameters (H/gT^2 and h/gT^2), it was noticed that these large waves should be described by cnoidal wave theory. Wave pressures at the seabed, however, could well be estimated by using linear wave theory. Because eventually the impact of waves on sediment transport has to be calculated, one is again bound to linear wave theory. On the other hand, the more frequent smaller waves resemble sinusoidal waves better.

The most important phenomena that occur when a wave field passes over a channel are shoaling, refraction and reflection, see Figure 6-11, depending on the incoming wave angle.

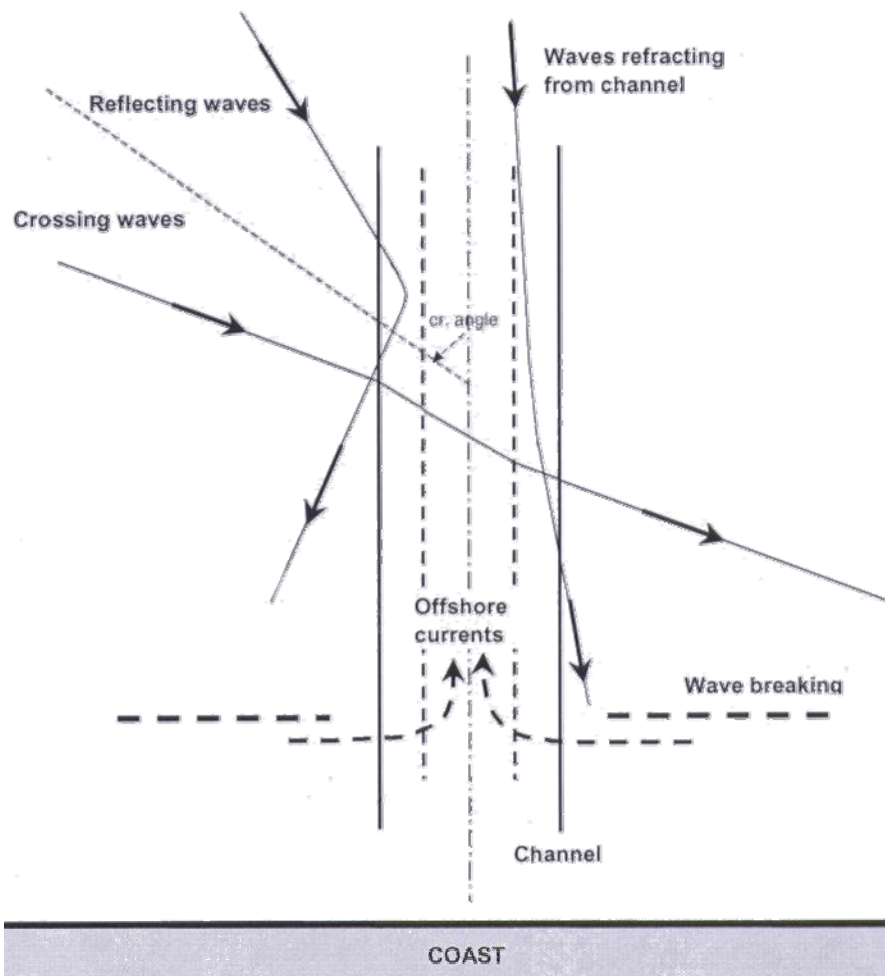


Figure 6-11: Schematization of wave phenomena in channels

Perpendicular waves

When waves are propagating perpendicular to the channel axis, the most important effect will be the adaptation of the wave height to the increased water depth. Most of the times this will result in a small decrease in wave height, but that does not necessarily occur for every wave period. The wave energy flux U will be equal inside and outside the channel:

$$U = Ec_g = Enc = \frac{1}{8} \rho_w g H^2 \cdot \left[\frac{1}{2} + \frac{kh}{\sinh 2kh} \right] \cdot \frac{gT}{2\pi} \tanh kh \quad \text{Eq. 6-17}$$

in which

- E = wave energy per unit surface area
- c_g = wave group velocity
- c = wave velocity
- n = ratio between c_g and c

The wave propagation velocity c will increase when the water depth increases, but the wave group velocity can still decrease, depending on the wave period T and expansion in water depth h_1/h_0 . If the wave energy flux inside and outside the channel are equated, the following expression for the 'inversed' shoaling factor is found:

$$K_H = \frac{H_1}{H_0} = \sqrt{\frac{\tanh k_0 h_0 \left(1 + \frac{2k_0 h_0}{\sinh 2k_0 h_0} \right)}{\tanh k_1 h_1 \left(1 + \frac{2k_1 h_1}{\sinh 2k_1 h_1} \right)}} \quad \text{Eq. 6-18}$$

This expression is plotted for different wave periods and depth expansions in Figure 6-12. One can notice that especially for small wave periods, an increase of the wave height inside the channel is possible, but in most cases the wave height will be reduced with 10-20%. This seeming inconsistent behaviour can be explained by the fact that shorter waves are more looking like 'deep' water waves than longer waves. Shorter waves will therefore experience only a minor velocity increase, because the depth expansion is less felt, while the wave group velocity even decreases, resulting in a larger wave height inside the channel.

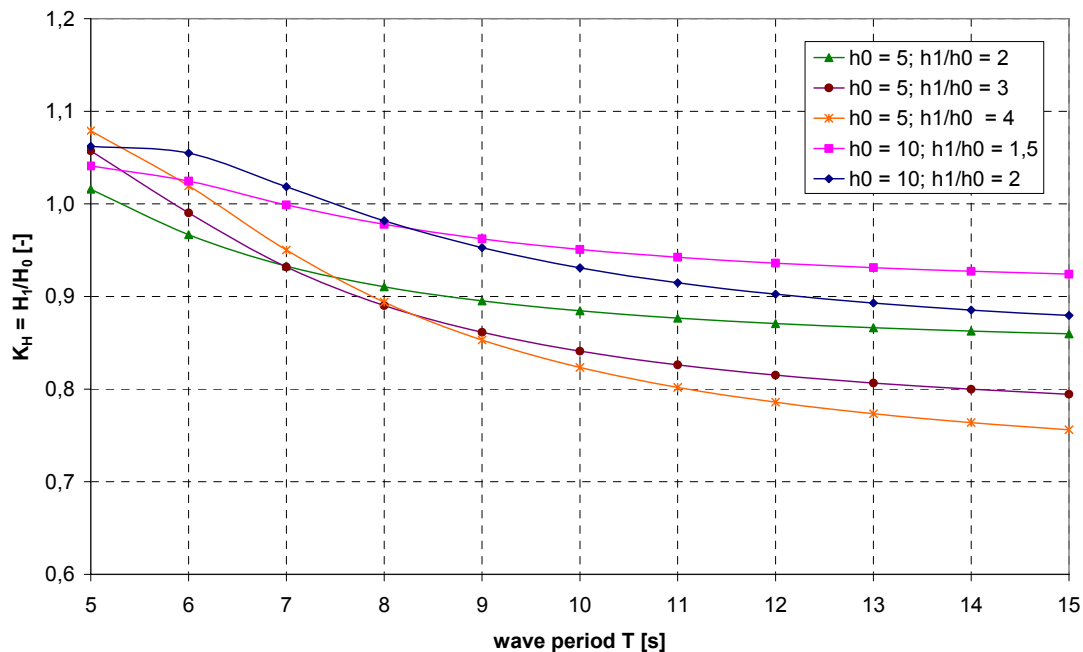


Figure 6-12: 'Inverse' shoaling factor for different water depths and expansions against wave period

At very steep side slopes wave reflection can occur as a secondary effect besides the above mentioned "inverse" shoaling effect.

Parallel waves

Waves that are propagating parallel to the channel axis will primarily experience the adaptation of the wave height to the increased water depth. At the side slopes wave diffraction occurs due to the larger wave velocities inside the channel, resulting in curved wave crests. This diffractive effects reduce wave energy at the side slopes.

Oblique waves

Waves directed oblique to the channel axis yield the most complex pattern. Waves will experience "inverse" shoaling and refraction effects, depending on the incoming wave angle α_{w0} , the wave period T , the surrounding water depth h_0 and the relative depth expansion h_1/h_0 .

The underlying concept is conservation of wave energy flux between two wave rays:

$$Ub = Ec_g b = Encb \quad \text{Eq. 6-19}$$

At a way similar to perpendicular waves, the ratio between wave height inside and outside the channel can be deduced:

$$\frac{H_1}{H_0} = \sqrt{\frac{n_0 c_0 b_0}{n_1 c_1 b_1}} = K_H \sqrt{\frac{b_0}{b_1}} \quad \text{Eq. 6-20}$$

Snel's Law is used to calculate the refracted wave angle inside the channel:

$$\frac{\cos \alpha_{w0}}{\cos \alpha_{w1}} = \frac{c_0}{c_1} \quad \text{Eq. 6-21}$$

in which α_{w0} is the angle between the wave propagation and the channel axis. Because the longitudinal distance between two wave rays remains constant, the following relation of the (orthogonal) distances between two wave rays holds:

$$\frac{\sin \alpha_{w0}}{\sin \alpha_{w1}} = \frac{b_0}{b_1} \quad \text{Eq. 6-22}$$

Just as with rays of light there is a certain critical incoming angle which forms the transition between reflected waves and refracted waves. Waves entering the channel under the critical angle will be refracted parallel to the channel axis and will therefore not be able to cross the channel. This critical flow angle can be determined from Snel's Law, assuming a refracted wave ray that is completely parallel to the channel axis, so $\cos(\alpha_{w1})=1$:

$$\cos \alpha_{w0;crit} = \frac{\tanh k_0 h_0}{\tanh k_1 h_1} \quad \text{Eq. 6-23}$$

In Figure 6-13 can be observed that this critical incoming wave angle shows a large variation for different wave periods, water depths and relative depth expansions. Waves with an approach angle larger than this critical wave angle will refract, others will reflect.

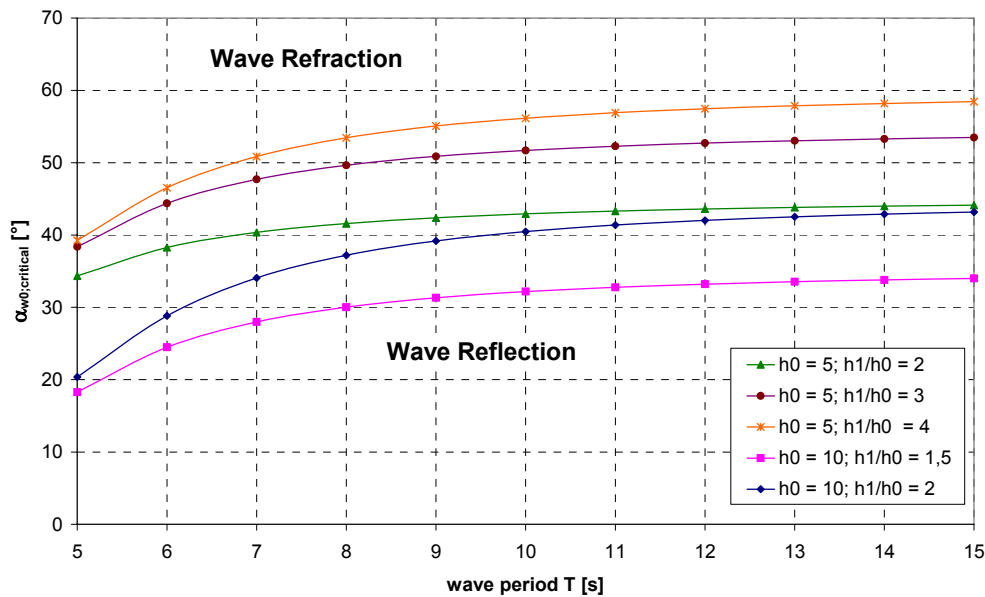


Figure 6-13: Critical incoming wave angles for different water depths and relative depth expansions

The ratio of wave heights inside and outside has been calculated for three wave periods ($T = 5, 10$ and 15 s) and for both the 'navigation channel', see Figure 6-14, and the 'pipeline trench', Figure 6-15. Please note the different y-axis: the left axis represents the wave height ratio inside and outside the channel; the right axis represents the refracted wave angle inside the channel or trench. Equal colours and markers represent equal wave periods. If the angle of the incoming waves is only slightly larger than the critical wave angle, the waves will seriously refract and the width between two streamlines become so small that an increase of the wave height inside the channel can be observed. Enlarging the wave angle always reduces the wave height ratio H_1/H_0 , but for a wave period of 5s, this ratio remains larger than 1. So for very short waves, always a minor increase of the wave height occurs. Maximum wave height reduction of up to about 10% occurs for medium to large period waves with large incoming flow angles.

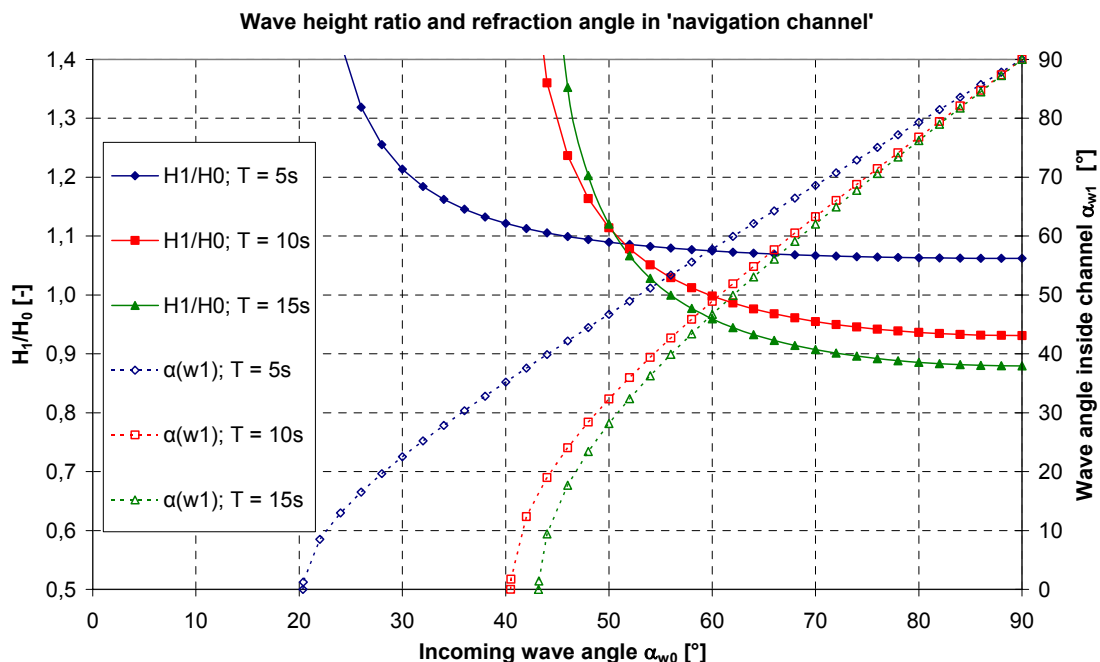


Figure 6-14: 'Solid' wave height ratio and 'dashed' refracted wave angle for three different wave periods in 'navigation channel'

In case of the 'pipeline trench' similar behaviour is observed. Critical wave angles are larger, although this critical angle of long period waves is less affected by the depth reduction than the critical angle of short period waves. Wave height reductions are up to 15% for perpendicular waves as could also be concluded from Figure 6-12.

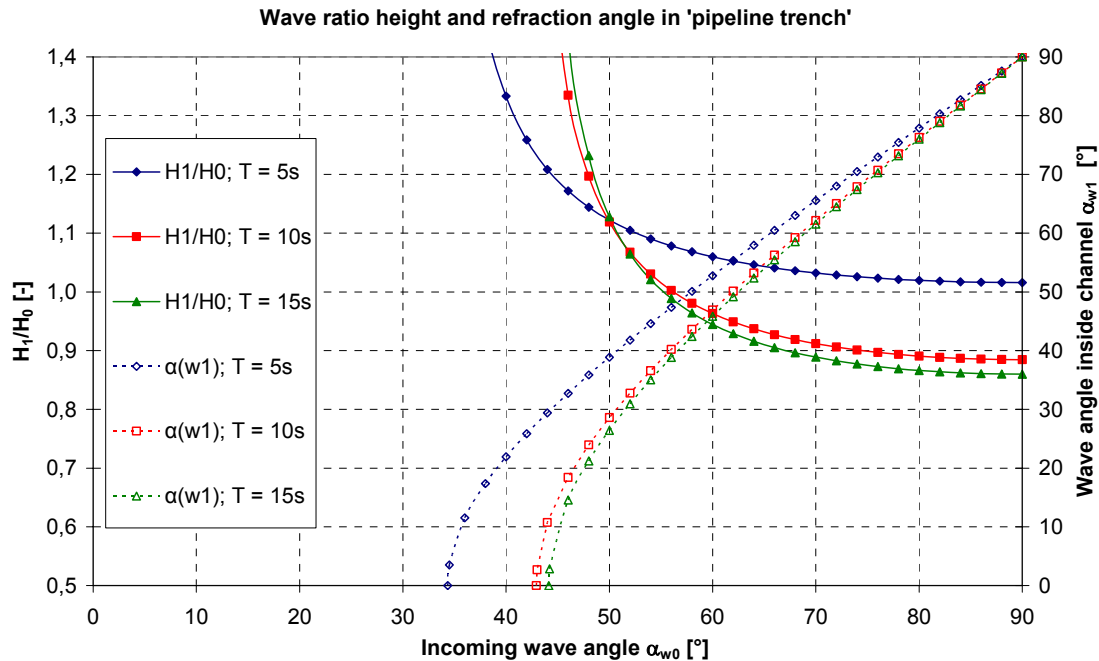


Figure 6-15: 'Solid' wave height ratio and 'dashed' refracted wave angle for three different wave periods in 'pipeline trench'

6.3 Combined action of currents and waves

If a current has a flow component in the wave propagation direction, i.e. if the waves and current are not directed perpendicular to each other, the wave characteristics will change. If the current is directed in the same direction of the waves, the wave height will decrease and the wavelength increase. If the current has the opposite direction, the wave height will increase and the wavelength decrease. For every current, this adapted wave characteristics can be calculated with help of a fixed and moving coordinate system. It is however rather cumbersome to calculate the adapted wave characteristics for each discrete step of the tidal cycle and for every location inside the channel. Besides, this effect will be significant for small angles between waves and current, but almost negligible for situations with a longshore current and a cross-shore wave field, which is often the case for channels and trenches. It was also assumed that the current motion will be dominant over the wave motion. Therefore and for numerical-model-considerations, this effect will be neglected from now on.

7 Threshold of motion on a slope

Opposite to sand borrowing pits or trenches in inland seas, offshore channels will almost always be subjected to significant bottom shear stresses induced by currents and waves. In case of seepage by waves (see Paragraph 5.1) the stability of the outer grains could be determined; in case of currents and waves it is impossible to speak of a fixed limit state; it is more useful to define a threshold of motion under various conditions.

In this way a fast rough estimate can be made whether continuous movement of the outer grains (sediment transport) will take place or not. If so, the situation would not have to be catastrophic, it is just an indication that morphology has to be considered in more detail. Instead of stability formulas of Shields and Sleath, transport formulas like the formulae of Bijker or Van Rijn have to be considered, see Chapter 8.

7.1 Currents

7.1.1 Threshold of motion on a plane bed

In 1936 Shields developed a stability formula for particles in uniform flow (logarithmic velocity profile) on a plane bed. With this formula the threshold of motion can be determined. This 'threshold' can best be defined as 'continuous movement of all grains' or as the 'start of sediment transport'. Van Rijn turned the well-known graph of Shields into a more usable form. A mathematical approximation of this graph is presented in Figure 7-1, in which the stability parameter Ψ_c is put along the vertical axis and the dimensionless grain diameter d_* along the horizontal axis:

$$\Psi_c = \frac{\tau_c}{(\rho_s - \rho_w)gd} = \frac{u_{*c}^2}{\Delta g d} \quad \text{Eq. 7-1}$$

$$\text{and} \quad d_* = d(\Delta g / \nu^2)^{1/3} \quad \text{Eq. 7-2}$$

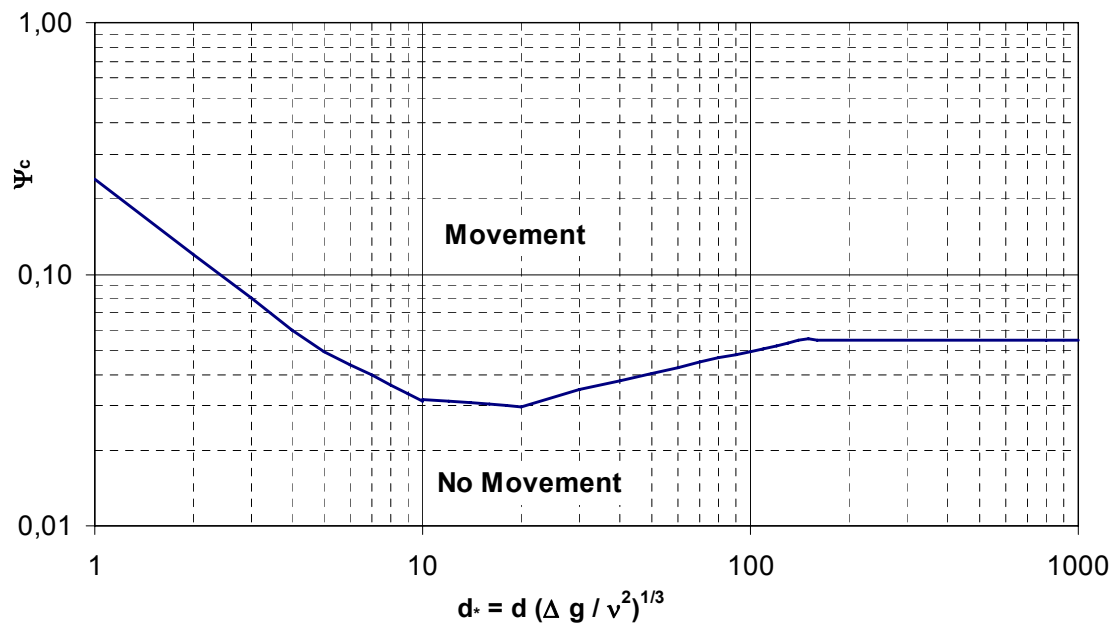


Figure 7-1: Van Rijn's mathematical approximation of the Shields' curve

For different dimensionless parameters d_* this graph can be described by the following mathematical expressions:

$$\begin{aligned}
 \Psi_C &= 0,24d_*^{-1} & \text{for } 1 \leq d_* \leq 4 \\
 \Psi_C &= 0,14d_*^{-0,65} & \text{for } 4 < d_* \leq 10 \\
 \Psi_C &= 0,04d_*^{-0,1} & \text{for } 10 < d_* \leq 20 \\
 \Psi_C &= 0,013d_*^{0,29} & \text{for } 20 < d_* \leq 150 \\
 \Psi_C &= 0,055 & \text{for } 150 < d_* \leq 1000
 \end{aligned}
 \tag{Eq. 7-3}$$

This Shields curve is normally used to find a diameter that is stable under the given conditions. In this thesis it is the other way around. Sediment properties of the reference soils are known, but we are interested under what flow conditions, still no transport takes place. Therefore the hydrodynamics of Chapter 6 are used, i.e. the formula for uniform wall flow, the formula of Chézy and the 'smoothness' coefficient of Nikuradse-Colebrook for a hydraulic rough situation and with a Prandtl-Von Karman logarithmic velocity profile:

$$u_* = \frac{\bar{u}\sqrt{g}}{C} \tag{Eq. 7-4}$$

$$C = \frac{\sqrt{g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) \tag{Eq. 7-5}$$

Combining equations 7-1, 7-2, 7-4 and 7-5 yields an expression for the critical flow velocity:

$$\bar{u}_{crit} = \sqrt{\Psi_c d_*} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) \tag{Eq. 7-6}$$

in which

$$\begin{aligned}
 \kappa &= \text{constant of Von Karman} = 0,4 & [-] \\
 \nu &= \text{kinematic viscosity} & [\text{m}^2/\text{s}]
 \end{aligned}$$

With this formula one can easily determine for which depth-averaged current velocities transport will take place, if the sediment properties, the water depth and the bed roughness are known. The stability parameter Ψ_C can be determined graphically from Figure 7-1 or analytically from equation 7-3.

The bottom roughness is a very influential parameter. A lot of research has been done. For a flat bed Engelund and Hansen (1973) found $k_r = 2 d_{65}$; Van Rijn proposed (1984, 1986) $k_r = 3 d_{90}$, which is about 4 to 5 d_{50} . Lammers (1997) and Boutovski (1998) found $k_r = 6 d_{50}$. Here a grain-related roughness of 5 d_{50} will be used.

If larger bed forms occur, this assumption is too low. A flat bed in offshore conditions isn't likely. In literature, values varying from 0,01 to 0,06 m often are used. Klein [2003] got good results applying 0,03 m in a number of tests within the framework of the Sandpit project for grain sizes varying from 100 to 300 μm in laboratory and field experiments. As a first approximation therefore a roughness of 0,03 m is assumed (comparable to ripples with a height of about 1 cm).

The above mentioned critical depth-averaged velocities are calculated for different grain diameters on a plane bed, see Table 7-1. In the left column, the bed roughness is assumed to be $k_r = 5 d_{50}$ (grain-dependent) and in the right column $k_r = 0,03 \text{ m}$ (constant for all grain sizes). Surprisingly, if the bed roughness isn't dependent on the grain diameter, small particles ($d_* < 4$; $d_{50} < 190 \mu\text{m}$) are equally stable. The apparent minimum at a particle diameter of 200 μm is only due to the mathematical approximation (equation 7-3).

Table 7-1: Critical depth-averaged current velocities

d_{50} [μm]	u_{crit} [m/s]					
	$h = 5 \text{ m}$		$h = 10 \text{ m}$		$h = 20 \text{ m}$	
	$k_r = 5 d_{50}$ [m]	$k_r = 0,03$ [m]	$k_r = 5 d_{50}$ [m]	$k_r = 0,03$ [m]	$k_r = 5 d_{50}$ [m]	$k_r = 0,03$ [m]
50	0,423	0,259	0,447	0,283	0,470	0,307
100	0,399	0,259	0,423	0,283	0,447	0,307
200	0,372	0,257	0,396	0,281	0,419	0,304
350	0,391	0,284	0,417	0,310	0,442	0,336
500	0,407	0,307	0,435	0,335	0,463	0,363
1000	0,536	0,433	0,575	0,473	0,615	0,513

This approximation can be combined with equation 7-6 to obtain formulations for the critical depth-averaged flow velocity. Assuming a constant bed roughness ($k_r = 0,03 \text{ m}$) a further simplification can be obtained:

$$\begin{aligned} \bar{u}_{crit} &= \sqrt{0,24} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) \approx 0,283 & \text{for } 1 \leq d_* \leq 4 \\ \bar{u}_{crit} &= \sqrt{0,14} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) d_*^{0,175} \approx 0,216 d_*^{0,175} & \text{for } 4 < d_* \leq 10 \\ \bar{u}_{crit} &= \sqrt{0,04} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) d_*^{0,45} \approx 0,116 d_*^{0,45} & \text{for } 10 < d_* \leq 20 \quad \text{Eq. 7-7} \\ \bar{u}_{crit} &= \sqrt{0,013} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) d_*^{0,645} \approx 0,066 d_*^{0,645} & \text{for } 20 < d_* \leq 150 \\ \bar{u}_{crit} &= \sqrt{0,055} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right) d_*^{0,5} \approx 0,136 d_*^{0,5} & \text{for } 150 < d_* \leq 1000 \end{aligned}$$

These critical depth-averaged flow velocities have been plotted both for a constant bed roughness ($k_r = 0,03 \text{ m}$) and for a grain-related bed roughness ($k_r = 5d_{50}$). In Figure 7-2 can clearly be observed that the critical flow velocity is constant to about $d_{50} = 190 \mu\text{m}$.

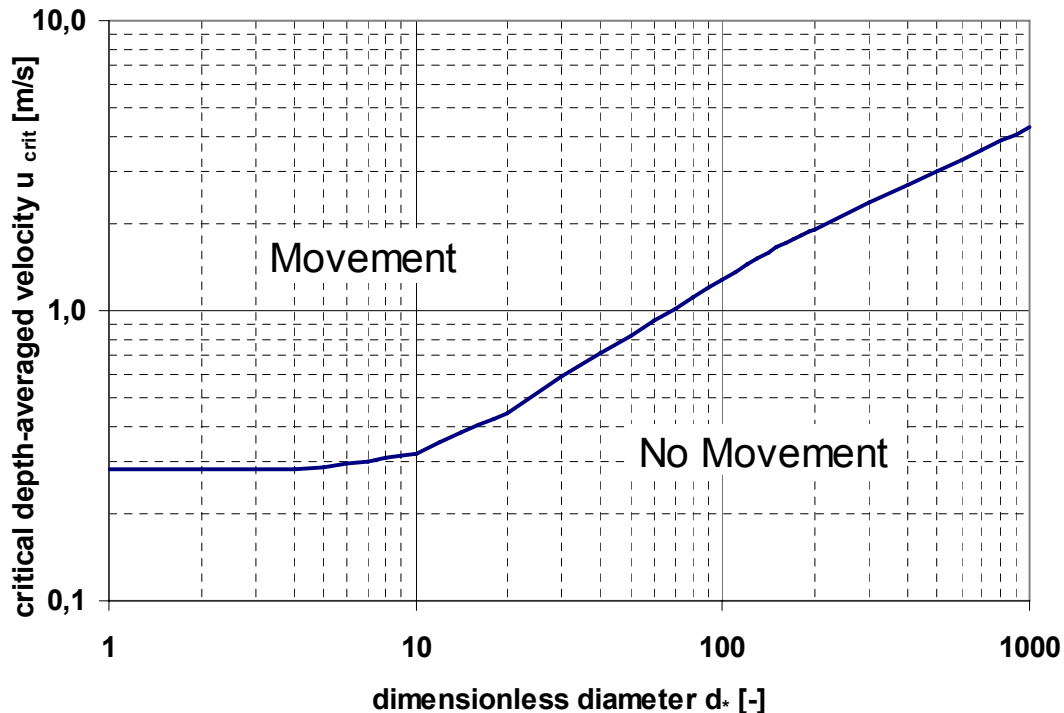


Figure 7-2: Critical depth-averaged velocities for a constant bed roughness $k_r = 0,03 \text{ m}$; water depth $h = 10 \text{ m}$

If the bed roughness is dependent on the particle diameter, the graph changes somewhat and shows a minimum at $d_* \approx 5$, ($d_{50} \approx 235 \mu\text{m}$). This is sometimes explained by the fact that very small particles are completely situated in the viscous sub-layer, where the bottom shear stress no longer is related to the square of the velocity.

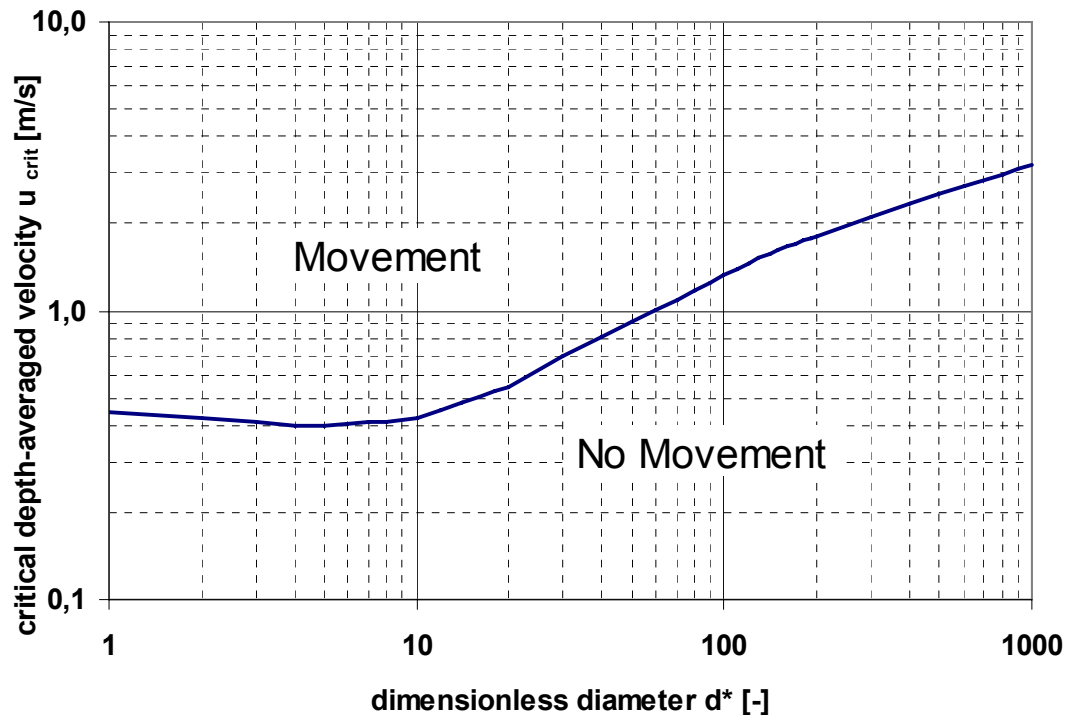


Figure 7-3: Critical depth-averaged velocities for grain-dependent bed roughness $k_r = 5d_{50}$; water depth $h = 10 \text{ m}$

7.1.2 Threshold of motion on a sloping bed: reduction coefficient

When currents pass over a channel, the depth-averaged current velocity drops, but on the other hand the grains on a slope are less stable, because gravity becomes an active force, see Figure 7-4.

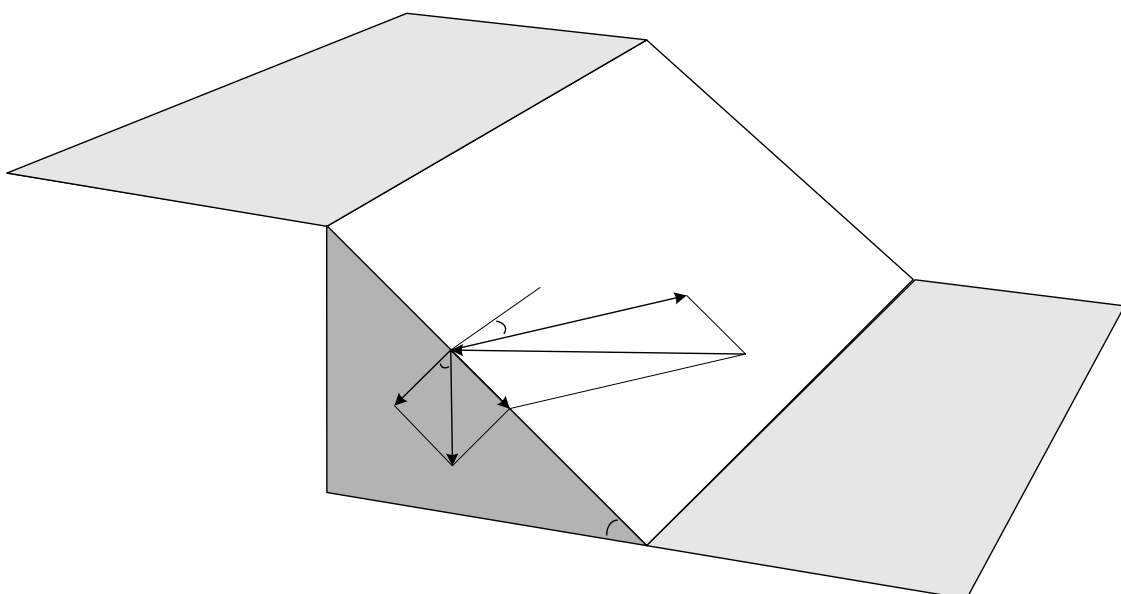


Figure 7-4: Forces on a slope: gravity force, flow force and counteracting friction force

If the slope angle β becomes larger than the angle of repose ϕ any load will induce movement. Of course we are interested in slope angles smaller than the angle of repose: $\beta < \phi$. To take this reduction of strength into account while applying the Shields formula, a reduction factor $K_{\alpha,\beta}$ will be deduced. This factor is dependent on the angle of the flow force with the channel axis (α) and the slope angle (β).

The friction force ($F_{\text{friction}} = W \cos \beta \tan \phi$) must balance the gravity force ($F_g = W \sin \beta$) and the flow force ($F_{\text{flow},\alpha,\beta}$ with a component along the channel axis ($F_{\text{flow},\alpha,\beta} \cos \alpha$) and a component along the slope ($F_{\text{flow},\alpha,\beta} \sin \alpha$)). These vectors have to be added vectorially to calculate the flow force.

$$(F_{\text{flow},\alpha,\beta} \cos \alpha)^2 + (F_{\text{flow},\alpha,\beta} \sin \alpha + W \sin \beta)^2 = (W \cos \beta \tan \phi)^2 \quad \text{Eq. 7-8}$$

The flow force then becomes:

$$F_{\text{flow},\alpha,\beta} = -W \sin \alpha \sin \beta + W \cos \beta \sqrt{\tan^2 \phi - \cos^2 \alpha \tan^2 \beta} \quad \text{Eq. 7-9}$$

When dividing $F_{\text{flow},\alpha,\beta}$ by the flow force on a normal plane bed ($F_{\text{flow}} = F_{\text{friction}} = W \tan \phi$), the reduction factor $K_{\alpha,\beta}$ then becomes:

$$K_{\alpha,\beta} = \frac{-\sin \alpha \sin \beta + \cos \beta \sqrt{\tan^2 \phi - \cos^2 \alpha \tan^2 \beta}}{\tan \phi} \quad \text{Eq. 7-10}$$

for $\beta < \phi$ and α is positive for flow on a downward slope. This rather complex formula reduces for a plane bed ($\beta = 0$) to $K = 1$. For a sloping bed and a current perpendicular to the channel axis ($\alpha = 90^\circ$) it becomes:

$$K_{\alpha=90^\circ,\beta} = \frac{-\sin \beta + \cos \beta \tan \phi}{\tan \phi} = \frac{-\sin \beta \cos \phi + \cos \beta \sin \phi}{\sin \phi} = \frac{\sin(\phi - \beta)}{\sin \phi} \quad \text{Eq. 7-11}$$

This factor is also known as the Schoklitsch-factor. For a current parallel to the channel axis ($\alpha = 0^\circ$), the reduction factor becomes:

$$K_{\alpha=0^\circ,\beta} = \frac{\cos \beta \sqrt{\tan^2 \phi - \tan^2 \beta}}{\tan \phi} = \cos \beta \sqrt{1 - \frac{\tan^2 \beta}{\tan^2 \phi}} = \sqrt{1 - \frac{\sin^2 \beta}{\sin^2 \phi}} \quad \text{Eq. 7-12}$$

The reduction factor $K_{\alpha,\beta}$ (Eq. 7-10, 7-11, 7-12) has to be substituted into equation 7-6 under the first square root, as it is a reduction of the stability.

In Figure 7-5 the reduction factor is plotted against slope angle β for different current angles α . As expected, the most unfavourable situation occurs for a downward slope in the flow direction. It can also be seen that according to this theory upward slopes can increase the stability of a particle on a slope. In practice however one should be very careful, because in most of the situations currents do not always approach the slope from this favourable angle or at the same magnitude. In case of tidal flow a turn of the current will occur. Reduction factors larger than 1 are therefore considered to be only theoretical, but not useful in engineering practice.

To simplify calculations in offshore conditions most of the times flow directed perpendicular to the channel will be considered. In reality, tidal currents will be more or less perpendicular to the channel axis. This means that the lowest reduction factors ($\alpha = 90^\circ$) are taken as a safe upper boundary. In inland navigation channels or tidal gullies parallel flow is more likely ($\alpha = 0^\circ$) and slightly higher reduction factors can be applied.

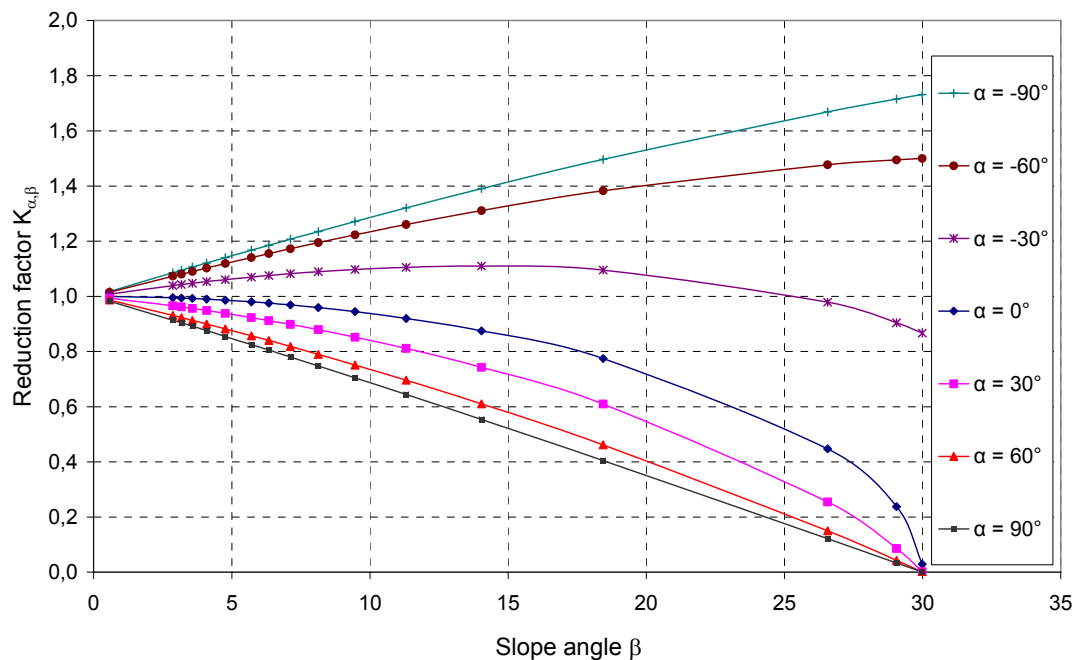


Figure 7-5: Reduction factors for different slope angles ' β ' and flow angles ' α '

Of course, these reduction factors have to be combined with local flow velocities. When looking at a gentle channel profile with side slopes of 1:5 (no flow separation), one may assume that the velocity profiles stays logarithmic and because of continuity, the depth-averaged velocity is inversely proportional to the depth. The situation at the top of the slope will be normative in case of a uniform sloping bottom; the depth-averaged velocity is largest here. When descending along the slope, the soil particles become more stable.

In Figure 7-6 (perpendicular current) and Figure 7-7 (parallel current) one can easily determine for a given flow condition the steepest possible slope, if one does not allow transport. In this way, it is a quick aid to gain insight, when dredging a side slope, in which situations (I) transport does not occur, (II) already occurs at the surrounding seabed or (III) will occur when dredging a steep slope. In most offshore conditions transport already occurs at the initial situation and dredging of a slope only stimulates more transport. Most transport equations contain formulations for the threshold of motion. The difference between the actual velocity and the critical velocity appears in transport equations to the power of 3 to 5, so the determination of this critical velocity is rather important. The morphology of trenches and channels will be discussed from Chapter 8.

The two graphs below show some differences. With flow perpendicular to the channel axis, the negative influence of the slope is already noticeable for relative gentle slopes, while this influence with flow parallel to the channel axis is negligible for slopes gentler than about 1:4. As explained above, there are no differences between small particles ($d_{50} < 200 \mu\text{m}$) for a constant bed roughness. The steepest possible slope occurs of course for the angle of internal friction and no current: $\tan \phi = 1:1,73$. Please note the two scales on the y-axis, because of the relation $u_* = u\sqrt{g/C}$.

Example: Particles with $d_{50} = 500 \mu\text{m}$ are stable in a depth-averaged uniform flow of 0,30 m/s at a water depth of 10 m. If this flow is parallel to the slope, these particles remain stable until the slope gets steeper than 1:3. If this flow is perpendicular to the slope, the particles already become unstable at a slope of 1:9.

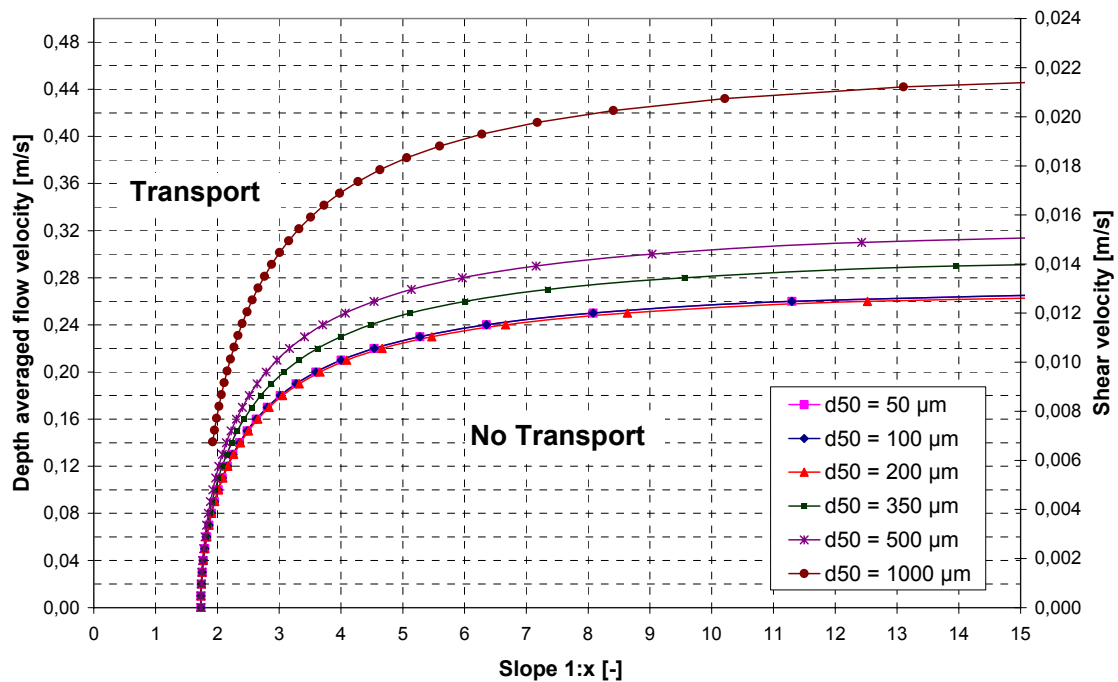


Figure 7-6: Critical slopes for different current velocities perpendicular to the channel axis; water depth $h = 10$ m

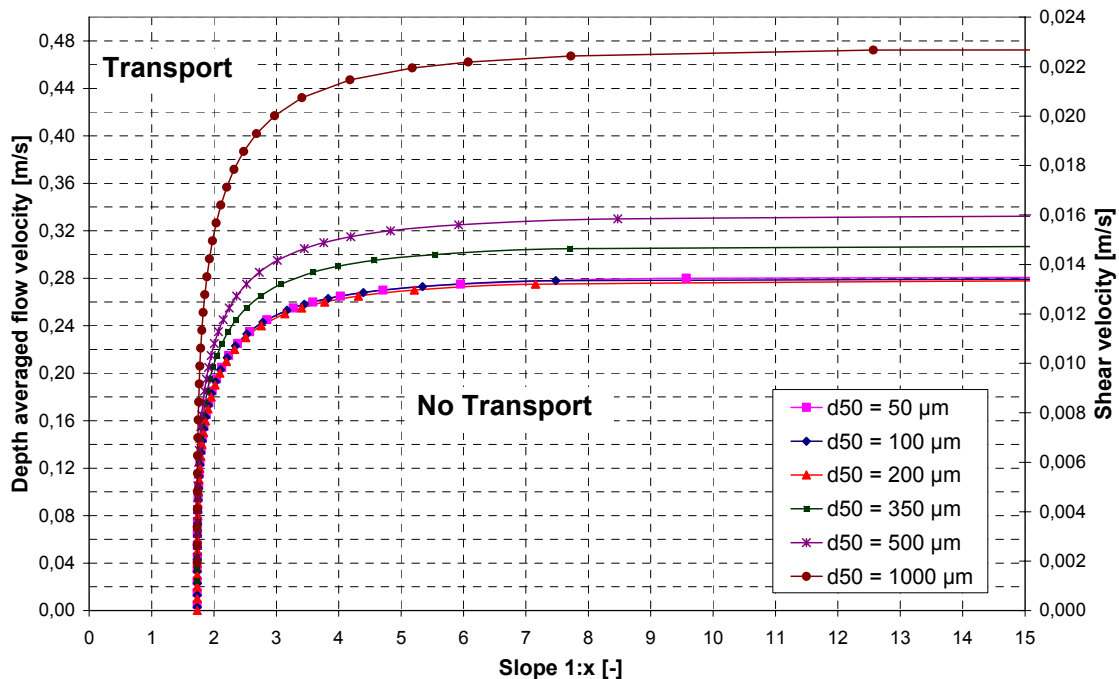


Figure 7-7: Critical slopes for different current velocities parallel to the channel axis; water depth $h = 10$ m

The above figures are only applicable for a water depth of 10m. To increase the applicability of both graphs, another auxiliary graph is presented, which is based on the relation between the bed shear velocity and the depth-averaged velocity according to a logarithmic velocity profile (Eq. 7-4). The left y-axis of the above figures, in fact, has to be multiplied by the correction factor obtained from Figure 7-8 for a certain water depth.

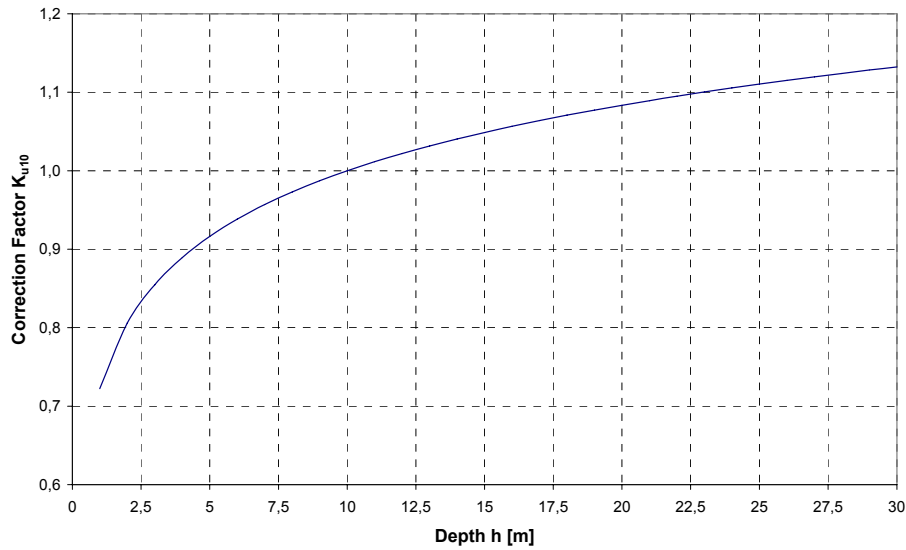


Figure 7-8: Correction Factor for other water depths than 10m

7.1.3 Threshold profile

If currents enter a navigation channel perpendicularly, the depth-averaged velocity will decrease because of continuity. When the side slopes are straight, this means that grains at the toe of the slope will be more stable than grains at the top of the slope. For grains to be *equally stable at the top and at the toe* of a side slope, this slope can become steeper towards the channel bottom. First the critical slope angle β_{crit} of the side slope has to be defined. With help of equations 7-1, 7-2 and 7-6 first an expression for the reduction factor $K_{\alpha,\beta}$ will be deduced:

$$K_{\alpha,\beta} = \frac{q_{crit}^2 \kappa^2}{\Psi_C d_{50} \Delta g h^2 \ln^2 \left(\frac{12h}{k_r} \right)} \quad \text{Eq. 7-13}$$

in which

- q_{crit} = discharge at which the grains at the surrounding (plain) seabed are at the threshold of motion, $q_{crit} = u_{crit}/h_0$;
- u_{crit} = critical velocity calculated from equation 7-7;
- Ψ_C = stability parameter calculated from equation 7-3;
- h = water depth somewhere along the side slope; $h > h_0$.

Assuming a current directed perpendicular to the channel axis (Eq. 7-11) the following formula for the critical slope angle is obtained:

$$\beta_{crit} = \phi - \arcsin \left(\frac{\sin \phi q_{crit}^2 \kappa^2}{\Psi_C d_{50} \Delta g h^2 \ln^2 \left(\frac{12h}{k_r} \right)} \right) \quad \text{Eq. 7-14}$$

This expression has been plotted in Figure 7-9. This graph clearly shows that a depth expansion from 10 to 20 m allows for much steeper slopes. A further increase in channel depth results in a slower increase of the slope steepness, eventually resulting in the natural slope ($1/\tan\beta = 1/\tan\phi$) as an upper boundary.

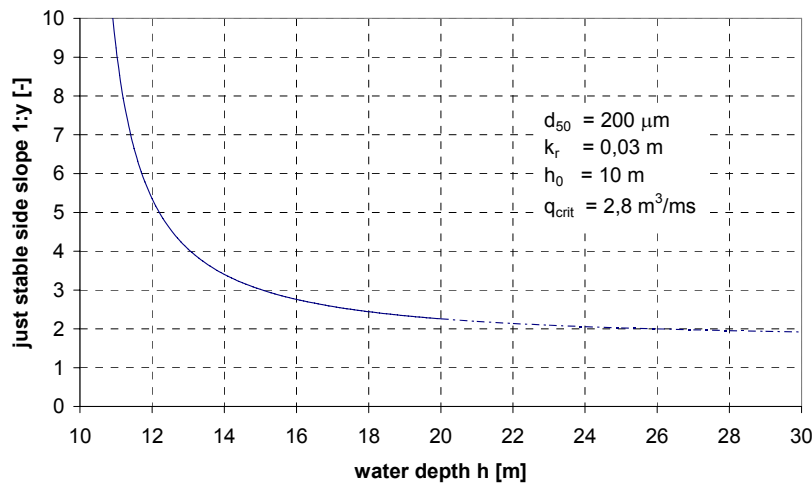


Figure 7-9: Just stable side slopes for a channel in a surrounding seabed of MSL-10m

Because of this significant increase of just stable side slopes within the limits of a navigation channel, it seems relevant to define a channel geometry with side slopes that are just stable (or at the threshold of motion). This particular channel geometry will be named the 'threshold profile'. It is assumed that the side slopes first becomes steeper until half of the slope height and then flatter again. In this way, when straight side slopes are dredged, the eroded volume on top of the slope is equal to the accreted volume at the toe of the slope. The upper half of the side slope is described by equation 7-14; the lower half of the slope is point symmetric to the upper half, see Figure 7-10.

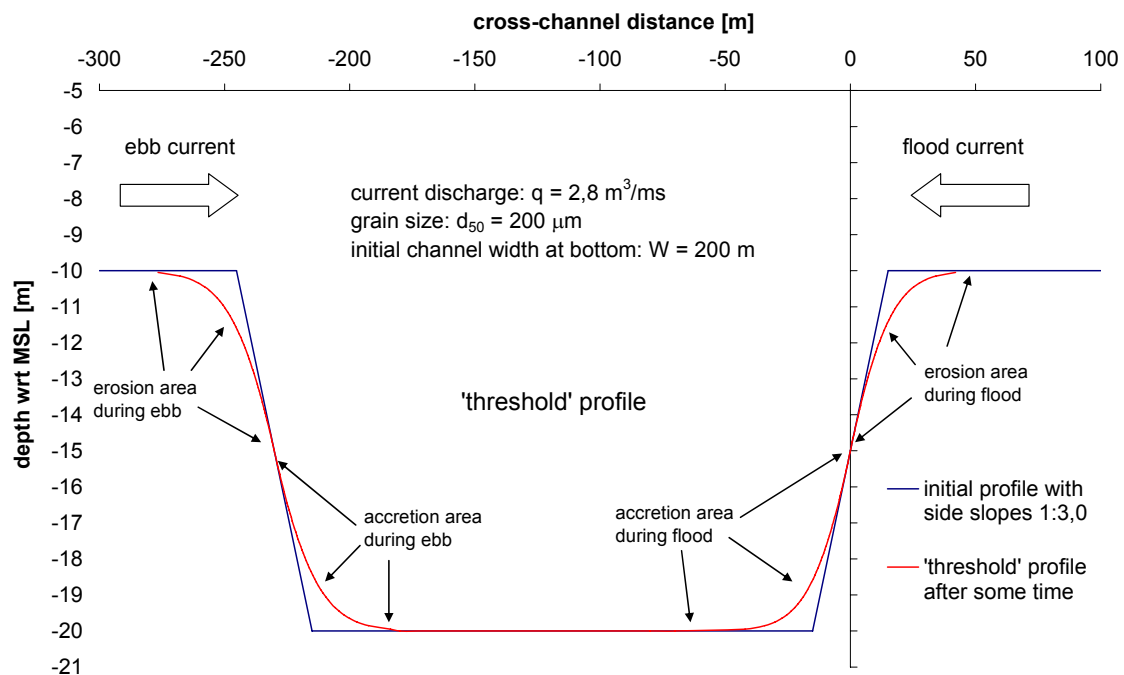


Figure 7-10: 'Threshold' profile that is just completely stable during ebb and flood current

The 'blue' geometry is the initial channel geometry with side slopes of 1:3,0; the 'red' geometry will develop under very mild hydrodynamic conditions, not exceeding the critical discharge: the top of the side slope will erode and roll down, until the threshold profile is reached. In case of a flood current only the upper half of the upstream slope will erode, because all other grains are more stable (milder slope or favourable influence of gravity).

This 'threshold profile' is of course dependent on water depth and slope height, but not on grain size, at least when the bed-form related bed roughness is used. After all, this profile is defined at the critical discharge for a certain grain size.

For grain sizes smaller than 200 μm , this critical discharge is equal, see Figure 7-2 or Figure 7-6. For larger grain sizes, the critical discharge increases to 3,07, 3,35 and 4,73 m^3/ms for grain diameters of respectively 350, 500 and 1000 μm for $h_0 = 10 \text{ m}$.

Because sediment transport is related to the 3rd to 5th power of the difference of the actual flow velocity and the critical flow velocity, the difference between the actual geometry and the 'threshold geometry' could also be a good measure to estimate the development of the channel geometry. Because of the larger critical velocities of larger grains, the morphological development will be much slower for larger grains.

As a final remark it is emphasized that, although critical flow velocities are rather small inside the channel, there is some uncertainty of the resemblance of the velocity profile to the logarithmic profile, especially at the steeper parts of the side slopes. Nevertheless, the 'threshold profile' has a very natural appearance.

7.2 Waves

Not only currents induce bed shear stresses, but also waves appear to be very effective stirring up sediment particles. Considering the water depths in this thesis varying from 5 to 20m, this subject is positioned in the 'transitional water depth'-range judging by Table 7-2. The smallest waves that satisfy the shallow water assumption and the largest waves that satisfy the deep water assumption are calculated for three water depths. Common waves will have wave periods between 5 and 14 s and wavelengths between 40 and 100 m, so are situated in the 'transitional water depth' range.

Table 7-2: Smallest waves satisfying 'shallow water assumption' and biggest waves satisfying 'deep water assumption'

	shallow water			deep water			
h	5	10	20	5	10	20	m
L	>100	>200	>400	<10	<20	<40	m
L_{limit}	100	200	400	10	20	40	m
c	7,1	10,0	14,1	4,0	5,6	8,0	m/s
T	14,1	20,0	28,3	2,5	3,5	5,0	s

Although waves that are entering shallow water become more and more non-sinusoidal, they still can quite reasonably be described by linear wave theory (see Paragraph 5.1). The horizontal orbital velocity 'u' and excursion 'a' are given by:

$$u = \frac{\omega H}{2} \frac{\cosh k(h+z)}{\sinh kh} \sin(\omega t - kx) \quad \text{Eq. 7-15}$$

$$a = -\frac{H}{2} \frac{\cosh k(h+z)}{\sinh kh} \cos(\omega t - kx) \quad \text{Eq. 7-16}$$

On top of the wave boundary layer at a small distance δ from the bed the amplitude of the horizontal orbital velocity $\hat{u}_0$ and the excursion $\hat{a}_0$ can be defined as:

$$\hat{u}_0 = \frac{\omega H}{2} \frac{1}{\sinh kh} \quad \text{Eq. 7-17}$$

$$\hat{a}_0 = \frac{H}{2} \frac{1}{\sinh kh} \quad \text{Eq. 7-18}$$

The thickness of this wave boundary layer ' δ ' is defined for turbulent flow by Fredsøe:

$$\delta = 0,15 \left(\frac{\hat{a}_0}{k_r} \right)^{-0,25} \quad \hat{a}_0 = 0,15 k_r^{0,25} \left(\frac{H}{2 \sinh kh} \right)^{0,75} \approx 0,037 \left(\frac{H}{\sinh kh} \right)^{0,75} \quad \text{Eq. 7-19}$$

for a wave-related bottom roughness k_r of 0,03 m. As can be expected, the boundary layer thickness increases with decreasing water depth, increasing wave height and increasing wave period. A longer period means more time for the boundary layer to grow. Jonsson found in 1966 the following relation for the amplitude of the wave-related bed-shear stress:

$$\hat{\tau}_w = 0,5 \rho f_w \hat{u}_0^2 \quad \text{Eq. 7-20}$$

in which the friction factor f_w can be described as:

$$f_w = e^{-5,977 + 5,213 \left[\frac{\hat{a}_0}{k_r} \right]^{-0,194}} \quad \text{if} \quad \left[\frac{\hat{a}_0}{k_r} \right] \geq 1,59$$

$$f_w = 0,30 \quad \text{if} \quad \left[\frac{\hat{a}_0}{k_r} \right] < 1,59 \quad \text{Eq. 7-21}$$

Now the situation becomes a bit more complicated. An increase of the wave period means a larger wavelength and therefore a larger horizontal excursion $\hat{a}_0$ and boundary layer thickness ' δ ', which results in a smaller friction factor ' f_w '. On the other hand, the horizontal velocity will increase, although slower and slower. As a result, if the wave period increases, the shear stress will 'decreasingly' increase for larger water depths ($h = 15; 20\text{m}$) and will decrease for small water depths ($h = 5\text{m}$), see Figure 7-11. The apparently inconsistent phenomenon of larger bed-shear stresses in deeper water is caused by the assumption of depth-limited waves ($H = 0,5h$): in deeper water the presented wave is higher in amplitude, larger in length and longer in period.

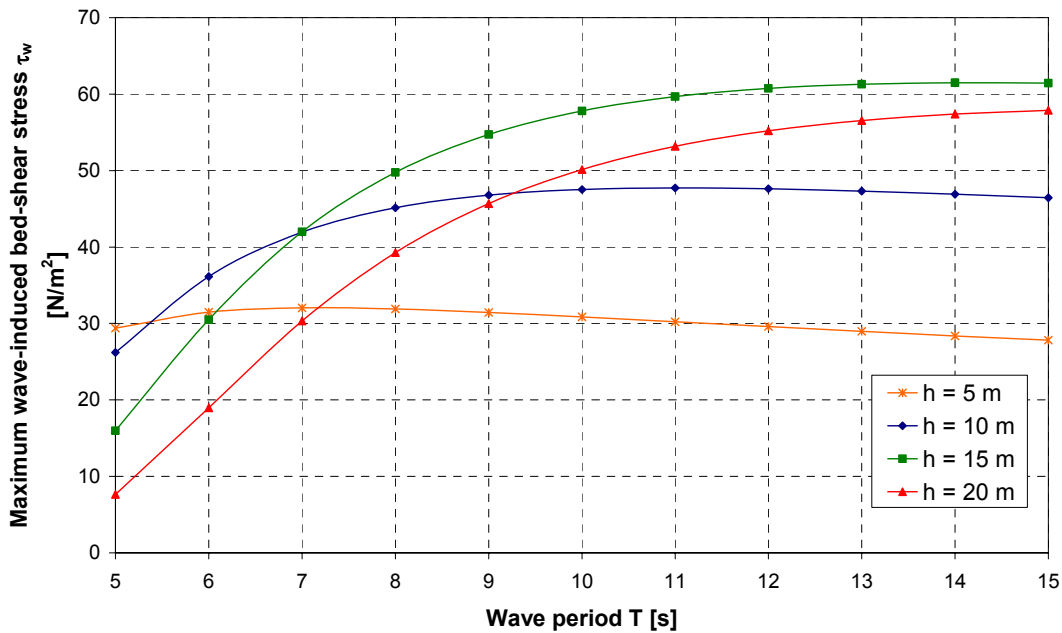


Figure 7-11: Wave-induced bed-shear stresses for depth-limited waves against wave period

Now the bed shear stress is known, the threshold of motion for a certain grain diameter has to be defined. Because of the wave boundary layer development that is completely different from the situation with currents, the Shields' curve was modified by Sleath to represent stability of loose grains in non-breaking waves.

In the same way Van Rijn expressed the Shields' Curve into mathematical formulations, Sleath's Curve can be formulated:

$$\begin{aligned} \Psi_{C;w} &= 0,1231d_*^{-0,6405} & \text{for } 1 \leq d_* \leq 20 \\ \Psi_{C;w} &= 0,018 & \text{for } 20 \leq d_* \leq 30 \\ \Psi_{C;w} &= 0,0011d_*^{0,8234} & \text{for } 30 \leq d_* \leq 120 \\ \Psi_{C;w} &= 0,055 & \text{for } 120 \leq d_* \leq 1000 \end{aligned} \quad \text{Eq. 7-22}$$

The above formulations are plotted in Figure 7-12. The Shields' Curve for uniform flow (dashed line) is shown as a reference.

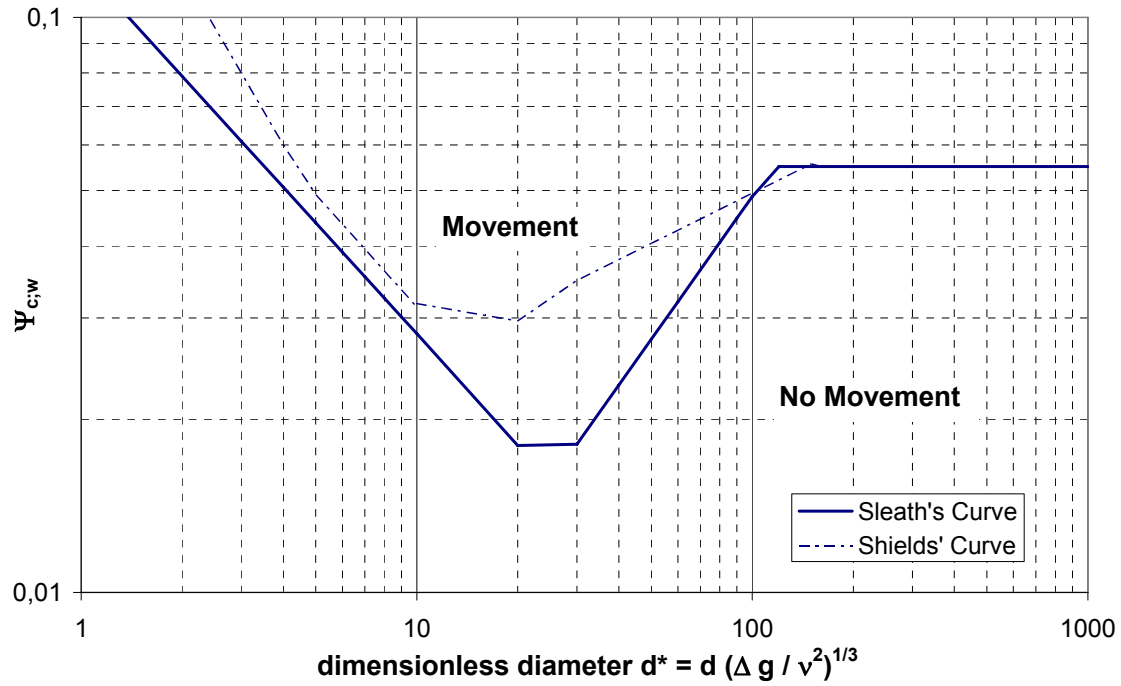


Figure 7-12: Mathematical approximation of Sleath's Curve (solid line) and Shields' Curve (dashed line);

Sleath used the same stability criterion as Shields:

$$\Psi_{C;w} = \frac{\tau_{C;w}}{(\rho_s - \rho_w)gd} \quad \text{Eq. 7-23}$$

So for the critical wave-induced bed shear stress is obtained:

$$\tau_{C;w} = (\rho_s - \rho_w)gd\Psi_{C;w} \quad \text{Eq. 7-24}$$

If this expression is equated with Jonsson's expression 7-20, the following relation for the maximum amplitude of the orbital velocity at the bed is found:

$$\hat{u}_0 = \sqrt{\frac{2\Delta g d_{50} \Psi_{C;w}}{f_w}} \quad \text{Eq. 7-25}$$

Combining this expression with equation 7-17 yields the formula for the critical wave height:

$$H_{crit} = \frac{2\sqrt{2} \sinh kh}{\omega} \sqrt{\frac{\Delta g d_{50} \Psi_{C;w}}{f_w}} \quad \text{Eq. 7-26}$$

This equation can only be solved by iteration, because the wave friction factor f_w is dependent on the wave height. Because the grain sizes of the reference soils are maximum 1000 μm ($d_* = 21,24$), they are almost completely situated at the 'first' branch ($d \leq 940 \mu\text{m}$) of the mathematical approximation. This means that the part " $d_{50} \Psi_{C;w}$ " in equation 7-26 can be substituted:

$$d_{50} \Psi_{C;w} = 2,083 \cdot 10^{-4} d_{50}^{0,3595} \quad \text{for} \quad d_{50} \leq 940 \mu\text{m} \quad \text{Eq. 7-27}$$

If also the following values are used for $\Delta=1,65$, $g=10 \text{ m/s}^2$, $\nu=1,3 \cdot 10^{-6} \text{ m}^2/\text{s}$, equation 7-26 can be further simplified to:

$$H_{crit} = 2,639 \cdot 10^{-2} d_{50}^{0,18} \frac{T \sinh kh}{\sqrt{f_w}} \quad \text{for} \quad d_{50} \leq 940 \mu\text{m} \quad \text{Eq. 7-28}$$

which is only dependent on the wave characteristics (T and k are related to each other), the grain diameter, the bed roughness and the water depth.

Again the difference between bed-related or grain-related bed roughness can be made. The critical wave height appears to be very sensitive to the bed roughness. To illustrate this behaviour, in Figure 7-13 the critical wave heights are plotted for the reference case "navigation channel", based on bed-related roughness ($k_r = 0,03$). The solid lines represent the surrounding seabed and the dashed lines the channel bottom. Especially the short period waves are less felt on the channel bottom.

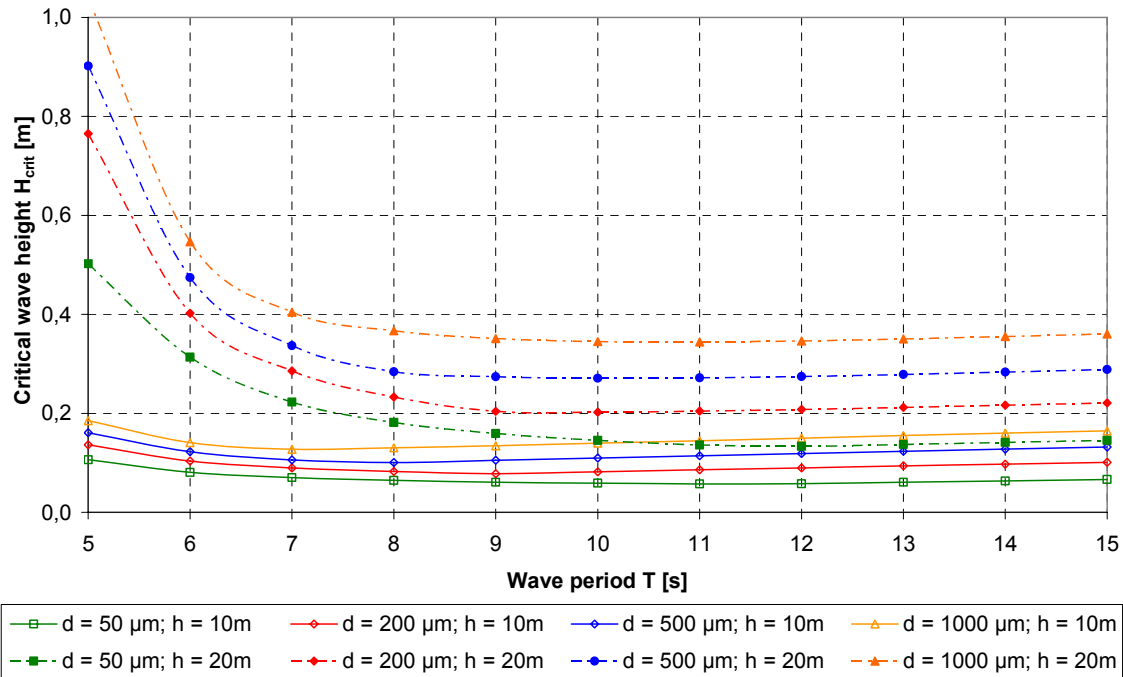


Figure 7-13: Critical wave heights for different wave periods at surrounding seabed and at the channel bottom, based on bed-related roughness ($k_r = 0,03\text{m}$)

When the seabed is rough, very small waves of only a few decimetres are already capable of stirring up some sediment. The same procedure for a grain-related bed roughness ($k_r = 5d_{50}$) yields larger waves, see Figure 7-14. In both graphs, the difference between the surrounding seabed and the channel bottom is clear.

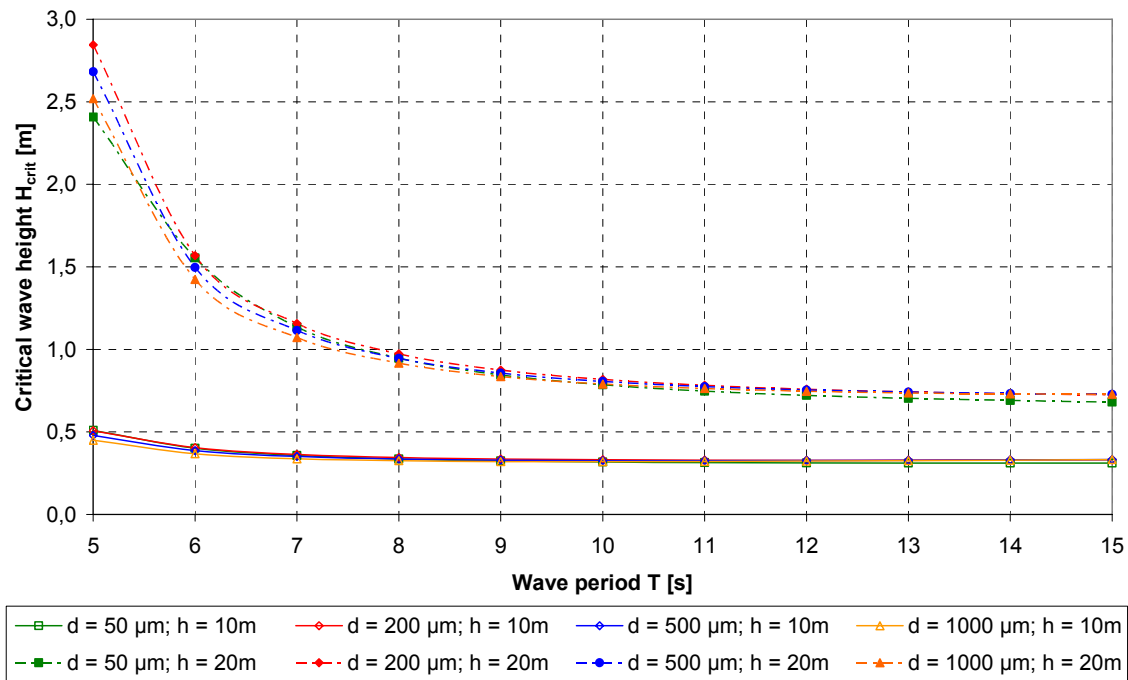


Figure 7-14: Critical wave heights for different wave periods at surrounding seabed and at the channel bottom, based on grain-related roughness ($k_r = 5d_{50}$)

Although there is no net transport over a wave cycle, it is very obvious that waves combined with a current are very effective in reducing the threshold of motion and therefore increasing sediment transport.

7.3 Combined action of currents and waves

Although the action of currents and waves has been treated separately in the previous paragraphs, in most cases both forces will be present. Bijker (1967) stated that, when the current is dominant over waves, the influence of these waves can be taken into account by adding the current and orbital velocity vectorially.

Two problems that occur are:

- at which depth should both forces be added;
- the current velocity is assumed stationary, whereas the wave motion is assumed to be sinusoidal.

Bijker added both current- and wave-induced shear stress at a height $z_t (=ez_0$ in which z_0 is the zero-velocity level) and integrated the resultant over a full wave cycle. Without giving the complete derivation, an expression is presented that is not longer dependent on the wave direction:

$$\bar{\tau}_{cw} = \tau_c + \frac{1}{2} \hat{\tau}_w = \tau_c \left[1 + \frac{1}{2} \left(\xi \frac{\hat{u}_0}{V} \right)^2 \right] \quad \text{Eq. 7-29}$$

in which:

$$\xi = C \sqrt{\frac{f_w}{2g}} \quad \text{Eq. 7-30}$$

One should realize that this bed shear stress is a time-averaged value; during a wave cycle larger stresses occur. However, when considering sediment transport (see Chapter 8), this time-averaged approach is legitimate.

Four different hydrodynamic situations have been calculated for the navigation channel:

$q = 5 \text{ m}^3/\text{ms}$; $H = 1\text{m}$; $T = 8\text{s}$;
 $q = 5 \text{ m}^3/\text{ms}$; $H = 2\text{m}$; $T = 8\text{s}$;
 $q = 10 \text{ m}^3/\text{ms}$; $H = 1\text{m}$; $T = 8\text{s}$;
 $q = 10 \text{ m}^3/\text{ms}$; $H = 2\text{m}$; $T = 8\text{s}$;

The resulting bed shear stresses have been plotted to illustrate the decaying behaviour with increasing depth, see Figure 7-15.

It appears that the wave-induced bed shear stress at the channel bottom is roughly 40% of the surrounding-bed-value. The larger water depth reduces the wave impact, but also results in a larger wavelength. The net effect, however, is still a decrease of the horizontal excursion a_0 and horizontal particle velocity u_0 . Because the excursion is reduced inside the channel, the wave friction factor f_w increases. The overall effect on the bed shear stress is a reduction inside the channel of 60%, whereas the water depth has doubled.

The current-induced bed shear stress at the channel bottom is about 20% of the surrounding-bed-value. This reduction is mainly caused by the halved current velocity, which would have resulted in a four times smaller bed shear stress. Because also the Chézy-coefficient increases (so friction decreases) a value of roughly 20% is obtained.

The combined response of current and waves to the expansion in depth is a reduction of the bed shear stress up to 70%, see Figure 7-15. This calculation example clearly shows that the stability of particles inside the trench can be much larger. Because the sediment transport capacity is related to this bed shear stress, it is therefore obvious that there will be a large gradient in sediment transport over the channel.

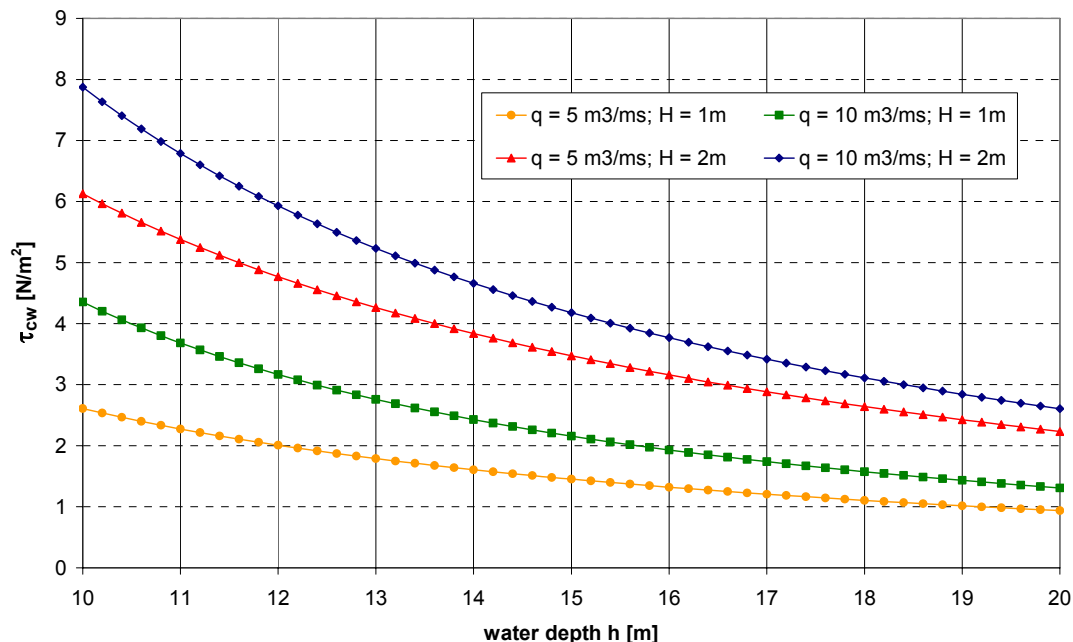


Figure 7-15: Bed shear stress due to the combined action of current and waves against water depth; reference case "navigation channel"

8 Sediment transport in trenches and channels

8.1 Choice of sediment transport formula

Before describing the characteristics of sediment transport in trenches and channels and on sloping bottoms in particular, the pros and cons of a number of sediment transport formulae will be balanced against each other, so the appropriate sediment transport theory will be used. An extensive comparison of five well-known sediment transport formulae by Camenen en Larroudé [2003] gave insight in the applicability of each formula. The formulae under investigation were:

- Bijker total load formula [1968] which was developed in a river environment and adapted to a coastal environment. It distinguishes between bed load and suspended transport; waves are taken into account via an increase of the bottom shear stress;
- Bailard formula [1981] which is based on a formulation of energy for sediment transport caused by waves;
- Van Rijn total load formula [1984, 1989] expresses total sediment also as bed load and suspended load transport;
- Dibajnia and Watanabe total load formula [1992], which accounts for the instantaneous velocity due to wave and current interaction and the resulting sediment movement;
- Ribberink bed load formula [1998], a quasi-steady model of sediment transport using instantaneous shear stresses.

The main conclusion of this research with regard to this thesis was that the Van Rijn total load formula yielded reasonable results (which is good compared to the others) in situations with prevailing currents, outside the breaker zone. However, Van Rijn is weak at predicting sediment transport with prevailing waves, because in his approach waves only increase the bottom shear stress and do not act as a transport mechanism. The direction of the sediment transport will therefore always be that of the current. As long as the current velocity will be greater than the (oscillating) wave velocity, this error remains limited.

Wave transport due to wave asymmetry isn't taken into account, although waves in coastal zones are substantially non-sinusoidal. To describe these waves well, 2nd order-Stokes theory or even better cnoidal theory (see Chapter 5) should be used.

The important phase-lag-effect (the suspended sediment in the vertical lags behind on the present hydrodynamic conditions), which becomes more significant with finer sediment, causes large differences between calculated and observed sediment transport, especially in tidal conditions.

The Van Rijn formula asks for long computation times compared to the other formulae because the integral of the suspended load over the depth needs to be computed.

Taking all of the above into account it may be concluded that the Van Rijn formula is preferable to the other sediment formulae for the conditions in this thesis (current-induced transport outside the breaker zone, both bed-load and suspended transport). Therefore Van Rijn's line of thought will be used throughout this thesis.

8.2 Currents only

Backfilling of trenches will occur due to sediment transport, when the sediment capacity is less within the channel than outside, and due to gravity effects, when a bed load particle starts to move. The backfilling of dredged channels and trenches is predicted successfully only if a detailed description of the flow *and* sediment transport process is applied.

In the previous chapters, the hydrodynamics in channels and threshold of motion on slopes were studied. Some basic assumptions had to be done regarding the current velocity profile (logarithmic profiles).

If a certain channel or trench is exposed to such 'serious hydrodynamic' conditions, it is not sufficient to consider the threshold of motion, but transport equations come into play.

The natural backfilling mechanisms are partly levelling of side slopes due to bed load movement and partly settling of suspended sediment carried across the channel by currents. It is important to distinguish between the combined wave-current motion offshore in the non-breaking area and the primarily wave-induced longshore sediment transport. In the latter case, the transport will be concentrated on the breaker bars, whereas in the first case the sediment transport will be more equally distributed and can be better represented by a 2-dimensional approach. As was already mentioned, the emphasis of this thesis will be on the non-breaking regime.

8.2.1 Types of transport

Transport of sediment can be split up into two different kinds of transport: bed load transport and suspended load transport. The bed load transport will immediately be affected by a change of the bed form, while the suspended load, which is completely surrounded by water, needs some time to adapt. To contrast bed load with suspended load one needs to define an artificial bed layer in which the bed load transport takes place, because in reality such a sharp boundary does not exist. One usually distinguishes three types of particle motion of which the first two types often are considered as bed load transport (Van Rijn, 1984a):

- I. rolling and sliding motion or combined
- II. saltation motion (i.e. jumping over a maximum distance of a few particle diameters)
- III. suspended particle motion

In this bed-load layer the particle motion is gravity dominated and turbulent forces are of little significance. According to Bagnold, particles come into suspension if the bed shear velocity u_* exceeds the particle fall velocity w_s . For the reference soils, this means that some sediment primarily is transported as bed load, like the coarse sands. Very fine sediments (MSI), on the other hand, will always come into suspension as soon as the critical bed shear velocity is exceeded. In between are the medium sands, which sometimes just are transported in the form of bed load and sometimes also in the form of suspended load, depending on the conditions. In Table 8-1 an example is presented for a uniform flow in absence of waves to get an idea. Please note the difference between the surrounding bed and the trench bottom. It is needless to say that in presence of waves transport capacities increase considerably.

Table 8-1: Types of transport according to Bagnold for reference case 'Navigation Channel'

Navigation Channel ($h_0=10\text{m}$; $h_1=20\text{m}$)				u (depth averaged) [m/s]		u (depth averaged) [m/s]	
				0,5		1	
				u^* surrounding bed	u^* channel bottom	u^* surrounding bed	u^* channel bottom
soil type	d_{50} [μm]	w_s [m/s]	u_{crit} [m/s]	0,0245	0,0109	0,0489	0,0218
MSI	50	0,0029	0,0137	<i>suspended</i>	<i>no transport</i>	<i>suspended</i>	<i>suspended</i>
MFS	200	0,0252	0,0135	<i>bed load</i>	<i>no transport</i>	<i>suspended</i>	<i>bed load</i>
MMS	500	0,0663	0,0161	<i>bed load</i>	<i>no transport</i>	<i>bed load</i>	<i>bed load</i>
MCS	1000	0,1084	0,0228	<i>bed load</i>	<i>no transport</i>	<i>bed load</i>	<i>no transport</i>

Van Rijn [1984b] considered the Bagnold-criterion "as an upper limit at which a concentration profile starts to develop" and suggested an intermediate stage "at which locally turbulent bursts of sediment particles are lifted from the bed into suspension". This intermediate stage can for the grain sizes under consideration be defined as:

$$u_{*transition} = \frac{4w_s}{d_*} \quad \text{for} \quad 1 \leq d_* \leq 10 \quad \text{Eq. 8-1}$$

Expressed as the depth-averaged current velocity, the above equation changes to:

$$\bar{u} = \frac{4w_s C}{d_* \sqrt{g}} \quad \text{Eq. 8-2}$$

For the reference soils MSI, MFS and MMS for which this expression is valid, the depth-averaged current velocities become: $d_{50} = 50, 200, 500 \mu\text{m}$, $\bar{u} = 0,22, 0,48$ and $0,51 \text{ m/s}$. For the same situations of Table 8-1, the type of sediment transport has been determined according to Van Rijn's criterion. It can be observed that at the surrounding seabed also some suspended transport can occur for MFS in a depth-averaged current velocity of $0,5 \text{ m/s}$ and for MMS in a current velocity of 1 m/s .

Table 8-2: Types of transport according to Van Rijn for reference case 'Navigation Channel'

Navigation Channel ($h_0=10\text{m}$; $h_1=20\text{m}$)					u (depth averaged) [m/s]		u (depth averaged) [m/s]	
					0,5		1	
soil type	d_{50}	d_*	u_{*crit}	$u_{*transition}$	$u_{*surrounding\ bed}^*$	$u_{*channel\ bottom}^*$	$u_{*surrounding\ bed}^*$	$u_{*channel\ bottom}^*$
	[μm]	[-]	[m/s]	[m/s]				
MSI	50	1,07	0,0137	0,0109	0,0245	0,0109	0,0489	0,0218
MFS	200	4,27	0,0135	0,0236	suspended	no transport	suspended	suspended
MMS	500	10,69	0,0161	0,0248	suspended	no transport	suspended	bed load
					bed load	no transport	suspended	bed load

When looking at slope development, this division in bed load and suspended transport will turn out to be important. Now, both types of transport are described with special attention to the influence of a sloping bottom, starting with bed load transport.

8.2.2 Bed load transport

Van Rijn describes the bed load transport due to currents as follows:

$$q_b = 0,25 \rho_s d_{50} d_*^{-0,3} \left[\frac{\tau'_{b,c}}{\rho_w} \right]^{0,5} \left[\frac{\tau'_{b,c} - \tau_{b,cr}}{\tau_{b,cr}} \right]^{1,5} \quad \text{Eq. 8-3}$$

in which

q_b	= bed-load transport;	[kg/ms]
$\tau'_{b,c}$	= grain-related bed-shear stress due to current;	[N/m ²]
$\tau'_{b,crit}$	= critical grain-related bed-shear stress;	[N/m ²]

The grain-related bed-shear stress can be deduced from the efficiency factor of the current and the bed-shear stress as follows:

$$\tau'_{b,c} = \mu_c \tau_{b,c} \quad \text{Eq. 8-4}$$

in which

$\tau_{b,c}$ = critical bed-shear stress according to Shields;
 μ_c = efficiency factor current, described by:

$$\mu_c = \frac{f'_c}{f_c} = \frac{0,24 \log^{-2}(12h/3d_{90})}{0,24 \log^{-2}(12h/k_{r,c})} \quad \text{Eq. 8-5}$$

Influence of sloping bottom on bed load transport

The influence of a sloping bottom was already proved to influence the critical bed shear stress, see Paragraph 7.1.2. Downward sloping bottoms reduced the threshold of motion, hence $K_{\alpha\beta}$ was smaller than 1; upward sloping bottoms were favourably influenced by gravity, so $K_{\alpha\beta} > 1$. Also, the bed load transport rates are affected by a sloping bottom and should be multiplied by a slope correction factor. This so-called Bagnold-factor is larger than 1 for downward sloping bottoms (β positive) and smaller than 1 for upward sloping bottoms (β negative) and can be described for the current component directed perpendicular to the channel axis as follows:

$$K_{\beta 2} = \frac{\tan \phi}{\tan \phi - \tan \beta} \quad \text{Eq. 8-6}$$

So, sediment particles located at a downward sloping bottom not only are more easily picked up by a current, but also more easily transported as bed-load transport. Suspended sediment transport has no contact with the seabed and will not be affected by a sloping bottom.

8.2.3 Dynamic Equilibrium Slopes

In Paragraph 7.1.3 a threshold profile was defined. It was concluded that this profile is independent on grain size, because the critical discharge at the surrounding seabed was used. Although the values of these critical discharges differed for different grain sizes, the shape of the threshold profile was the same. According to the same line of thought, the question arises whether an 'equal sediment transport-profile' can be defined. If such a profile exists, this means that the entire cross-section of the channel is in equilibrium: just as much sediment is transported at the surrounding seabed, at both side slopes and at the channel bottom. In this situation no morphological changes occur, at least as long as the hydrodynamic conditions remain the same.

To come to such a profile, one should first realise that it can only be deduced for bed-load transport, which requires sufficient large particle sizes and sufficient small bed shear velocities, see Table 8-1 and Table 8-2. Suspended transport is not taken into consideration, because this type of transport isn't able to adapt instantaneously to the changing seabed. Furthermore, suspended transport is not just dependent on bed load transport, but also on the height of the water column, as will be explained later on. This implies that although the bed load transport at two different depths (and different sloping sea beds) can be the same, but the suspended transport will differ due to a different transport capacity at different water depths. So, the first assumption is that only bed-load transport is allowed and therefore this calculation will be executed for a grain diameter of 500 μm .

The second assumption contains logarithmic velocity profiles across the channel. The influence of a sloping bottom is only represented by two slope correction factors: Schoklitsch-factor $K_{\alpha\beta}$ (threshold of motion) and Bagnold-factor $K_{\beta 2}$ (bed-load transport).

Furthermore, a constant unidirectional current ($u = 1\text{m/s}$) and a water depth of 10 m is assumed. At this surrounding seabed a certain equilibrium bed load transport can be calculated according to formula 8-3:

$$q_b = 0,005 \sqrt{\tau'_{b,c}} \left[\frac{\tau'_{b,c} - \tau_{b,cr}}{\tau_{b,cr}} \right]^{1,5} \quad \text{Eq. 8-7}$$

At a water depth of 10m, this results in $q_{b,h0} = 54,0 \text{ g/ms}$. When a current enters a navigation channel at the upstream side slope, this bed load transport can only remain

constant if the negative influence of the increasing water depth is counterbalanced by the positive influence of the downward sloping bottom. This steepening effect is demonstrated in Figure 8-1 for four discharges. Due to the additional Bagnold-factor, the 'equilibrium' side slopes become somewhat smaller (=flatter) for larger discharges. At infinite water depth, of course the angle of internal friction is reached.

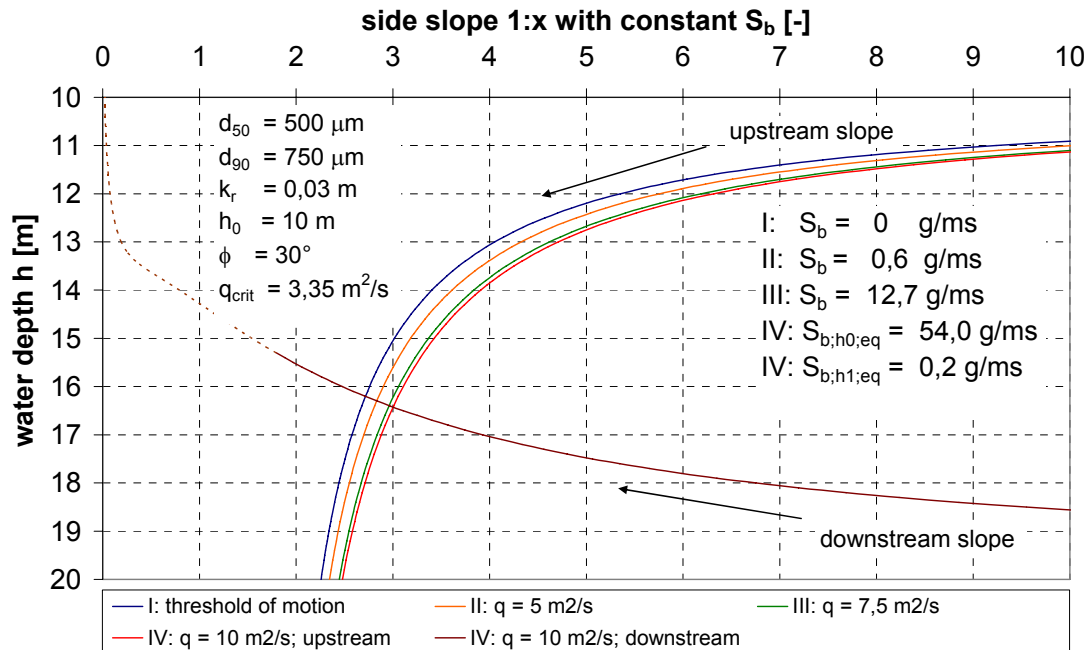


Figure 8-1: Side slopes, for which the bed-load transport is constant, against water depth with perpendicular currents of 3,35 m²/s (threshold of motion on plane bed), 5,0 m²/s, 7,5 m²/s and 10 m²/s

As the water depth is increasing, the upstream side slope becomes steeper and steeper, until the channel bottom is reached. The bed load transport inevitably has to adapt to the local conditions. The equilibrium bed load transport at the channel bottom is: $q_{b;h1} = 0.203 \text{ g/ms}$. This abrupt change in sediment transport would cause an enormous accretion at the channel bottom near the toe of the upstream slope and this uncoupled approach of a constant bed load transport at the surrounding bed and upstream side slope and a constant bed load transport at the channel bottom is therefore not very realistic. One would observe a gradually flattening lower half of the upstream side slope. A true equilibrium profile cannot exist, because the lower part of the upstream side slope 'feels' the channel bottom. As siltation proceeds, a larger and larger part of the upstream slope starts feeling the bottom and the above described slope steepening process becomes less pronounced.

As the current flows across the channel bottom, it becomes totally adapted to the equilibrium bed load transport at the channel bottom, which is in fact very small. When the current arrives at the downstream side slope an inverse process occurs: due to the decreasing water depth and the corresponding increasing bed shear stress, the sediment particles need a stabilizing gravity force to maintain the amount of bed load transport. The downstream side slope becomes steeper, until the angle of repose is reached. The side slope cannot respond to the increasing erosive capacity of the current by increasing the gravity component anymore and erosion of the upper part of the side slope is inevitable. This effect will extend over a very large distance downstream of the trench, so this approach seems only valid for the upper part of the upstream slope and the lower part of the downstream slope. The lower part of the upstream slope will be subjected to large sedimentation and the upper part of the downstream slope to large erosion, until the navigation channel has completely silted up and the initial, flat equilibrium seabed is restored.

Before this discussion is proceeded, one should be aware that no influence of waves, gravity effects or velocity profiles deviating from the logarithmic profile are taken into account.

This brief explanation can be described in symbols, starting from an initial channel geometry with side slopes β_{slope} . The subscripts 'sb', 'cb', 'eq', 'up' and 'down' respectively refer to the surrounding seabed, channel bottom, equilibrium, upstream and downstream slope. Please note that $\beta_{\text{up;eq}}$ and $\beta_{\text{down;eq}}$ are functions of water depth. In direction of the current, the following processes can be observed, starting from initial straight side slopes:

- ⇒ top of upstream slope:
 - ⇒ $\beta_{\text{slope}} > \beta_{\text{up;eq}}$
 - ⇒ $S_{b;\text{slope}} > S_{b;\text{sb;eq}}$
 - ⇒ erosion
 - ⇒ $\beta_{\text{slope}} \downarrow \rightarrow \beta_{\text{up;eq}}$
 - ⇒ slope flattening until $\beta_{\text{up;eq}}$ is reached
- ⇒ somewhere along upstream slope:
 - ⇒ $\beta_{\text{slope}} = \beta_{\text{up;eq}}$
 - ⇒ $S_{b;\text{slope}} = S_{b;\text{sb;eq}}$
 - ⇒ no bed level or slope steepness change
- ⇒ toe of upstream slope:
 - ⇒ $\beta_{\text{slope}} < \beta_{\text{up;eq}}$
 - ⇒ $S_{b;\text{slope}} < S_{b;\text{sb;eq}}$
 - ⇒ sedimentation
 - ⇒ $\beta_{\text{slope}} \downarrow \rightarrow \beta_{\text{up;eq}}$
 - ⇒ slope steepening until $\beta_{\text{up;eq}}$ is reached
- ⇒ channel bottom:
 - ⇒ $\beta_{\text{slope}} = 0 < \beta_{\text{up;eq}}$
 - ⇒ $S_{b;\text{sb;eq}} > S_{b;\text{cb;eq}}$
 - ⇒ sedimentation until $S_{b;\text{cb;eq}}$ is reached
- ⇒ toe of downstream slope:
 - ⇒ $\beta_{\text{slope}} > \beta_{\text{down;eq}}$
 - ⇒ particles experience positive influence of gravity
 - ⇒ $S_{b;\text{slope}} < S_{b;\text{cb;eq}}$
 - ⇒ sedimentation
 - ⇒ $\beta_{\text{slope}} \downarrow \rightarrow \beta_{\text{down;eq}}$
 - ⇒ slope flattening until $\beta_{\text{down;eq}}$ is reached
- ⇒ somewhere along downstream slope:
 - ⇒ $\beta_{\text{slope}} = \beta_{\text{down;eq}}$
 - ⇒ $S_{b;\text{slope}} = S_{b;\text{cb;eq}}$
 - ⇒ no bed level change
- ⇒ somewhere further along downstream slope:
 - ⇒ $\beta_{\text{slope}} < \beta_{\text{down;eq}} \leq \phi$
 - ⇒ $S_{b;\text{slope}} > S_{b;\text{cb;eq}}$
 - ⇒ erosion
 - ⇒ $\beta_{\text{slope}} \uparrow \rightarrow \beta_{\text{down;eq}}$
 - ⇒ slope steepening until $\beta_{\text{down;eq}}$ is reached

Eventually the required slope angle $\beta_{\text{down;eq}}$ to maintain equal bed load transport can become larger than ϕ . It is however not very likely that slopes steeper than the angle of internal friction can sustain. Once put into motion, the particles will rain down along the slope.

- ⇒ somewhere along downstream slope where $\beta_{\text{down;eq}} \geq \phi$
 - ⇒ $\beta_{\text{slope}} \leq \phi \leq \beta_{\text{down;eq}}$
 - ⇒ $S_{b;\text{slope}} >> S_{b;\text{cb;eq}}$
 - ⇒ continuous erosion
 - ⇒ $\beta_{\text{slope}} \uparrow$

Once again, this is a very schematized approach, but it shows that slopes tend to a certain slope steepness to minimize the gradients in sediment transport. After the side slopes have reached their dynamic equilibrium, the sedimentation of the entire channel still proceeds. The side slopes start feeling the channel bottom more and more and

finally the side slopes start feeling each other. In fact, one can not longer speak of side slopes and a channel bottom, it is just a pit in the seabed.

This idea of dynamic equilibrium side slopes based on constant bed-load transport shows some realistic 'smooth' slopes, but will initial straight slopes in reality develop to these equilibrium slopes? The simulations in SUTRENCH in Chapter 10 will resolve these questions.

8.2.4 Suspended transport

Once sediment particles are mixed over the vertical, one speaks of suspended sediment transport. The suspended sediment transport is the product of the velocity distribution and the concentration distribution, integrated from the reference level a to the water depth h .

$$q_b = \int_a^h u c dz \quad \text{Eq. 8-8}$$

in which the velocity profile is described by the logarithmic Von Karman-profile and Van Rijn describes the concentration profile at water depths larger than the reference height as follows:

$$\frac{dc}{dz} = - \frac{(1-c)^5 c w_s}{\epsilon_{s,c}} \quad \text{Eq. 8-9}$$

in which

c = concentration

w_s = particle fall velocity, described in Paragraph 3.1.2

$\epsilon_{s,c}$ = sediment mixing coefficient for current only, described in Paragraph 8.4

As a boundary condition the reference concentration ' c_a ' at a height ' a ' above the bed will be used. In fact, the bed load transport is the product of the thickness of the bed load layer (or saltation height) ' δ ', the velocity of the bed-load particles and the bed load concentration. To avoid large errors in the concentration profile, the effective reference concentration at a somewhat larger height (reference height) above the bed is used, although no physical basis exists.

$$q_b = c_b u_b \delta = c_a \bar{u}_a a \quad \text{Eq. 8-10}$$

This effective reference concentration can be expressed as:

$$c_a = \frac{q_b}{\bar{u}_a a} = 0,015 \rho_s d_*^{-0,3} \frac{d_{50}}{a} \left[\frac{\tau'_{b,w} - \tau_{b,cr}}{\tau_{b,cr}} \right]^{1,5} \quad \text{Eq. 8-11}$$

In principle this bed load layer is a layer just above the bed in which the mixing is so small that it cannot affect the particles, so it is impossible for them to get into suspension. According to Van Rijn, this thickness of the bed load layer (' a ') is the maximum of the current-related and wave-related roughness, which are considered to be equal throughout this thesis.

8.3 Combined action of currents and waves

Although waves can cause a net transport sediment transport, they will be most effective in stirring up sediment. In combination with a current, the sediment transport will heavily increase. It was already mentioned that in this thesis currents and waves act as a sediment pick-up mechanism and that only currents act as a convection mechanism, see Figure 8-2.

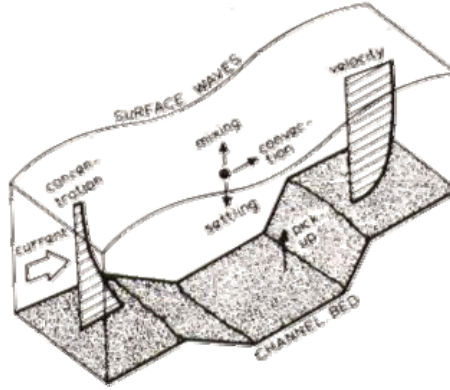


Figure 8-2: Sediment transport mechanisms

8.3.1 Bed load transport

The formula of the bed load transport changes somewhat if waves are superimposed on a current. A calibration factor is added and the (time-averaged) bed shear stress represents the influence of both waves and current, according to Paragraph 7.3.

$$q_b = 0,25 \alpha_H \rho_s d_{50}^{-0,3} \left[\frac{\tau'_{b,cw}}{\rho_w} \right]^{0,5} \left[\frac{\tau'_{b,cw} - \tau_{b,cr}}{\tau_{b,cr}} \right]^{1,5} \quad \text{Eq. 8-12}$$

in which:

α_H = calibration factor ($=1-(H_s/h)^{0,5}$);
 $\tau'_{b,cw}$ = grain-related time-averaged bed-shear stress due to current and waves.

8.3.2 Suspended transport

The formula of suspended transport (Eq. 8-8) still holds, but the velocity and concentration profile will change. The logarithmic velocity profile will be distorted by the orbital velocity of the waves. To represent the real situation best, the instantaneous transport should be calculated within a wave cycle and then averaged over the wave period. The applied numerical model (SUTRENCH, see Chapter 9) is based on time-averaged wave influence. This means that the logarithmic velocity profile of 'current only' is used for the situation with current and waves. The concentration profile is modified through a different sediment mixing coefficient, see the next paragraph.

8.4 Sediment mixing coefficient

The sediment mixing coefficient is related to the fluid mixing coefficient according to the following expression:

$$\varepsilon_s = \beta \phi \varepsilon_f \quad \text{Eq. 8-13}$$

in which:

ε_s = sediment mixing coefficient	[m ² /s]
ε_f = fluid mixing coefficient	[m ² /s]
β = ratio sediment mass and fluid momentum mixing coefficients	[-]
ϕ = turbulence damping factor	[-]

Factor ' β ' represents the difference in mixing of a fluid 'particle' and a sediment particle. Often values larger than 1 have been found in experiments, especially for large u_*/w_s -values. This can be explained by the fact that sediment particles are thrown out of the turbulent eddies. For small u_*/w_s -values the sediment diffusivity tends to become smaller than the fluid diffusivity, so $\beta < 1$. Because factor β cannot be predicted with high accuracy, in this thesis a value of 1 will be used.

Factor ' ϕ ' represents the damping of turbulence on the velocity profile by the sediment particles and the adaptation of the sediment diffusion coefficient to the sediment concentration. The latter appears to be most important, however, this influence remains limited for sediment concentrations smaller than 10 g/l. Therefore this influence will be neglected in this thesis: $\phi = 1$. So, the sediment mixing coefficient is only dependent on the fluid mixing coefficient: $\varepsilon_s = \varepsilon_f$.

For gradually varying flows simple mixing coefficient distributions for equilibrium conditions are applied:

Current alone

Instead of a parabolic distribution, a parabolic-constant distribution is used, see also Figure 8-3:

$$\begin{aligned} \varepsilon_{s;c} &= \varepsilon_{s;c;\max} - \varepsilon_{s;c;\max} \left(1 - \frac{2z}{h}\right)^\eta & \text{for } \frac{z}{h} < 0,5 \\ \varepsilon_{s;c} &= \varepsilon_{s;c;\max} = 0,25\beta\kappa u_* h & \text{for } \frac{z}{h} \geq 0,5 \end{aligned} \quad \text{Eq. 8-14}$$

in which:

η = coefficient which equals 2 for the situation of a current alone;
 u_{*c} = bed-shear velocity of a logarithmic velocity profile:

$$u_{*c} = \frac{\kappa}{1 + \ln\left(\frac{h}{0,03k_r}\right)} \frac{Q}{bh} \quad \text{Eq. 8-15}$$

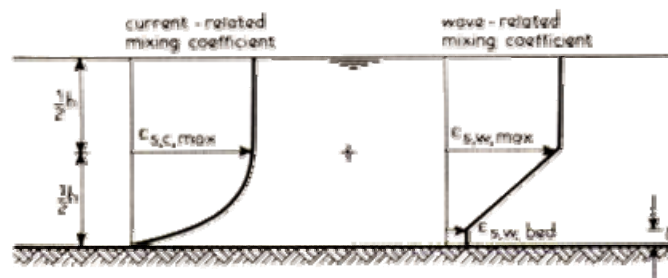


Figure 8-3: Current- and wave-related mixing coefficient

Waves alone

The wave-related mixing coefficient is constant for the upper half of the water depth. Furthermore the sediment mixing coefficient is constant in the wave boundary layer. In between there is a linear relation:

$$\begin{aligned} \varepsilon_{s,w} = \varepsilon_{s,w,bed} &= 0,00065 d_*^2 \alpha_{br} \delta \hat{u}_0 & \text{for } z \leq \delta \\ \varepsilon_{s,w} = \varepsilon_{s,w,max} &= 0,035 \alpha_{br} \frac{h H_s}{T'_s} & \text{for } z \geq 0,5h \\ \varepsilon_{s,w} = \varepsilon_{s,w,bed} + (\varepsilon_{s,w,max} - \varepsilon_{s,w,bed}) &\left(\frac{z - \delta}{0,5h - \delta} \right) & \text{for } \delta \leq z \leq 0,5h \end{aligned} \quad \text{Eq. 8-16}$$

in which:

α_{br} = breaking coefficient, representing the influence of breaking waves on the sediment mixing process = 1, if $H_s/h < 0,6$;

d_* = dimensionless particle diameter, according to Eq. 7-2;

$\hat{u}_0$ = amplitude of orbital velocity at the sea bed, according to Eq. 7-17;

T'_s = significant wave period (relative to moving coordinate system).

Current and waves

When a wave field is superimposed on a current, the overall sediment mixing coefficient is obtained by linear addition of the wave-related and current-related mixing coefficients:

$$\varepsilon_{s,cw} = \varepsilon_{s,c} + \varepsilon_{s,w} \quad \text{Eq. 8-17}$$

The wave-related mixing coefficient is assumed to still satisfy equation 8-16; the current-related mixing coefficient is expected to adjust to the presence of waves. Power ' η ' in equation 8-14, which for the current-alone case equals 2 (parabolic distribution), is now assumed to be in the range of 1 to 2:

$$\begin{aligned} \eta &= -0,25 \frac{\hat{u}_0}{|\bar{u}|} + 2 & \text{for } 0 \leq \hat{u}_0 \leq 4|\bar{u}| \\ \eta &= 1 & \text{for } \hat{u}_0 > 4|\bar{u}| \end{aligned} \quad \text{Eq. 8-18}$$

8.5 Channel morphology

Some characteristic phenomena, that will be observed later on, will be explained in more detail.

Direction of sediment transport

As was mentioned in Chapter 6 on hydrodynamics, the direction of the current and waves to the channel axis determines the current and wave action inside a navigation channel or pipeline trench to a large extent. Therefore, also sediment transport capacities are affected by the channel alignment. If the sediment transport is directed parallel to the channel axis, the transport capacity inside the channel will be larger than outside the channel. The morphology however is governed by cross-channel transport, although there isn't any cross-channel component of the current velocity. The deposition of side slope particles on the channel bottom occurs mainly due to the action of gravity. This effect cannot properly be modelled and only currents with a significant cross-channel component will be considered in this thesis.

Trapping efficiency

The trapping efficiency factor is defined as the relative difference of the incoming suspended transport and the minimum suspended transport (somewhere) in the channel:

$$e = \frac{b_0 S_{s0} - b_1 S_{s1, \text{minimum}}}{b_0 S_{s0}} \quad \text{Eq. 8-19}$$

in which

b_0 = width of approaching streamtube;

b_1 = width of streamtube in channel;

S_{s0} = incoming suspended load transport per unit width;

S_{s1} = suspended load transport in channel per unit width.

The basic parameters which strongly determine this trapping efficiency factor are the approach angle (α_0), the approach bed-shear velocity (u_{*0}), the particle fall velocity (w_s), the channel depth ($h_1 - h_0 = D$), the channel width (W) and of course the channel side slope ($\tan \beta$). Other parameters are of less significance [Van Rijn, 1986a], such as the approach velocity (u_0), the relative wave height (H/h_0) and the relative roughness (k_r/h_0).

In the framework of this thesis, it is also relevant to define the trapping efficiency of the side slope (e_{slope}). While the general definition of the trapping efficiency is a measure for the overall rise of the channel bottom and the maintenance costs to be expected, the specific definition of the trapping efficiency of the side slope is important for the development of these side slopes. If this trapping efficiency is small, suspended transport does not affect slope development much and the bed load transport is normative. If this trapping efficiency is large, bed load and suspended transport control slope development. The above of course only holds if bed load and suspended transport are of the same order, which is the case for fine sands under most hydrodynamic conditions. Slope development in medium to coarse sands can be attributed to bed load transport; in silts bed load transport is small compared to suspended transport.

$$e_{\text{slope}} = \frac{b_0 S_{s0} - b_{\text{toe}} S_{s;\text{toe}}}{b_0 S_{s0}} \quad \text{Eq. 8-20}$$

in which

b_{toe} = width of streamtube at toe of (upstream) side slope;

$S_{s;\text{toe}}$ = suspended load transport at toe of (upstream) side slope per unit width.

Slope development and trench migration

Slopes will change due to sediment transport and gravity effects. The more a slope differs from its dynamic equilibrium profile, the faster slope development will take place. Trench migration will occur in case of unidirectional flow or in asymmetrical tidal flow and can cause problems if the channel differs too much from the initial alignment. This migration however is very hard to define in a satisfactory way. Most problems arise from the fact that the slope geometry itself does not keep its initial shape. So, the migration of which part of the slope or the channel has to be considered? More practical or more theoretical definitions can be used. Theoretical definitions tend to describe the migration of a mathematically formulated channel geometry. Practical definitions are related to a simpler application in practice and to the actual functionality of a navigation channel or pipeline trench, which is often based on providing a required depth over a certain width.

In Jensen [1999] the migration of the centre of gravity of the channel geometry below the surrounding seabed is used as a measure for channel migration. This is a very sound theoretical measure, but is rather laborious and does not provide any information on the functionality of the channel.

Walstra et al. [2002] use another mathematical description of the channel geometry to express the channel development. A logarithmic function through the lower inflection points on the side slopes is used to describe the channel geometry. This approximation shows reasonable resemblance to actual profiles, but is also rather laborious and provides little information on the slope steepness, which is an important parameter of this thesis. So also, this method has on purpose not been adapted here.

But what are satisfactory measures of slope development and trench migration with respect to the objectives of this research study on slope development? When dealing with slope stability the steepest parts of the slope will be normative. However, if this steepest parts only stretch over a limited height, the consequences will be minor, unless a very erosive turbidity current develops. Besides, the location of the steepest part along the slope is of importance. A steep toe is more susceptible to static liquefaction than a steep top. Of course this also depends on the soil properties.

So a method to describe slope development and trench migration that is completely legitimate in all cases and that is easily applicable in engineering practice perhaps does not exist.

Therefore the steepest slope over a cross-channel width of 4 to 5 m (= 3 grid points) is taken as a measure for the slope steepness. Often is found that this steepness extends over a far greater height. This method, resulting in just one value, is complemented with a graphical observation of the developed slopes.

The trench migration should of course be related to the navigability and is therefore defined as the migration of that point at the upstream side slope where 90% of the initial channel depth with respect to the surrounding seabed ' $h_1 - h_0$ ' (which is equal to the slope height ' D ') is reached.

In this thesis not much attention is given to the morphological influence of the trench on the surroundings, which was the case when considering large scale morphological mining pits [Walstra et al., 2002]. The dimensions of navigation channels and pipeline trenches are so small that this morphological influence will remain limited. As a consequence the exact total volume of a trench is not as important as the useful volume (required depth times the width at which this depth is guaranteed) for practical reasons of navigability.

9 SUTRENCH, model description

9.1 Introduction

In rivers or narrow tidal inlets one can apply simple 1-dimensional (depth-averaged) models to calculate the general hydraulic conditions. In seas and wide estuaries 2-dimensional models have to be used, which are mostly depth-averaged. In Chapter 6 on hydrodynamics was already shown that currents refract when entering the channel due to the instantaneous effect of continuity and the more gradual effect of longitudinal acceleration. In 2DV-models this effect is accounted for by an increase of the apparent channel width. This increase is in reality a slowly developing process over the channel width. Quasi-3D-models can capture the depth-averaged current refraction, based on assumptions of hydrostatic pressure distributions and logarithmic velocity profiles.

Besides this depth-averaged current refraction, on the side slopes also a distortion over the vertical velocity profile occurs; the bed shear stress will be more refracted on the side slopes than the surface velocity. This effect will be more significant if side slopes become steeper. Quasi-3D-models can only account for unidirectional horizontal velocities. A full 3D-model is necessary to capture the complete process.

In this thesis the stability of a trench and its slopes will be treated as a 2-dimensional problem (the 'slices' along the channel axis are all similar). This assumption to deal with the problem in a 2DV-environment implies that primarily cross-channel flow will be modelled and that oblique currents have to be changed into cross-channel currents with adapted stream widths ' b_1 ' and channel widths ' W '. With this trick, sediment transport can be reasonably modelled, although the difference in direction between the depth-averaged velocity and the bed shear stress is neglected. Parallel currents or currents with very small angles to the channel axis cannot be simulated. The width-averaged 2DV-program SUTRENCH by Delft Hydraulics will be applied to model the morphology of a channel.

Before continuing with the model description, one should realize that at the present state of research the application of numerical models that are not calibrated to specific field conditions has a very limited predicting value, at least when it comes to *absolute* values. Most numerical models, however, represent a satisfactory to good *relative* behaviour over wide ranges of wave and current conditions, as was concluded by Davies et al. [2002] within the framework of the SEDMOC Project. The TRANSPOR-model, which is used in SUTRENCH, proved to yield results that are in the middle of the results of all tested models. When compared to actual field measurements, more than half of the calculated results lie within a confidence band of a factor 2 (and the resemblance is even better for depths between 5 and 15 m and significant sediment concentrations, which is the range of interest of this thesis. Moreover, the *relative* behaviour is assumed to resemble nature very well.

This means that the time scales of the development of a trench and its side slopes in particular may be somewhat inaccurate, but the occurring changes in profile and slopes should be reliable, at least for channel geometries that are not too sharp-edged.

9.2 Model description

SUTRENCH (Van Rijn and Tan, 1985; Van Rijn, 1986) is a two dimensional vertical mathematical model for simulation of bed-load and suspended load transport under the conditions of steady state currents and wind-induced waves. It can be applied at space-scales from 1 to 5 kilometres and at time-scales from 1 to 100 years in regions outside the surf zone where wave breaking is limited. On the other hand, knowledge on sediment transport in deeper water (>20 m) is still limited. Therefore in this thesis only water depths between 5 and 20 will be considered.

In this thesis SUTRENCH will be applied at the lower limits of its space and time scales: the emphasis will lay on space scales around 800-1200 m in order to minimize grid spaces and on time scales up to one year in order to study the short-term development.

The SUTRENCH model is restricted to well-mixed gradually varying flow conditions over a sediment bed consisting of fine particles with narrow size gradation ($d_{90}/d_{10} = \pm 3$). In this thesis a gradation of $d_{90}/d_{10} = 1,5$ (see Paragraph 3.1.2) is applied. The model is applicable with or without waves. Tidal flow can be represented by schematizing the real tidal period to quasi-steady flow periods. SUTRENCH uses the TRANSPOR-2000 sediment transport model. During the SEDMOC-project TRANSPOR-1993 has been improved to this newer version.

Basic processes

Hydrodynamic

- Modification of velocity profile and associated bed-shear stress due to the presence of waves;
- Modification of velocity and associated bed-shear stress due to the presence of sloping bottom.

Sediment transport

- Advection by horizontal and vertical mean current;
- Vertical mixing (diffusion) by current and waves;
- Settling by gravity;
- Entrainment of sediment from bed due to wave- and current-induced stirring;
- Bed-load transport due to combined current and wave velocities (instantaneous intra-wave approach);
- Slope-related transport components (bed-load);
- Effect of mud on initiation of motion of sand;
- Non-erosive bottom layers.

Basic simplifications

Hydrodynamic

- Logarithmic velocity profiles and associated bed-shear stress also in conditions with waves: steep sided trenches and channels (steeper than 1:5) cannot be modelled correctly;
- Shoaling and refraction of wind waves is not implicitly modelled, but can be taken into account by the input data;
- Current refraction (veering) is not implicitly modelled.

Sediment transport

- Steady state sediment mass conservation integrated over the width of the flow (stream tube approach);
- No longitudinal mixing (diffusion);
- No wave-related suspended sediment transport (no oscillatory transport components)
- Uniform grain size (no mixtures or grain distributions).

Numerical

- Forward-marching numerical scheme (transport due to near-bed return currents can not be modelled);
- Explicit Lax-Wendorff numerical scheme for bed level changes (smoothing effects).

Drawbacks

- SUTRENCH becomes unstable for very small waves and currents
- a frequent problem of 'simple' numerical transport models is also the underestimation of sediment concentrations compared to field conditions. This is attributed to a residual amount of suspended sediment in calm conditions (e.g. near slack water) caused by a certain existing background level of turbulence which is not included in the model formulation

Continuity equation for suspended sediment

The time-averaged sediment convection-diffusion equation in the 2DV-plane can be described as follows:

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x} uc - \frac{\partial}{\partial x} \left(\varepsilon_{s,cw} \frac{\partial c}{\partial x} \right) + \frac{\partial}{\partial z} ((w - w_s)c) - \frac{\partial}{\partial z} \left(\varepsilon_{s,cw} \frac{\partial c}{\partial z} \right) = 0 \quad \text{Eq. 9-1}$$

in which:

- u = longitudinal velocity at height z above the bed;
- c = sediment concentration;
- w = vertical flow velocity;

If one assumes steady state conditions, neglects longitudinal mixing (usually an order of magnitude smaller than the other terms) and applies a stream tube approach to account for diverging or converging flows, this equation reduces to:

$$\frac{\partial}{\partial x} buc + \frac{\partial}{\partial z} (b(w - w_s)c) - \frac{\partial}{\partial z} \left(b \varepsilon_{s,cw} \frac{\partial c}{\partial z} \right) = 0 \quad \text{Eq. 9-2}$$

in which:

- b = width of stream tube;

If the flow is in equilibrium and the width of the stream tubes is constant, equation 9-2 reduces to:

$$cw_s + \varepsilon_{s,cw} \frac{\partial c}{\partial z} = 0 \quad \text{Eq. 9-3}$$

But most of the times the flow will be gradually changing and equation 9-2 has to be solved numerically. Therefore the flow width b as a function of distance x must be known a priori. Also the flow velocities, the sediment mixing coefficients and the particle fall velocities should be known. All but the vertical flow velocity have been discussed before. The vertical flow velocity can be computed from the (width-integrated) equation of continuity of the fluid:

$$w = - \int_{z_b+z_0}^{z_b+z} \frac{\partial u}{\partial x} dz - \frac{1}{b} \frac{db}{dx} \int_{z_b+z_0}^{z_b+z} \frac{\partial u}{\partial x} u dz \quad \text{Eq. 9-4}$$

Boundary conditions

All necessary boundary conditions will be mentioned briefly.

Flow domain

The initial bottom level 'z_b', the water level 'h' and the flow width 'b' have to be defined.

Inlet boundary

The location of the inlet boundary should not experience any morphological changes. This means that for unidirectional flow the inlet boundary can be close to the upstream slope, while for tidal flow the inlet boundary also acts as the outlet boundary; then the location should be as far away from the area of interest as possible. However, one wants to describe the relative sharp channel geometry in a sufficient fine mesh (small Δx). Due to a limited number of grid points and a constant grid size, this means that the channel geometry has to be carefully positioned in calculation space for every calculation. For these types of problems an adjustable grid size would be appropriate, especially in tidal flow.

Assuming logarithmic velocity profiles, only the discharge 'Q' has to be defined for the fluid. The sediment concentration $c(z,t)$ is in this thesis considered to be in equilibrium, although also measured or zero-concentration profiles can be set at the inlet boundary.

Outlet boundary

The location of the outlet boundary should be far away from the downstream slope, taking the above considerations into account.

Water surface

No vertical sediment transport can take place through the water surface:

$$\left(w_s c + \varepsilon_s \frac{\partial c}{\partial z} \right)_{z=z_b+h} = 0 \quad \text{Eq. 9-5}$$

Bed boundary

The vertical flow velocity at the zero-velocity level ($z_0 = 0,033k_r$) has to be zero: $w=0$ at $z=z_0$. For the sediment transport the equilibrium bed concentration ' $c_{a,e}$ ' and the upward sediment flux E_a have to be specified. The bed concentration can be calculated according to formula 8-11 and is plotted in Figure 9-1. The sediment flux (gradient of concentration profile at reference level 'a') can be deduced from equation 9-3:

$$E_a = \left(-\varepsilon_s \frac{\partial c}{\partial z} \right)_a = -0,015 w_s d_*^{-0,3} \frac{d_{50}}{a} \left(\frac{\tau'_{b,cw} - \tau_{cr}}{\tau_{cr}} \right)^{1,5} \quad \text{Eq. 9-6}$$

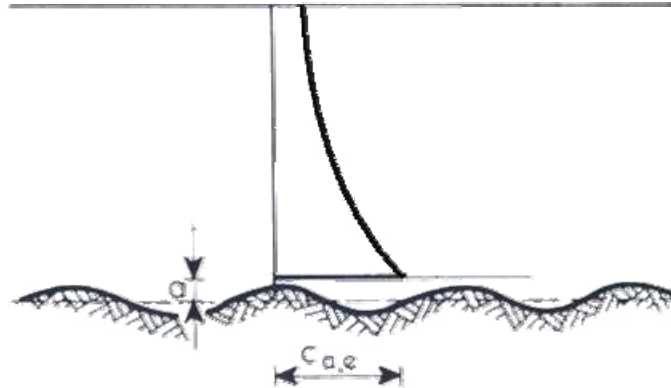


Figure 9-1: Schematization of equilibrium concentration profile and bed boundary condition at reference level 'a'

Bed level changes

If the concentration profiles and the bed load and sediment transport are known at every location, the bed level changes can be computed:

$$\frac{\partial b z_b}{\partial t} + \frac{1}{\rho_s(1-n)} \left[\frac{\partial b h \bar{c}}{\partial t} + \frac{\partial S_{tot}}{\partial x} \right] = 0 \quad \text{Eq. 9-7}$$

in which:

$\bar{c}$ = depth-averaged concentration

S_{tot} = total sediment transport = $S_b + S_s$

The storage term ' $\partial b h \bar{c} / \partial t$ ' can be neglected if flow conditions are considered quasi-steady.

Numerical solution

The continuity equation for local sediment (Eq. 9-2) and the equation to compute bed level changes (Eq. 9-7) have to be solved numerically.

Therefore a 2-dimensional grid (Δx in cross-channel direction and Δz in direction of depth) is defined. The horizontal grid size is constant, while the vertical grid size decreases towards the bed in order to obtain a greater resolution where largest gradients exist. Some rough guide lines are:

Δx : at least 10 grid points over the characteristic length scale (side slope). In this thesis cross-channel grid sizes of 2 to 2,5 m are used, which is rather small at locations where flow conditions are constant, but may be rather large at the side slopes, especially when the side slopes are very steep.

Δz : at least 10 grid points for current-alone conditions and at least 20 points if waves are superimposed on a current. In this thesis values no larger than 15 are applied, because there is a restriction to the total number of grid points. The formula of maximum calculation space is:

$$5(8N + 4NV + N \cdot NV) \leq 50.000 \quad \text{Eq. 9-8}$$

in which:

N = total horizontal grid points;
 NV = total vertical grid points.

This restriction to calculation space yields for a length scale of 1 km (with $\Delta x = 2,5\text{m}$) a number of 400 horizontal grid points and thus maximum 16 vertical grid points. This example demonstrates that it is very hard to fulfil all requirements.

The numerical scheme (Lax-Wendroff) that is used to compute the new bed level at time $t+\Delta t$ is presented to show how SUTRENCH handles with sharp transitions in bed level, like the side slopes:

$$z_{b,x}^{t+\Delta t} = z_{b,x}^t - \frac{N_t \Delta t}{2(1-n)\rho_s b \Delta x} (S_{x+\Delta x}^t - S_{x-\Delta x}^t) + \frac{1}{2} \alpha_s (z_{b,x+\Delta x}^t - 2z_{b,x}^t + z_{b,x-\Delta x}^t) \quad \text{Eq. 9-9}$$

in which

α_s = bed-level smoothing coefficient
 N_t = number of time steps between computation of two subsequent bed levels
 $z_{b,x}^t$ = bed level at a certain time t and location x
 S_x^t = total sediment transport at a certain time t and location x

This numerical scheme implies that due to numerical smoothing even with zero or very small sediment transport bed level changes occur. During simulations of various geometries and soils, numerical problems would arise. It appeared that a very clever choice of smoothing coefficient ' α_s ', time step and calculation space for every different situation had to be made to deal with numerical diffusion or instability.

If the problem is perfectly suitable for computation with SUTRENCH, the numerical smoothing coefficient can be chosen very small. However, sometimes, especially in unidirectional flow, instabilities can occur due to oversteepening of the upstream slope. This oversteepening phenomenon of course is dependent on sediment transport and, more particular, the transport gradient. In reality some smoothing processes on very steep slopes are very likely to occur, so the application of a numerical smoothing coefficient is in some way legitimate, although there is no physical meaning. This artificial trick has some major consequences for the time step: if the time step is reduced, the numerical smoothing process starts to prevail over the real sediment transport process: every time step the bed level is smoothed with a constant rate,

while the sedimentation/erosion rates decrease, if the time step is reduced. So reducing the time step should be accompanied by an equal reduction of the numerical smoothing coefficient.

Concluding, it can be stated that numerical diffusion is smallest when:

- there is a significant sediment transport;
- the transport gradients are small, which is the case when suspended transport has a significant share in total sediment transport;
- the flow is reversing, like tidal flow;
- the initial channel geometry is not too sharp (gentle side slopes).

This again shows that in this thesis SUTRENCH has to be used at its limits of applicability to study the slope development of rather steep side slopes.

9.3 Relevant input parameters

Before discussing the results of simulations of SUTRENCH some input parameters will be explained. The choice of the input values of these parameters will be discussed.

Dimensions of computation space

The number of horizontal and vertical grid points ('N' and 'NV' in SUTRENCH) together with the horizontal grid size Δx and water depth h define the calculation space. The water depth at the surrounding seabed is kept constant at 10m, the number of vertical grid points is around 15 with decreasing Δy towards the seabed, automatically calculated by SUTRENCH. The horizontal grid size is 2 or 2,5 m and the number of grid points is 401 to 501, resulting in a horizontal calculation length of 800 to 1200 m. So, the horizontal grid size is explicitly inserted by the user, while the vertical grid size is implicitly calculated by SUTRENCH.

Dimensions of computation time

The computation time is defined rather cumbersome and consists of the product of the number of tidal cycles ($N_{\text{tidalcycles}}$) and the duration of one tidal cycle ($\sum(\Delta t_{TS})$), which is the sum (from 1 to N_{TS}) of all time steps (Δt_{TS}) during one tidal cycle:

$$\text{ComputationTime} = N_{\text{tidalcycles}} \sum_{TS=1}^{TS=N_{TS}} \Delta t_{TS} \quad \text{Eq. 9-10}$$

Numerical and physical parameters

- Reference level 'a' ('ZA' in SUTRENCH): the reference level at which the bed boundary is applied is usually assumed to equal half the bed-form height. In this thesis $a = 0,05\text{m}$;
- Acceleration of gravity 'g' = $9,81 \text{ m/s}^2$;
- Constant of Von Karman ' κ ' ('RK' in SUTRENCH) = 0,4;
- Kinematic viscosity of water ('RNU' in SUTRENCH) = $1 \cdot 10^{-6} \text{ m}^2/\text{s}$ for fresh water and $1,3 \cdot 10^{-6} \text{ m}^2/\text{s}$ for salt water;
- Angle of internal friction ('PHI' in SUTRENCH) is taken as 35° , representing medium packed coarse, medium and fine sand and considered to be an averaged value of all reference soils;
- Sediment density ('RHOS' in SUTRENCH) = 2650 kg/m^3 ;
- Fluid density ('RHOW' in SUTRENCH) = 1000 kg/m^3 for fresh water and 1030 kg/m^3 for fresh water;
- The numerical coefficient α_s ('ALFA' in SUTRENCH) is varied between 0,01 to 0,5, depending on the type of simulation. SUTRENCH allows values in a range of 0,001 to 1,0;
- The sediment upward boundary condition can be inserted as an upward sediment flux or as an equilibrium bed concentration, which results in a rapid adjustment of the suspended load vertical to the equilibrium concentration. This was one of the assumptions: incoming sediment transport is in equilibrium;

Wave data

The wave data can be defined variable in time (and thus constant over the entire computation area) or variable over the computational area (and thus constant in time). As was mentioned before, the variation of wave height over the expansion in depth due to shoaling and refraction effects yielded only small variations in morphological behaviour. Therefore a time-dependent wave-field will be defined: at first with constant wave height, later various schematizations of varying wave heights.

Bottom profile

The bottom profile can be read from an input file (*.pol) or be filled in manually. SUTRENCH interpolates between the inserted bottom profile coordinates.

Boundary conditions per time step

- Discharge Q [m^3/s] at upstream boundary, positive for flood discharge and negative for ebb discharge;
- Water depth h with respect to reference level at $x=0$. Because the water level is assumed horizontal the water level is known over the entire computation area;
- Particle fall velocity w_s [m/s] of suspended material, which can be rather different from the bed material, especially for wide gradations;
- Particle diameter d_{50} [m] which follows from the definition of the reference soils;
- Particle diameter d_{90} [m] which was defined at $1,5d_{50}$, see Paragraph 3.1.2;
- Porosity n , which varies from 0,35 to 0,45 according to the definition of the reference soils, but is kept constant throughout the simulations at a value of 0,40, representing the 'average reference soil';
- The duration of this particular time step ('DT' in SUTRENCH), which varies between 1 to 6 hours depending on the flow schematization;
- Correction factor of bed-load concentration ('CA' in SUTRENCH), which determines suspended sediment transport. If measured sediment verticals are available, this coefficient can be varied to fit calculated sediment transport at the upstream boundary. In this thesis no such data are available, so $CA=1$;
- Correction factor of bed-load transport ('SB' in SUTRENCH), which determines bed-load sediment transport. The same holds for this correction factor, so $SB=1$;
- Coefficient β ('BF' in SUTRENCH) was mentioned before and considered to equal 1;
- The angle between current and wave direction ('RPHI' in SUTRENCH) is set to 90° for all simulations, except the simulations with oblique flow.

Per time step also flow refraction effects can be taken into account by varying the width of the stream tube across the channel. In case of perpendicular flow this flow width remains constant and therefore the width equals 1.

The current- and wave-related bed roughness can also be varied across the channel. Sometimes this roughness is considered to be depth-dependent, primarily because this allows for nice mathematical formulations (water depth drops out of the friction coefficient). Other researches like Davies et al. [2002] mention "the self-regulating nature of the seabed roughness, whereby rippled beds formed at low (orbital) flow stages (small waves) tend to enhance sediment transport, while plane beds formed at high flow stages tend to inhibit the transport". Not only the flow stage determines the bed roughness, but De Boer (1998) also mentions the packing of the seabed: "the transport on a densely packed bed, which is relatively smooth, is larger than on a loose bed." This sediment density however is not constant in time.

Representing the bed roughness as a constant value is easily applicable, but not completely legitimate. On the other hand describing the bed roughness as a function of depth, flow stage, time, sediment diameter and angularity, is almost impossible, especially because fitting data are very scarce. Current- and wave-related bed roughness are therefore kept constant at 0,03m, see Paragraph 7.1.1.

10 Results of SUTRENCH-simulations

Although many parameters can be changed, many hydrodynamic conditions can be investigated, many channel and trench geometries should be considered and multiple schematizations of reality should be verified, some choices had to be made.

At first an extensive sensitivity analysis was executed to gain insight in the sensitivity of 18 important input parameters. Then the "navigation channel" is subjected to a unidirectional current with a constant perpendicular wave field. Initial side slopes, magnitude of the current and wave height will be changed in order to find out what the consequences for the upstream and downstream slope steepness will be. The same procedure is followed for schematized tidal flow, represented by a block function. Now, the distinction between upstream and downstream slopes has disappeared. In order to obtain a better resemblance to real hydrodynamic conditions a real tidal and wave climate of the North Sea has been used. The wave climate will be schematized in different ways to investigate the influence of this schematization and to come to a wave height that is representative for a full year.

All of the above simulations are based on currents perpendicular and waves parallel to the channel axis, because this situation is representative for most navigation channels perpendicular to the coastline with a longshore tidal current and a wave field that has refracted towards the coast. As was discussed in great detail in Chapter 6, current velocities and wave heights are affected by an expansion in water depth and this effect is largely dependent on the angle to the channel axis.

Van Rijn [1986a] investigated the situation with waves directed opposite to the current, because then the reduction of the wave height inside the channel is largest (up to 20%). Although he found a further reduced sediment transport capacity due to the smaller sediment pick-up and wave-related mixing, the overall effect on bed level changes remained limited to only a few percent. This adaptation of the wave height is therefore considered to be of minor importance. Current refraction will be of significance, but will not be investigated in this thesis due to time restrictions. However, it is interesting to compare situations without current refraction, with instantaneous refraction solely due to continuity and with slowly adapting current refraction with each other. One should note that the effective length of a side slope increases if a current enters the channel under an angle: a current that has an incoming flow angle of 45° over a side slope of 1:5 'feels' this slope as it was 1:7. This effect is favourable with respect to resemblance to a logarithmic velocity profile.

10.1 Sensitivity Analysis

10.1.1 Previous results

To gain some insight in the sensitivity of the calculated results to small variations in the input parameters a sensitivity analysis was done. To be able to compare the results with previous researches, the conclusions of two of these researches are presented.

I. In his thesis Jenniskens [2001] studied large numbers of data on sediment transport and deduced a strongly simplified approach formula of the original total load formula of Van Rijn (1993). Although it is advised not to apply this formula in transport calculations, it tells a lot about the sensitivity to variation of the input parameters. An important drawback is the fact that only five input parameters are included, but these parameters are all within the range of this thesis. This formula reads:

$$S_{total;VanRijn_approach} = 2 \cdot 10^{-6} H_s^{1,7} T_p^{0,2} h^{-1,2} u^{2,7} d_{50}^{-1,75} \quad \text{Eq. 10-1}$$

with $0,5 \leq H_s \leq 3$ [m], $5 \leq T_p \leq 9$ [s], $3 \leq h \leq 15$ [m], $0,2 \leq u \leq 1,2$ [m/s] and $125 \leq d_{50} \leq 500$ [μm] on condition that $H_s / h \leq 0,6$.

In this thesis the current velocity and water depth are related via the discharge q , which remains constant when passing over a trench. Therefore the above formula will be modified to:

$$S_{total;VanRijn_approach} = 2 \cdot 10^{-6} H_s^{1,7} T_p^{0,2} h^{-3,9} q^{2,7} d_{50}^{-1,75} \quad \text{Eq. 10-2}$$

in which the water depth h has become the most influential input parameter, because this parameter affects the wave and current stirring action on the bed and the transport capacity of the current. On the trench bottom ($h=h_1$) not only the waves are 'felt' to a lesser degree, the current velocity has also dropped, which reduces bottom shear stresses and transport.

Furthermore it can be seen that the wave height H_s is far more influential than the wave period T_p . The bottom shear stress induced by waves is quadratic proportional to the wave height, so the increased 'stirring effect' is easily accounted for, but when it comes to transport the situation gets far more complicated and SUTRENCH does not well represent nature anymore. In presence of waves the velocity profile becomes distorted and if waves are getting bigger (and so are the horizontal orbital velocities), this profile cannot be presented like an ideal logarithmic profile anymore. Sediment transport is calculated as the depth integrated product of concentration and velocity. As soon as one of both profiles does not resemble reality, the calculated total transport becomes inaccurate.

Another important aspect is the before mentioned 'phase lag', i.e. the situation that small particles that are stirred up by waves in positive direction can be transported in negative direction due to the small particle fall velocities. This effect reduces with increasing wave period; a small positive relation of the sediment transport to the wave period is often noticed in experiments (Dohmen-Janssen (1999) as described by Camenen and Larroudé [2002]). The appearing 'coincidentally' power of 0,2 in Eq. 10-2 however, cannot be caused by this effect, because Van Rijn does not take sediment transport by waves into account. The small positive influence has to be explained in a somewhat different way. On the one hand, larger wave periods mean longer waves and therefore larger horizontal excursions and velocities; on the other hand, the wave boundary layer has more time to develop, thereby reducing the friction factor f_w . The overall net effect is that bed shear stresses first increase for larger wave periods, but eventually become smaller if the wave period further increases. This effect is also depending on the water depth, as described in Paragraph 7.2.

Jenniskens found a significant negative relation between sediment size and sediment transport (power of -1,75), which seems logical and is unconditionally true for unidirectional flow. However, because of the above mentioned 'phase-lag phenomenon', sediment transport can increase if sediment particles become larger, because these particles are subjected to the phase-lag to a lesser degree, because of their larger particle fall velocities. That's why sediment transport under waves may increase with increasing particle diameters. Again, Van Rijn does not account for this effect and according to his formula sediment transport always decreases with increasing particle diameters.

II. When Klein [2003] verified SUTRENCH on some laboratory and field experiments, he also executed a sensitivity analysis of ten different input parameters on both the (suspended) sediment transport and the sediment transport gradient. The most important conclusions were:

With respect to sediment transport:

- the results are extremely sensitive to variation of wave height H_s and wave-related bed roughness height ' $k_{r,w}$ ';

- the discharge ' Q ', the water level ' h ', the particle fall velocity ' w_s ' and the reference level ' a ' have a significant influence on the magnitude of sediment transport;
- the current-related bed roughness height ' $k_{r,c}$ ', particle sizes ' d_{50} ' and ' d_{90} ' and wave period ' T_p ' have little influence.

These conclusions are to a large extent compatible with the approach formula of Jenniskens. The conclusions of Klein on the influence of the water depth and the discharge, which are the most influential parameters according to Jenniskens, are justified if one adopts the idea that water depth and discharge can be measured quite accurately and are more or less regularly varying in time (with the tide), while the wave height and bed roughness are strongly varying in time and/or difficult to determine, especially bed forms during storms.

With respect to sediment transport gradient (erosion / sedimentation):

- the particle fall velocity ' w_s ' has the largest influence on the transport gradient;
- the particle size ' d_{50} ' and ' d_{90} ', the reference level ' a ' and the wave period ' T_p ' hardly have any influence;
- the other parameters have significant influence.

So it appears that if an expansion in water depth is present and the sediment transport capacity changes, the particle fall velocity becomes the governing parameter.

10.1.2 Input parameters

Are the above conclusions also found in the current sensitivity analysis? Besides, eight other input parameters are studied. Three parameters are related to the sediment properties, viz the sediment density ' ρ_s ', the sediment porosity ' n ' and the angle of internal friction ' ϕ '. These parameters are related to the slopes that will develop and are therefore included in this analysis. Two parameters are related to the current-wave-characteristics, viz kinematic viscosity ' ν ' and the angle between current and waves ' α_{c-w} '. Three parameters are numerical coefficients normally used to calibrate the results: a numerical smoothing coefficient ' α_s ', a correction coefficient 'CA' for the bed load concentration ' c_a ' (bed boundary condition for suspended transport) and a correction coefficient 'SB' for the bed load transport ' S_b '; the latter two equal 1 in theoretical equations, but can be useful when adjusting the calculated concentration profile to a measured profile. Because in this thesis the calculations cannot be calibrated with values from experiments, the sensitivity of these parameters is examined.

The variation of the different parameters is based on expected difficulties in determination of the exact value of a certain parameter in nature. Therefore the rate of variation is not the same for all input parameters. The three numerical parameters are varied between values that are common in previous calculations with this model. For instance, Klein found values for 'CA' and 'SB' between 0,5 and 1,15 for a roughness of 0,03. This implies that the default value of 1 might be an overestimation.

Of course all results have been compared to a reference case which is a calculation with all mean values as input parameters, see Table 10-1.

The bed geometry that is considered in this sensitivity analysis is the reference case 'navigation channel' with a width of 200 m, so the upstream and downstream slope will not be influenced by each other, because of total adaptation of flow and sediment transport to local conditions. Because of this large width, both slopes can develop autonomously. The water depth inside the channel is 20 m and the surrounding water depth is 10 m.

Table 10-1: input parameters of sensitivity analysis; variation around mean values

input par.	description	symbol	unit	var +/-	mean	min	max
ZA	reference level of bed-form height	a	m	40%	0,05	0,03	0,07
RNU	kinematic viscosity	ν	m ² /s	18%	1,10E-06	9,00E-07	1,30E-06
PHI	angle of repose of bed material	ϕ	degr	15%	35	30	40
RHOS	sediment density	ρ_s	kg/m ³	2%	2650	2600	2700
ALFA	pseudo-viscosity factor; numerical smoothing coefficient	α_s	-	90%	0,5	0,1	1
Hs	significant wave height	H_s	m	20%	1,5	1,2	1,8
TP	peak period	T_p	s	15%	7	6	8
Q	discharge; flood is positive; ebb is negative	Q	m ³ /s	10%	10	9	11
H	water surface level w.r.t.datum/reference level	h	m	5%	10	9,5	10,5
WS	particle fall velocity suspended material: 0,01-0,05	w_s	m/s	25%	0,0252	0,0189	0,0315
D50	50 % particle diameter of weight of bed material	d_{50}	μ m	25%	200	150	250
D90	90 % particle diameter of weight of bed material	d_{90}	μ m	25%	300	225	375
P	porosity of bed material	n	-	13%	0,4	0,35	0,45
CA	correction coefficient of bed load concentration	CA	-	30%	1	0,7	1,3
SB	correction coefficient of bed load transport	SB	-	30%	1	0,7	1,3
RPHI	angle between wave and current direction	α_{c-w}	°	20%	90	72	108
RC	current related bed roughness	$k_{r,c}$	m	67%	0,03	0,01	0,05
RW	wave related bed roughness	$k_{r,w}$	m	67%	0,03	0,01	0,05

10.1.3 Results of sensitivity analysis

Output parameters

The output parameters on which the separate runs will be compared are the bed-load, suspended and total sediment transport (S_b , S_s and S_t) on a plane bed (at 50 m from the upstream boundary) as well as the minimum gradient of bed-load, suspended and total transport on the upstream ($(dS_b/dx)_{min}$, $(dS_s/dx)_{min}$, $(dS_t/dx)_{min}$) slope as a measure for the maximum sedimentation that occurs somewhere along the slope. The transport gradient is a good measure for slope development, because not the sediment transport itself determines whether the seabed erodes or accretes, but the change in sediment transport.

A qualitative (visual) analysis of the morphological development after 1 month will be done for the most relevant input parameters to demonstrate the long-term effects of small variations in the input parameters.

The above mentioned period of 1 month (350 time steps of 2 hour = 29,17days) is considered to be representative, because slopes have adapted, but 'silting up problems' have not occurred yet. Of course, this relatively small period is related to the rather strong constant unidirectional current with a constant velocity of 1m/s and a relatively large (constant) significant wave height of 1,5m.

Sensitivity of sediment transport and gradient

In Table 10-2 the results of 37 sensitivity runs with SUTRENCH are presented. In design practice an engineer is likely to deal with variations of this magnitude. Because one parameter can better be estimated or measured than another, the resulting variation of each input parameter may differ significantly.

If grain mixtures are very well graded, a problem arises when applying SUTRENCH and errors may rise excessively: the suspended sediment is likely to differ from the bed sediment. Even under changing hydrodynamic conditions the grain distribution of the suspended sediment may constantly adapt itself.

Variations in bed-load, suspended and total sediment transport are presented in percentages. A negative value means that sediment transport is reduced. For instance when the wave height is increased by 20% (H_s from 1,5 to 1,8 m), the total sediment transport is increased by 42%, which can be mainly contributed to the suspended sediment transport, which is increased by 45%. Larger waves are extremely effective in stimulating bed particles to leave the bed-load layer and getting mixed throughout the sediment vertical. These values for the variations in sediment transport are all determined at the flat surrounding seabed (equilibrium situation), unlike the sediment transport gradients. These gradients are the largest negative gradients, occurring somewhere along the upstream slope. This is in fact a measure for the largest sedimentation to be expected.

Table 10-2: Output parameters of sensitivity analysis [%]

symbol	variation	sediment transport						minimum gradient (=max sedimentation)					
		S_b		S_s		S_t		$(dS_b/dx)_{min}$		$(dS_s/dx)_{min}$		$(dS_t/dx)_{min}$	
		min	max	min	max	min	max	min	max	min	max	min	max
a	40	0,0	0,0	40,2	-16,3	36,1	-14,6	0,0	0,0	51,0	-23,0	40,4	-18,6
v	18	5,4	-12,4	7,9	-16,1	7,7	-15,7	5,0	-11,6	4,7	-8,9	4,7	-9,9
ϕ	15	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,8	-0,4	0,6	-0,6
ρ_s	2	2,4	-2,3	2,2	-2,1	2,2	-2,1	2,0	-2,0	2,7	-1,9	2,5	-2,5
α_s	90	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
H_s	20	-14,6	17,8	-30,9	44,9	-29,3	42,2	-11,0	13,2	-14,4	16,3	-13,7	15,5
T_p	15	-13,7	9,7	-8,6	4,4	-9,1	5,0	-8,5	5,3	-8,2	5,1	-8,1	5,0
Q	10	-27,2	35,3	-41,0	64,1	-39,7	61,2	-27,4	35,3	-29,2	37,0	-28,9	36,6
h	5	23,2	-18,2	40,3	-27,9	38,6	-26,9	26,0	-20,0	27,2	-20,6	27,0	-20,5
w_s	25	0,0	0,0	116,1	-45,4	104,5	-40,8	0,0	0,0	-4,3	-0,4	-3,7	-0,6
d_{50}	25	-12,0	2,4	-16,6	10,5	-16,1	9,7	-12,4	3,1	-7,0	1,2	-8,1	1,6
d_{90}	25	-14,5	13,2	-6,2	5,1	-7,0	5,9	-13,5	12,2	-5,8	5,1	-7,8	6,2
n	13	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
CA	30	0,0	0,0	-30,0	30,0	-27,0	27,0	0,0	0,0	-30,0	30,4	-23,9	23,9
SB	30	-30,0	30,0	0,0	0,0	-3,0	3,0	-30,1	30,1	0,0	0,0	-6,2	5,9
α_{C-W}	20	22,5	14,8	0,4	0,1	2,6	1,6	19,5	12,4	0,8	0,4	4,3	2,5
$k_{r,C}$	67	26,5	-13,5	-5,6	1,5	-2,4	0,0	24,2	-12,7	17,1	-10,1	18,0	-10,6
$k_{r,W}$	67	-10,8	5,4	-22,7	18,1	-21,5	16,8	-10,9	5,3	-20,2	15,6	-18,3	13,4

In Table 10-2 variations in sediment transport or transport gradient exceeding 20% are plotted in bold. Some conclusions can be drawn:

- Reduction of the reference height 'a' introduces serious errors. This parameter should certainly not be chosen too small;
- The angle of internal friction ' ϕ ' of course does not affect sediment transport on a flat bed in equilibrium. The gradient, which is determined at the upstream slope, is slightly affected, although the gradient of the bed-load transport remains the same. This does not imply that the bed-load transport at the slope remains the same; this will increase for smaller friction angles;
- The solid sediment density ' ρ_s ' has little effect (about 2%);
- The numerical smoothing coefficient ' α_s ' does not affect initial sediment transport rates. However it does have great influence on morphological changes and will certainly become more important in the long run. Almost the same explanation holds for the bed porosity 'n', which affects bed level changes and therefore eventually sediment transport;
- The main wave parameter is the significant wave height: both bed-load (wave stirring) and suspended (wave mixing) transport will increase for larger wave heights. The wave period only affects bed-load transport due to larger accelerations near the bed;
- The flow velocity (Q/h) has a strong positive correlation with sediment transport and transport gradient, because sediment pick-up, convection and mixing are involved;
- The particle fall velocity usually is related to the particle diameter, but in SUTRENCH both properties have to be entered separately. An increase of the particle fall velocity reduces suspended (and total) sediment transport and has no effect on bed-load transport, while an increase of the grain size increases both bed-load and suspended transport due to the larger particle volume. In reality both properties are related and the overall effect will be a negative correlation;
- The correction factors for bed-load (SB) and suspended (CA) transport need no further explanation. The variation in input (30%) results in an equal variation in output, which is an indication that the SUTRENCH-results are sound;
- The angle between waves and currents primarily affects bed-load transport. Furthermore it appears that a wave field that is directed perpendicular to the current yields least sediment transport ('min' and 'max' value both have positive sign);
- Sediment transport is more sensitive to the wave-related bed roughness than to the current-related roughness.

Approximation and/or sensitivity formula

The influence of the most important input parameters can also be expressed as a power-law function of the total sediment transport, comparable to the approach of Jenniskens [2001].

$$S_t = C x_1^{p_1} x_2^{p_2} x_3^{p_3} x_4^{p_4} x_5^{p_5} \quad \text{Eq. 10-3}$$

When considering one input parameter at a time, two approximations can be calculated: one power-law function through the reference case and the minimum value of the input parameter and one power-law function through the reference case and the maximum value of the input parameter:

$$\begin{aligned} S_t &= C x^p && \text{through } (x_{\min}, S_{\min}) \text{ and } (x_{\text{mean}}, S_{\text{mean}}) && \text{yields } (p_{\min}, C_{\min}) \\ S_t &= C x^p && \text{through } (x_{\max}, S_{\max}) \text{ and } (x_{\text{mean}}, S_{\text{mean}}) && \text{yields } (p_{\max}, C_{\max}) \end{aligned}$$

in which:

$x_{\min}, x_{\text{mean}}, x_{\max}$	= minimum, reference and maximum input variable;
$S_{\min}, S_{\text{mean}}, S_{\max}$	= minimum, reference and maximum output of sediment transport;
$p_{\min}, p_{\max}$	= power in approximation formula;
$C_{\min}, C_{\max}$	= constant in approximation formula;

The following relations can be obtained:

$$p_{\min} = \frac{\log\left(\frac{S_{\min}}{S_{\text{mean}}}\right)}{\log\left(\frac{100 - \text{VAR}(X)}{100}\right)}, \quad p_{\max} = \frac{\log\left(\frac{S_{\max}}{S_{\text{mean}}}\right)}{\log\left(\frac{100 + \text{VAR}(X)}{100}\right)}, \quad C_{\min} = \frac{S_{\text{mean}}}{x_{\text{mean}}^{p_{\min}}}, \quad C_{\max} = \frac{S_{\text{mean}}}{x_{\text{mean}}^{p_{\max}}}$$

in which

$\text{VAR}(X)$ = the variation of the input variable in %.

If both approximations are averaged, the following 'powers' and 'constants' are obtained:

$$p = \frac{p_{\min} + p_{\max}}{2} \quad \text{and} \quad C = \frac{C_{\min} + C_{\max}}{2}$$

The values for all input parameters are presented in Table 10-3. A value of '0' means no influence, a value of '1' means a linear relation.

The most important sediment and hydrodynamic parameters will be used in the approximation formula:

$$S_{t;\text{sensitivity_analysis}} = 3,4 \cdot 10^{-2} H_s^{1,7} T_p^{0,5} h^{-6,4} q^{4,9} w_s^{-2,4} d_{50}^{0,5} \quad \text{Eq. 10-4}$$

which is centred around $H_s = 1,5$ m; $T_p = 7$ s; $h = 10$ m; $q = 10$ m³/ms; $w_s = 0,0252$ m/s; $d_{50} = 200$ μm and yields for this reference case $S_t = 0,506$ kg/ms. Please note that the constant of $3,4 \cdot 10^{-2}$ has a rather strange unit of about kg s m^{-4,2} and has no physical base. When comparing this formula with the formula found by Jenniskens, it appears that all signs are similar and the power of the wave height is exactly the same. The sensitivity to water depth and discharge is somewhat larger for this case. The influence of the sediment size is not so transparent, because of the presence of the particle fall velocity. However, if one assumes a linear relation between w_s and d_{50} (see Figure 3-1) the power of d_{50} becomes -1,9, whereas Jenniskens found -1,75.

Because all 'powers' resemble 'Jenniskens' powers' reasonably, the application of his formula should yield values for total sediment transport of at least the same order:
 $S_{t,Jenniskens} = 1,10 \text{ kg/ms}$, which differs about a factor 2.

Although Jenniskens has calibrated his formula with over 2500 results and expression 10-4 is only calculated around the reference case, it still yields a rather good sediment transport formula for this particular case and demonstrates the sensitivity to the various input parameters well. It should be evident that it makes no sense applying this strongly schematized formula in other conditions.

Table 10-3: Powers of sensitivity of total sediment transport to 15 input variables [-]

symbol	var [%]	power p [-]		
		S_b	S_s	S_t
a	40	0,0	-0,6	-0,5
v	18	-0,5	-0,7	-0,7
ϕ	15	0,0	0,0	0,0
ρ_s	2	-1,2	-1,1	-1,1
α_s	90	0,0	0,0	0,0
H_s	20	0,8	1,8	1,7
T_p	15	0,8	0,4	0,5
Q	10	3,1	5,1	4,9
h	5	-4,1	-6,7	-6,4
w_s	25	0,0	-2,7	-2,4
d_{50}	25	0,3	0,5	0,5
d_{90}	25	0,5	0,2	0,3
n	13	0,0	0,0	0,0
CA	30	0,0	1,0	0,9
SB	30	1,0	0,0	0,1
α_{c-w}	20	-0,1	0,0	0,0
$k_{r,c}$	67	-0,2	0,0	0,0
$k_{r,w}$	67	0,1	0,3	0,3

Morphological development

Until now the sensitivity of initial sediment transport and transport gradients ($t=0$) to some input parameters was discussed. This approach says little about actual morphological development of the channel or trench in time. Therefore some relevant parameters are studied in more detail, complete with a graph to explain the differences in morphological development.

Particle diameters d_{50} and d_{90}

Because of the occurrence of mixtures of different grain sizes and the difference in sediment particles at different locations (upstream or local) and at different depths (at the initial bed surface or at the bottom of the trench), the spread was assumed quite large (25%). It appears that the particle diameter itself has very little influence, see Figure 10-1. As was explained above, a larger sediment diameter causes a faster backfilling of the trench. This is solely due to the fact that the sediment transport is somewhat larger because of the larger particle volume, while the particle fall velocity is kept constant.

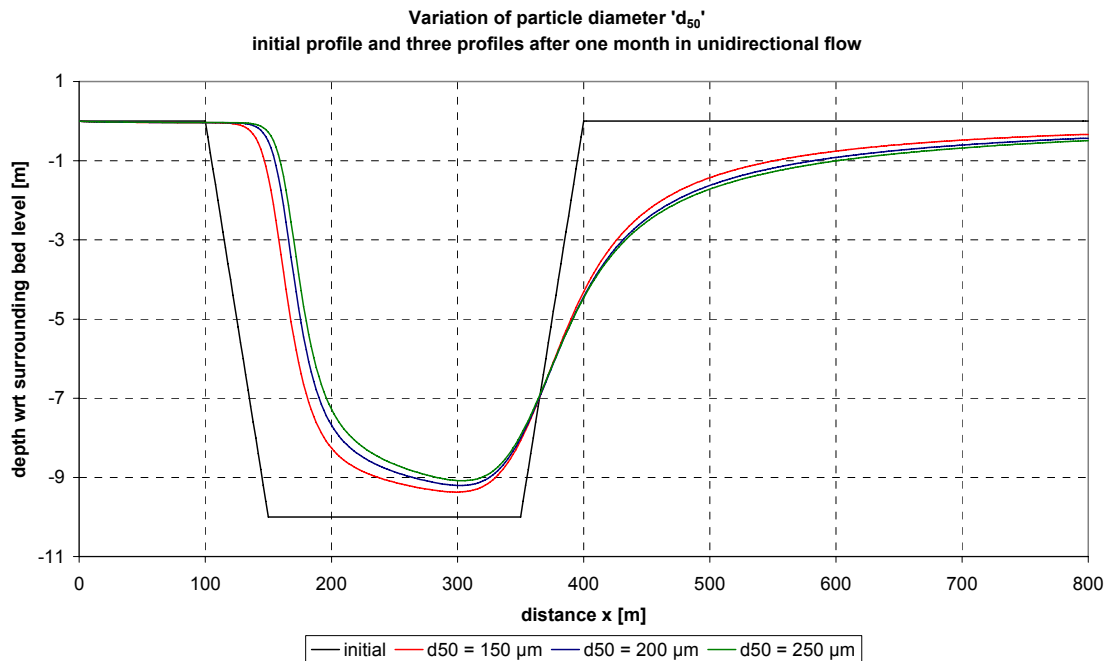


Figure 10-1: Morphological development caused by variation of d_{50} with 25%

Particle fall velocity w_s

Related to the grain size is the particle fall velocity. A variation of the grain sizes with 25%, also leads to a variation in particle fall velocities of about 25%.

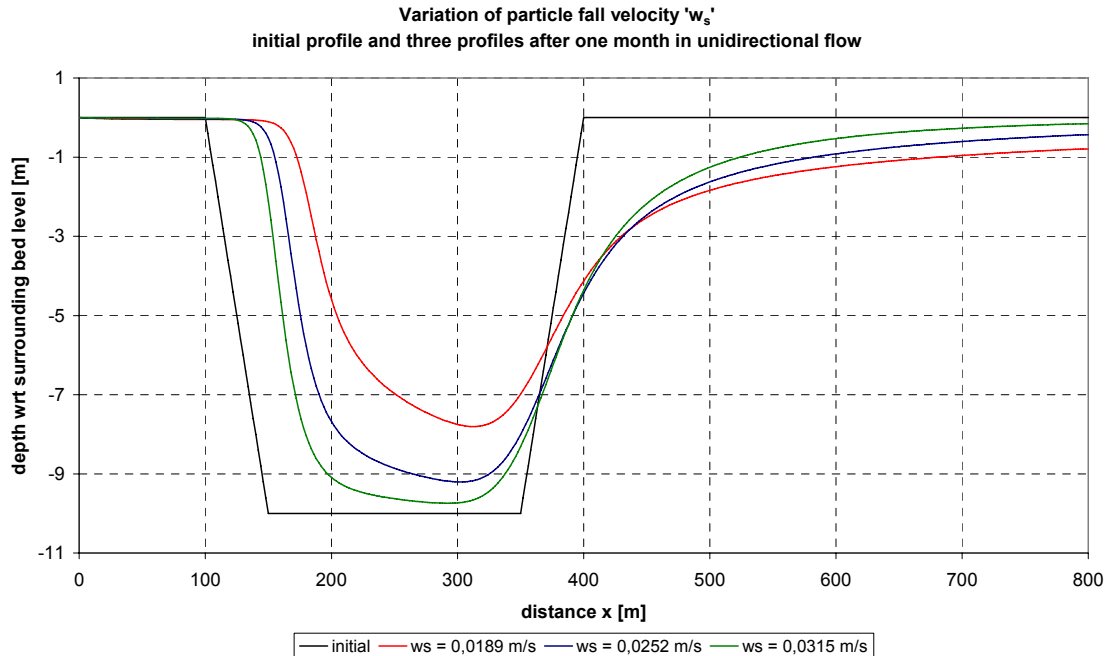


Figure 10-2: Morphological development caused by variation of w_s with 25%

In Figure 10-2 one can see that this parameter is very influential. The smaller the particle fall velocity, the more sediment particles will stay in the water column. When this larger amount of suspended sediment transport travels over the trench, most of the sediment will have enough time to settle and a rapid silting up of the channel occurs. Also can be seen that the deposited particles with a large particle fall velocity stay close to the upstream slope, leaving the initial bottom of the trench almost unaffected. Only shifting of the upstream slope occurs. At the downstream slope, the accelerating flow causes less erosion.

The particles with a small particle fall velocity are spread over a wide area and the navigable depth is reduced over the entire channel width. The net sedimentation (within the initial channel boundaries) differs between 483 and 1138 m³/m.

A sound determination of the particle fall velocity therefore seems to be inevitable, but will cause some problems in engineering practice because of the mixtures of sediment and differences between local sediment particles and sediment particles carried from further on.

Porosity of bed material n

Often little is known about the porosity of the initial bed, but even more questions exist about the porosity of the sedimented material. This is extremely dependent on the conditions, especially on the wave conditions, under which sedimentation takes place. When varying the porosity ' n ' between 0,35 and 0,45, the only observed changes in the bed geometry after one month, is the vertical location of the bed. On these locations where net sedimentation takes place (upstream slope and trench bottom), the bottom elevation is largest for $n = 0,45$; on these locations where net erosion takes place (the upper half of the downstream slope) the drop in bottom level is largest for $n = 0,45$. At the upstream slope at a trench depth of about 7 m, the three profiles coincide, because no net erosion or sedimentation occurs and the porosity ' n ' is of no interest.

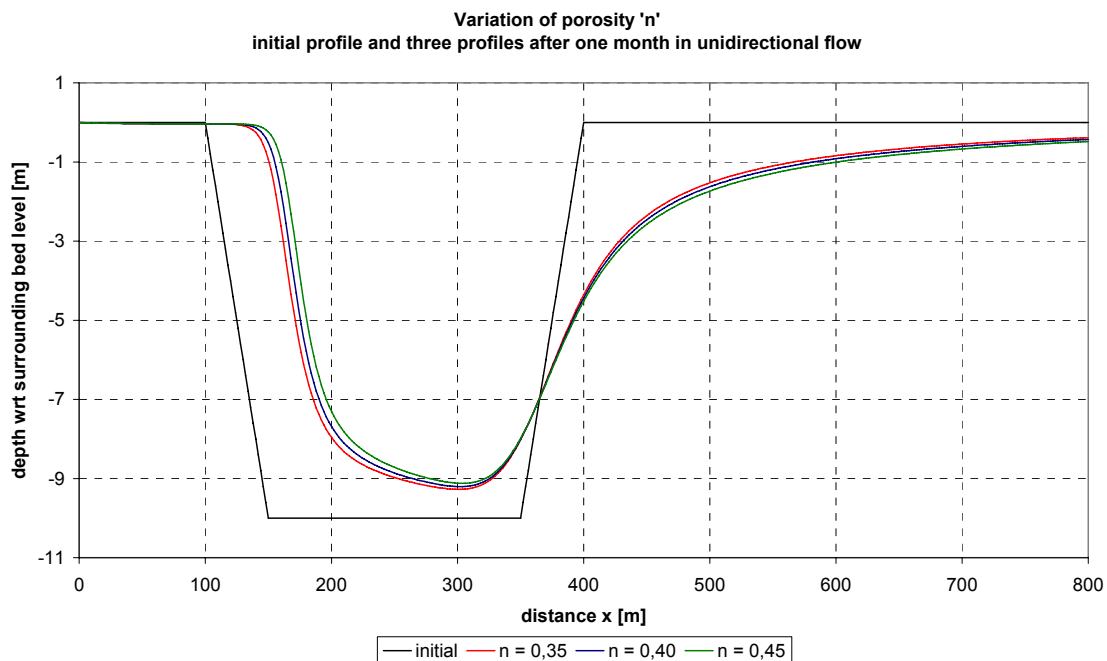


Figure 10-3: Morphological development caused by variation of n with 12,5%

In Paragraph 9.3 it was mentioned that the porosity also influences sediment transport. More densely packed sediment means a smoother seabed hence more sediment transport. Sediment transport formulae, however, do not consider this phenomenon implicitly. It can only be taken into account by variation of the bed roughness. In practice both porosity and bed roughness are very hard to determine. In this thesis the porosity is only a measure for the rate of bed level changes; the bed roughness is kept constant.

In the future a relation between bed roughness and porosity may be developed. One could for instance distinguish between the 'old, more densely packed bed' (with water depths larger than the initial profile) and the 'new bed' (with depths smaller than the initial profile).

Significant wave height H_s

Variation of the significant wave height affects sediment transport. Larger waves stir up more sediment, mix it better throughout the vertical and still have impact on particles located deeper in the channel. As a result an increase of the wave height has a somewhat larger effect than a decrease, see Figure 10-4.

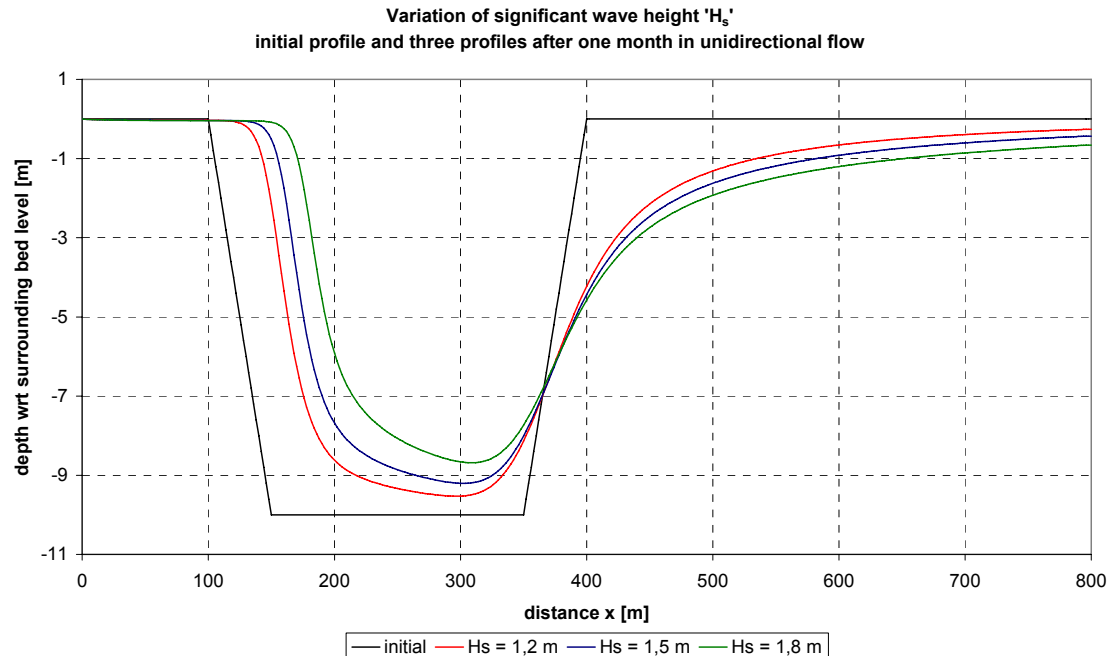


Figure 10-4: Morphological development caused by variation of H_s with 20%

Although it seems that smaller waves cause steeper upstream and downstream slopes, this conclusion cannot be drawn based on this graph. The morphological development of the case with larger waves is just ahead of time compared to the cases with smaller waves. During this process steeper slopes may have occurred. This phenomenon of slope development in time will be studied in the next paragraphs.

Numerical smoothing coefficient α_s

This parameter causes numerical smoothing at sharp discontinuities. Because the initial profile shows some sharp upper corners, without numerical smoothing 'oversteepening' of the upstream slope may occur, resulting in unrealistic steep slopes, even steeper than the natural slope. In Figure 10-5 one can see, that the influence of variation of this parameter mainly affects the steepness of the slopes, while the trench volume or depth barely changes. Also the total erosion and sedimentation on the upstream and downstream slope is almost the same, although the amount of local erosion/sedimentation shows some variation. Considering the three profiles, an α_s of 0,5 results in realistic geometries: not too sharp, not too slanted.

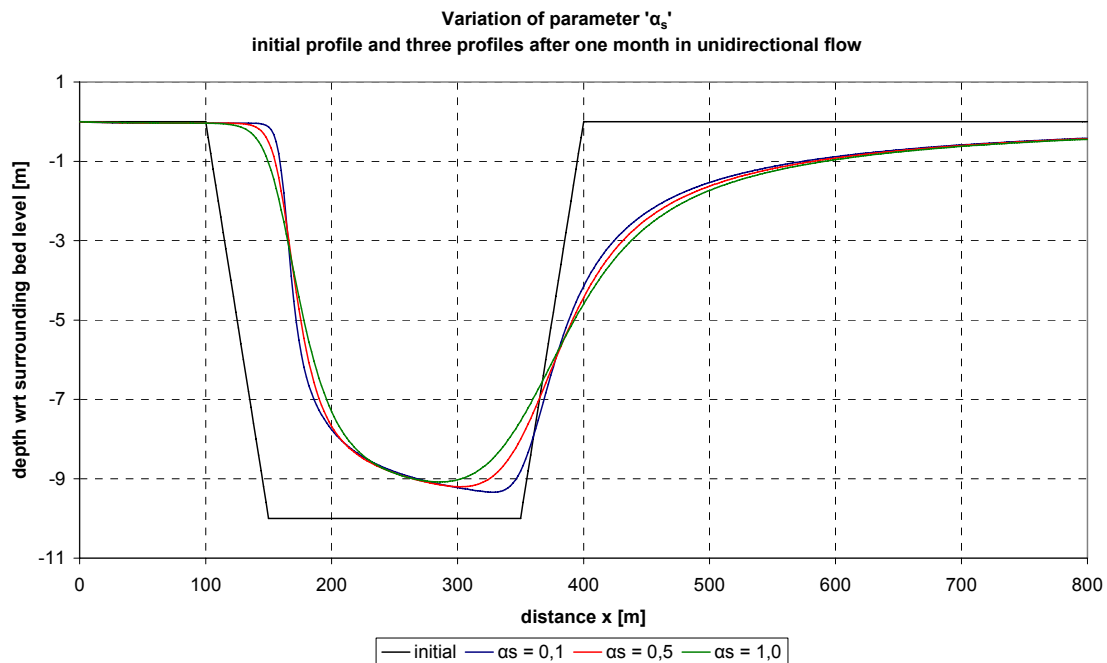


Figure 10-5: Morphological development caused by variation of α_s with 90%

Because variation of this parameter does not affect sediment transport and the time-scale in which the channel silts up, slopes can be compared at any moment. Many other parameters, as was explained at the section about the significant wave height, influence the morphological time-scale, which makes it impossible to compare slope development at a certain time. Moreover, the smoothing coefficient will turn out to be of great importance when considering slope development and will therefore be studied in more detail. Another graph-format is chosen to present the slope of the seabed, see Figure 10-6.

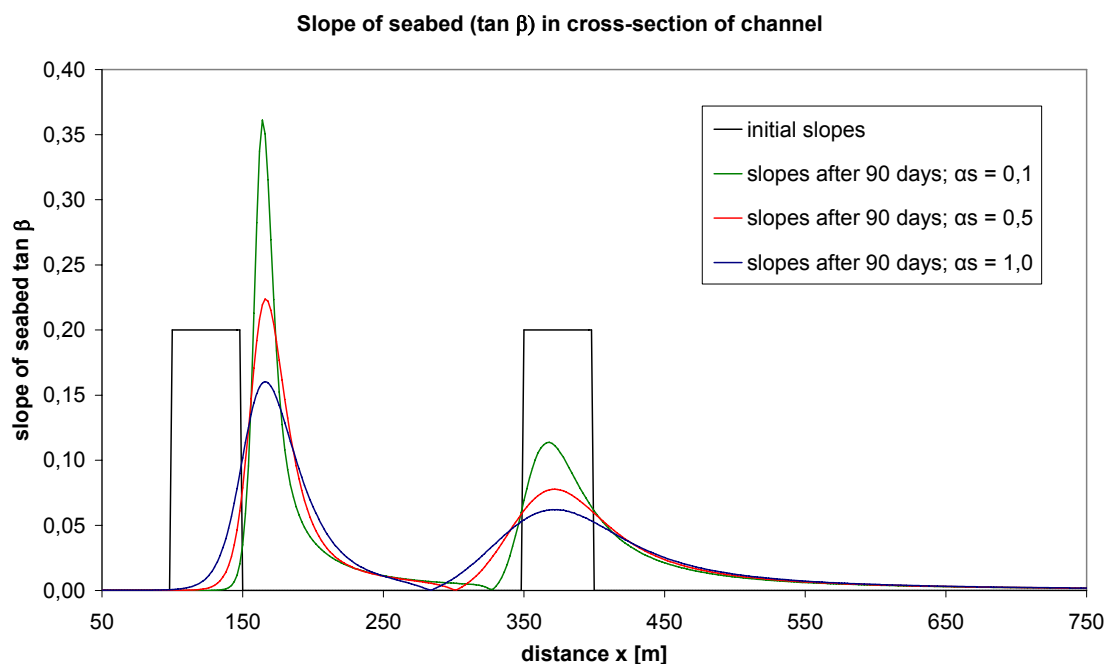


Figure 10-6: Slope of cross-channel seabed for various numerical smoothing coefficients

The initial side slopes are the 'bumps' with height 0,2, because the reference case "navigation channel" consisted of a flat surrounding seabed, flat channel bottom and two straight 1:5 side slopes. Because for geotechnical stability the direction of the slope does not matter, all slopes are represented by their absolute values. It can clearly be observed that a smaller smoothing coefficient results in sharp peaks; the maximum upstream slope is even steeper than 1:3 for $\alpha_s = 0,1$. At the middle of the navigation channel the smoothing effect has faded out and all profiles show equal bed slopes. So, although the numerical smoothing coefficient does not change the backfilling predictions (the main purpose of SUTRENCH), slope development is extremely sensitive to the value of this parameter. The only way to deal with this parameter properly is to treat it as a sort of dustbin-parameter which accounts for all morphological processes that SUTRENCH cannot model. This is perhaps the most important shortcoming of SUTRENCH when one is particularly interested in morphological development of the side slopes.

Conclusions

The sensitivity analysis revealed that all relevant input parameters show logical behaviour, which is in line with previous studies like Jenniskens [2001] and Klein [2003]. Regarding sediment transport, the most influential parameters are the discharge, water depth, significant wave height, particle fall velocity and the wave-related bed roughness. Morphological (particular slope) development is also heavily affected by numerical smoothening processes.

10.2 Simulations with unidirectional flow

10.2.1 Introduction

To study the slope development in unidirectional flow, a lot of calculations for all reference soils have been done with varying discharges (current velocities), wave heights and initial slopes. The results of these calculations should answer the following questions:

- What is the influence of the current velocity on slope development and backfilling of the trench for different soil types?
- What is the influence of the (significant) wave height on slope development and backfilling of the trench for different soil types?
- Does the initial slope affect slope development and backfilling ratio?
- Is there something like a 'dynamic equilibrium slope'?
- When do the steepest slopes occur? Is there a greater risk on instability in time?
- What is, starting from the idea that initial slopes are completely straight, the best design strategy for all reference soils.
- How reliable are the presented time scales of backfilling and slope development?

All executed calculations are presented in Table 10-4, complete with simulation names. 'F-types' are simulations with different initial side slopes; 'E-types' are simulations with different current discharges; 'H-types' are simulations with different wave heights and periods. The different simulations will be judged on the following criteria based on the geometries of both side slopes and the total channel. In this situation of unidirectional flow, it is useful to distinguish between upstream and downstream slopes:

- Upstream slope after 90 days;
- Downstream slope after 90 days;
- Maximum depth reduction [%] defined as the maximum depth after 90 days divided by the initial channel depth;

- Width reduction [%] defined as the width after 90 days at which 90% of the initial design channel depth is present divided by the initial channel width at which 90% of the initial design channel depth is present. This depth of 90% of the initial depth is chosen such that very small siltation and gravity effects on the grain particles on a slope will not affect the results too much. In reality there will always be some initial overdepth, so 90% seems reasonable.
- Remaining channel volume [%] according to Figure 10-7 defined as the channel volume after 90 days within the initial channel, which can be significantly smaller than the total channel volume after 90 days due to channel migration. The depth above the downstream slope increases because of slope flattening, but this extra depth will not be useful: a large area with a relatively small depth increase.

These criteria are formulated from a practical point of view, as was considered in Paragraph 8.5.

The above mentioned criteria together will give a good idea on problems to be expected with slope stability, dredging frequency and effective measurements to improve the channel geometry.

However, one should note that, because of the above mentioned logarithmic velocity profile, the model can yield slightly too steep upstream and downstream slopes if the initial slopes are steeper than 1:5, as was found in validation of SUTRENCH with laboratory experiments by Havinga (1992) [Walstra et al., 2000]. The acceleration on the downstream slopes causes higher near bed velocities than is predicted by the logarithmic velocity profile. The sediment picking up capacities will be somewhat larger, resulting in flatter slopes. Nevertheless steeper slopes are modelled in this thesis to be able to compare the magnitude of these differences.

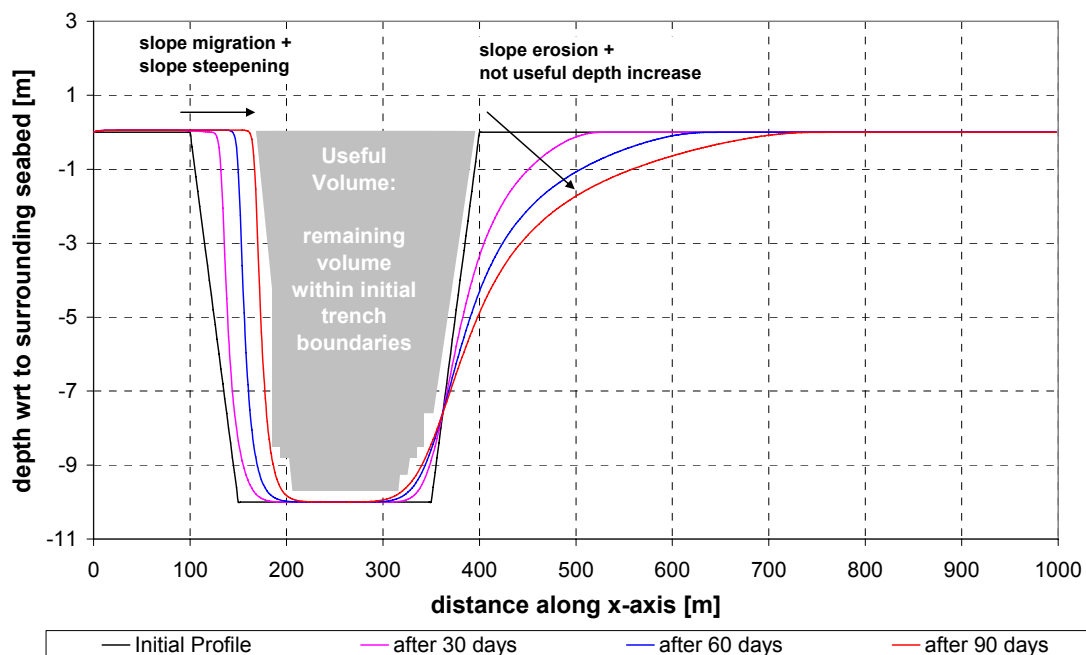


Figure 10-7: Schematization of remaining volume between initial channel boundaries (output simulation 'E04')

Table 10-4: Variation of different parameters; 'grey' cells illustrate the changes

	Name	Initial Slope	Current Discharge	Wave Height	Wave Period
		-	m ³ /s/m	m	s
Medium packed Coarse Sand	F06	1:2	5	1,5	6
	F10	1:3	5	1,5	6
	F14	1:4	5	1,5	6
	F02	1:5	5	1,5	6
	E03	1:5	7,5	1,5	6
	E04	1:5	10	1,5	6
	H03	1:5	5	0,5	5
	H04	1:5	5	2,5	7
	F18	1:6	5	1,5	6
	F22	1:7	5	1,5	6
	F26	1:8	5	1,5	6
Medium packed Medium Sand	F07	1:2	5	1,5	6
	F11	1:3	5	1,5	6
	F15	1:4	5	1,5	6
	F03	1:5	5	1,5	6
	E05	1:5	7,5	1,5	6
	E06	1:5	10	1,5	6
	H05	1:5	5	0,5	5
	H06	1:5	5	2,5	7
	F19	1:6	5	1,5	6
	F23	1:7	5	1,5	6
	F27	1:8	5	1,5	6
Medium packed Fine Sand	F05	1:2	5	1,5	6
	F09	1:3	5	1,5	6
	F13	1:4	5	1,5	6
	F01	1:5	5	1,5	6
	E01	1:5	7,5	1,5	6
	E02	1:5	10	1,5	6
	H01	1:5	5	0,5	5
	H02	1:5	5	2,5	7
	F17	1:6	5	1,5	6
	F21	1:7	5	1,5	6
	F25	1:8	5	1,5	6
Medium packed Silt	F08	1:2	5	1,5	6
	F12	1:3	5	1,5	6
	F16	1:4	5	1,5	6
	F04	1:5	5	1,5	6
	E07	1:5	7,5	1,5	6
	E08	1:5	10	1,5	6
	H07	1:5	5	0,5	5
	H08	1:5	5	2,5	7
	F20	1:6	5	1,5	6
	F24	1:7	5	1,5	6
	F28	1:8	5	1,5	6

10.2.2 Variation of initial side slopes

To study the existence of some sort of 'equilibrium slope', simulations with different initial slopes have been done for four different reference soils: MCS, MMS, MFS and MSI. Will these slopes evolve to the same slope? All other input parameters were kept constant, but the initial slopes were varied from 1:8 to 1:2. By way of illustration, the channel development in MFS is plotted in Figure 10-8. As can be seen, the slopes are slightly flattening and the water depth at the middle of the channel bottom remains constant. The downstream slope clearly starts to become flatter than the upstream slope and the entire channel starts to migrate in downstream direction.

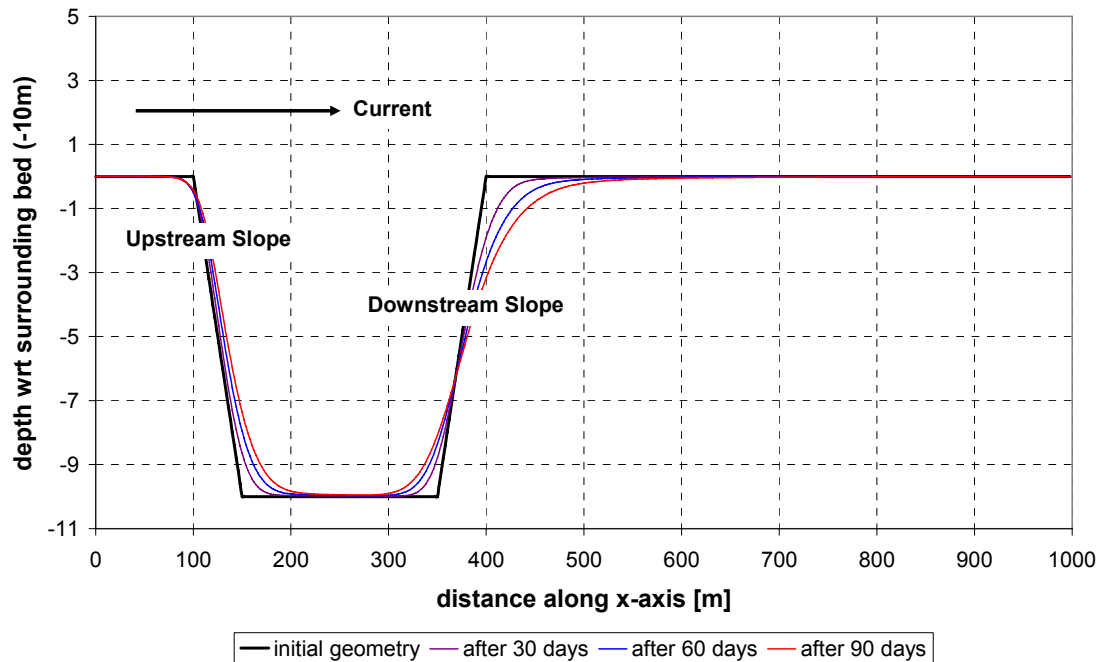


Figure 10-8: Developing slopes in unidirectional flow ($u = 0,5 \text{ m/s}$) in MFS (output simulation 'F01')

As can be seen from Figure 10-9, where slope development is plotted for different initial slopes in MCS, the upstream slopes are converging, but with a slight tendency to flatter slopes. At $t = 0$ days, all initial slopes are plotted on the y-axis. After 90 days in unidirectional flow the upstream slopes are all in the order of 1:6 to 1:8. The downstream slopes, which have not been plotted show a similar behaviour with slopes after 90 days in the order of 1:8 to 1:10. Remarkably all slopes flatter than 1:4 first show steepening behaviour, which can be important for slope stability.

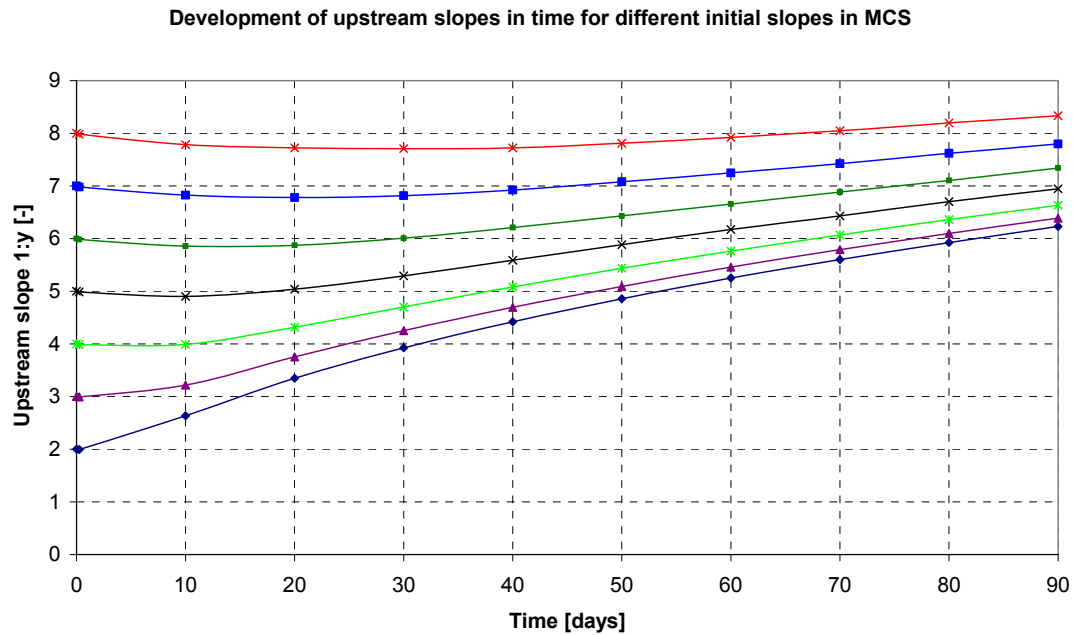


Figure 10-9: Slope development in unidirectional flow in MCS

When the grain size and the particle fall velocity are reduced (MMS), it appears that the upstream slopes become somewhat steeper, see Figure 10-10

Figure 10-10: Slope development in unidirectional flow in MMS

. After 90 days most upstream slopes are in the order of 1:6 to 1:7, downstream slopes 1:8 to 1:10.

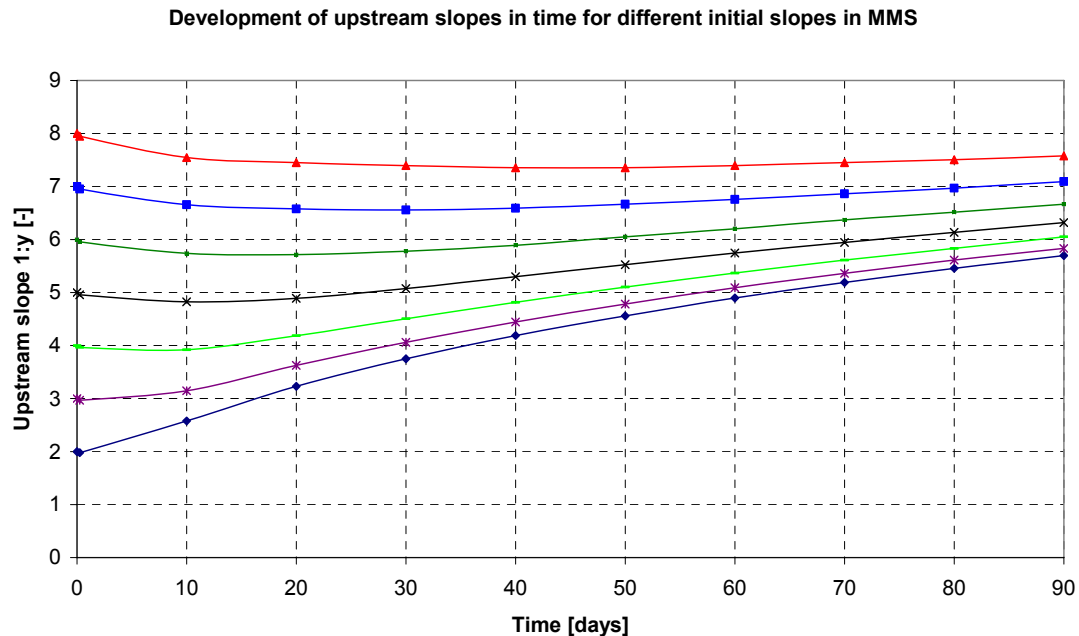


Figure 10-10: Slope development in unidirectional flow in MMS

Slope development in MFS shows similar behaviour: the upstream slopes become somewhat steeper (1:5 to 1:6), the downstream slopes somewhat flatter (1:9 to 1:11), see Figure 10-11. Again the upstream slopes will steepen, at least in the beginning except the '1:2 initial slope'.

Development of upstream slopes in time for different initial slopes in MFS

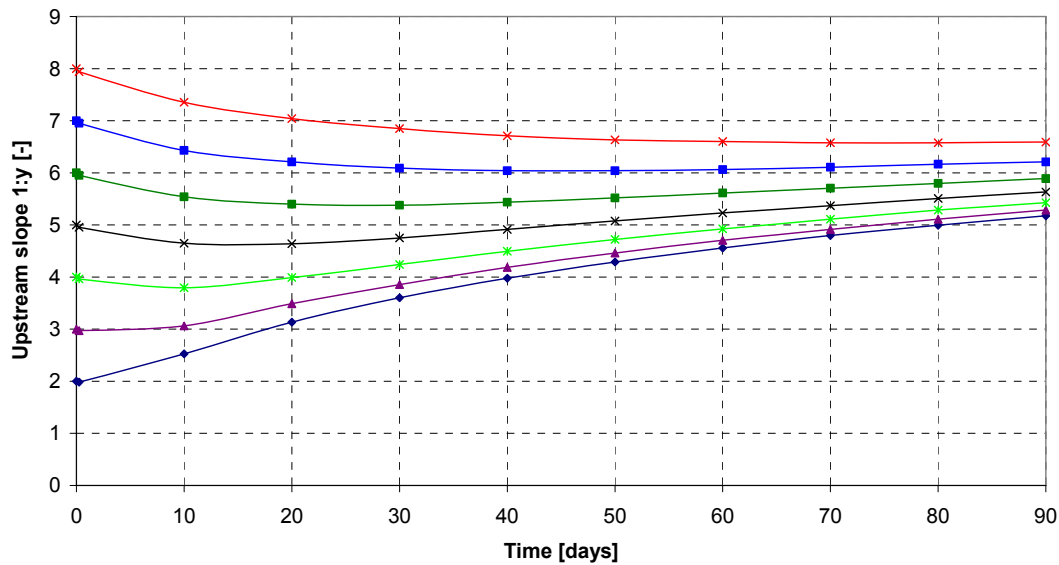


Figure 10-11: Slope development in unidirectional flow in MFS

The finest sediment under consideration, MSI, is transported in such large amounts under these flow conditions that both side slopes soon start to 'feel' each other. Because of the large sedimentation over the entire trench bottom, due to the very small fall velocity of the silt particles, slope flattening occurs very fast and is accelerated (at about $t = 60$ days) when the toes of both side slopes start to coincide, see Figure 10-12. As a result the upstream slope has become 1:22 and the downstream slope 1:33 with a remaining channel depth of only 3,6 m, which can be considered as 'failure of the channel'.

Development of upstream slopes in time for different initial slopes in MSI

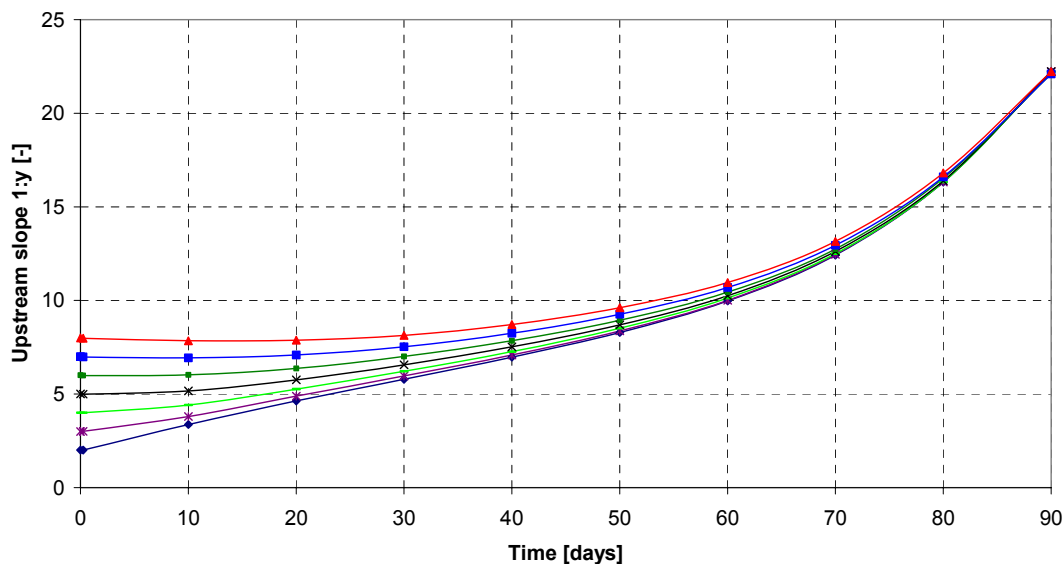


Figure 10-12: Slope development in unidirectional flow in MSI

Summarizing it can be concluded that under these flow conditions steepest upstream slopes are to be expected in fine sands (MFS). This soil combines the property of easily being picked up by current and waves and therefore transported in significant amounts and the property of a relatively large particle fall velocity, so grains start settling on the slope as soon the current passes over the channel, unlike silt which settles over the entire channel width.

10.2.3 Variation of unidirectional flow velocity

The development of side slopes in the previous paragraph was considered under just one hydrodynamic condition. Will this development be affected by other current velocities or wave heights? Now, simulations with three depth-averaged current velocities (0,5, 0,75 and 1,0 m/s) will be done. All of the current velocities under investigation are able to pick up at least some sediment and are representative for marine conditions. Larger flow velocities do occur, but mainly in tidal inlets, estuaries or closures. Eight simulations have been done, all with initial slopes of 1:5, and have been compared with the corresponding runs from the previous paragraph.

In Figure 10-13 can be seen that upstream slopes remain rather steep, while downstream slopes flatten significantly. In this MFS some reduction of the maximum channel depth and a little shift in downstream direction take place, which is an indication that a fair amount of total transport consists of suspended transport, which settles down not until the flow has reached the channel bottom. The upstream slope after 30 days seems to be steeper than the other plotted slopes, while the slope height is only slightly reduced.

Furthermore, the backfilling within the initial channel boundaries is plotted with respect to the initial channel geometry, which gives a good idea of the locations where sedimentation takes place.

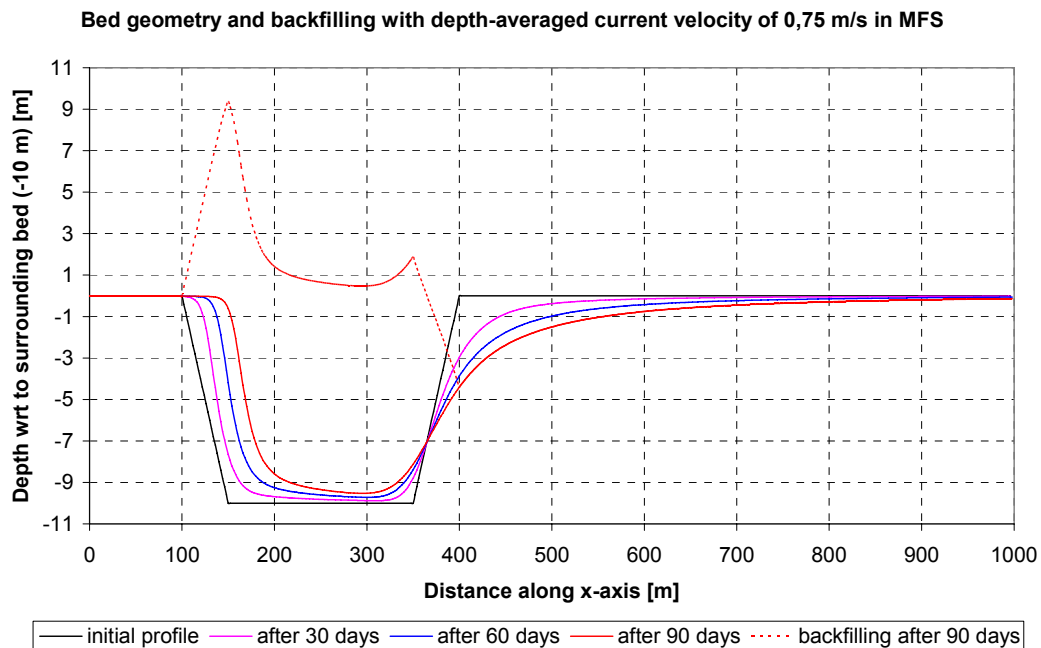


Figure 10-13: Development of bed geometry in time with depth-averaged current velocity of 0,75 m/s in MFS.

From Figure 10-14 can be concluded that an increase of the current velocity influences slope stability unfavourably in case the sediment diameter is in the order of 1 mm (MCS). Upstream slopes get steeper, if this velocity becomes larger than 0,75 m/s. One should note, that resulting slopes steeper than 1:2 may not be realistic, because of the existence of turbulent eddies behind the upstream slope and the already mentioned effect of increased bed-shear stresses on steep side slopes (see 10.2.1).

Besides, other instability mechanisms may already have caused slope failure, which SUTRENCH does not take into account. This observation is in line with observed failures in nature, where large supply of sediment by a river can cause slopes of its delta to collapse.

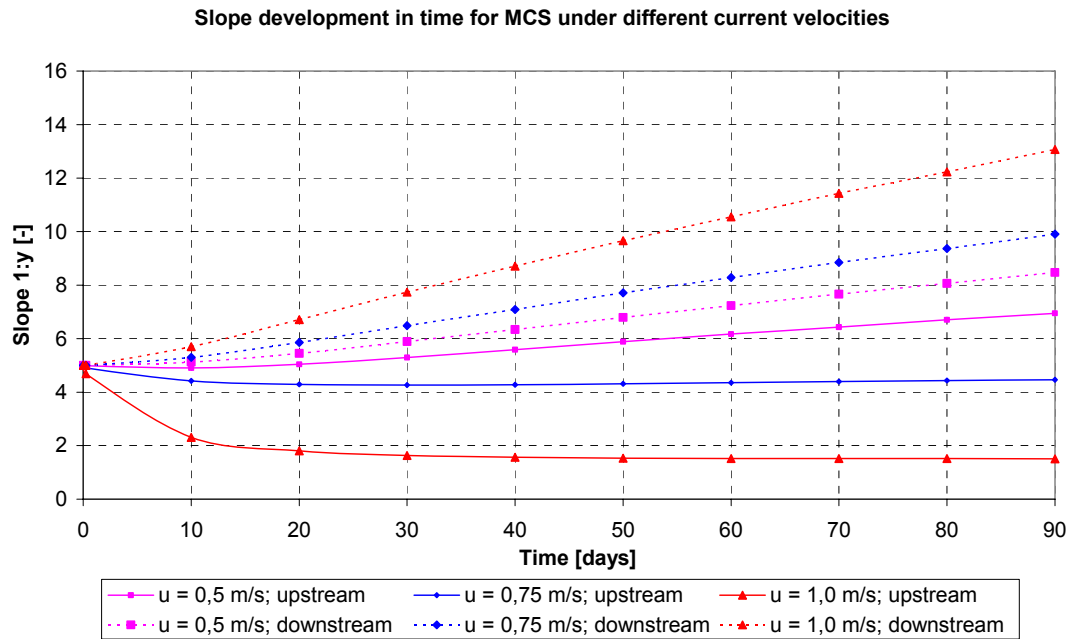


Figure 10-14: Slope development in unidirectional flow in MCS for different current velocities

The same behaviour can be observed with smaller diameters, see Figure 10-15 and Figure 10-16: for higher current velocities upstream slopes become steeper and downstream slopes flatter.

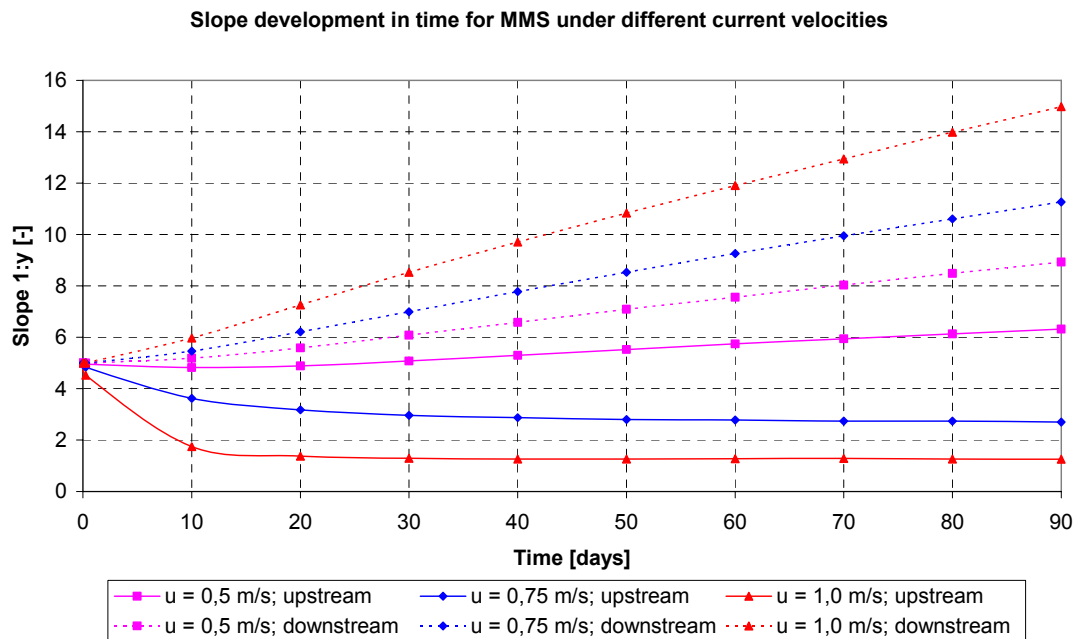


Figure 10-15: Slope development in unidirectional flow in MMS for different current velocities

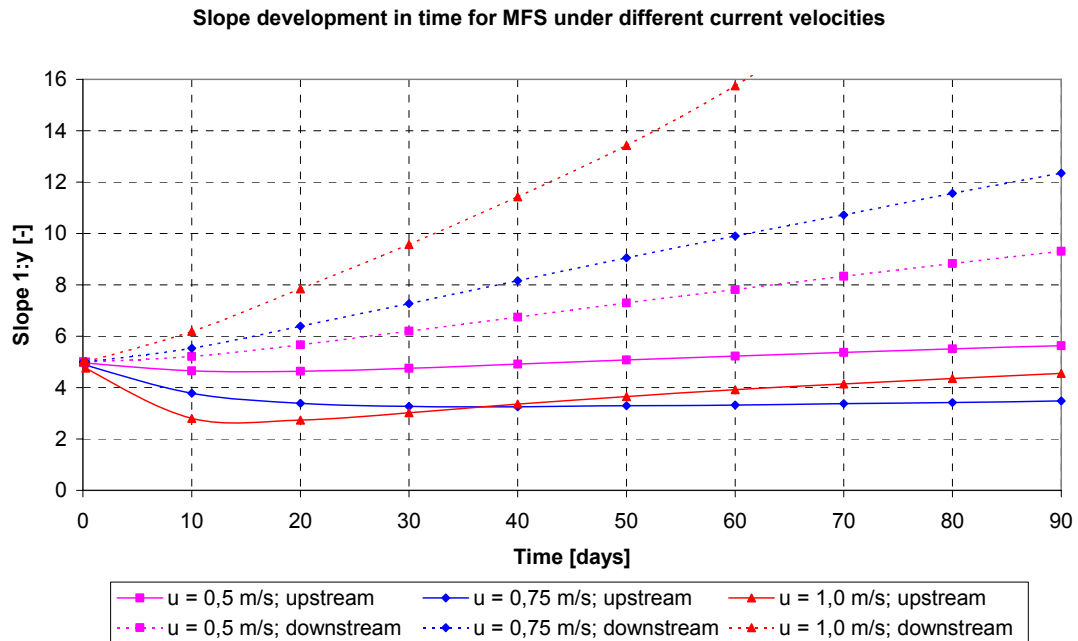


Figure 10-16: Slope development in unidirectional flow in MFS for different current velocities

The dip in the angle of inclination of the upstream slope for 1,0 m/s in Figure 10-16 is caused by infilling of the channel, which reduces the channel depth. This infilling becomes so large for MSI that it is only meaningful to speak about independent slope development in the first 30 days, see Figure 10-17. Because of the small fall velocity of silt, sediment is spread over the entire trench, which means that *slope steepening* is never to be expected.

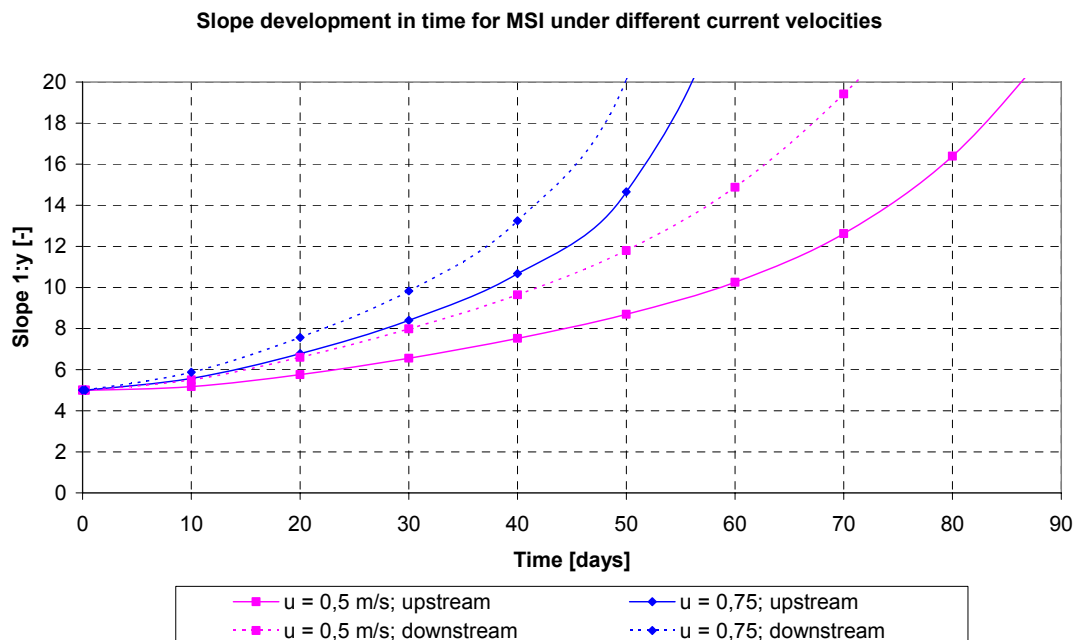


Figure 10-17: Slope development in unidirectional flow in MSI for different current velocities

A plot of the maximum slopes that do occur during this period of 90 days against grain diameters (Figure 10-18) shows that upstream slopes get steeper for larger current velocities and that the grain diameter for which the steepest slope occurs increases for increasing current velocities. This can be explained by the fact that weak currents hardly move any large sediment, but stronger currents are able to move more and more sediment, because of the strong nonlinear relationship between velocity and transport.

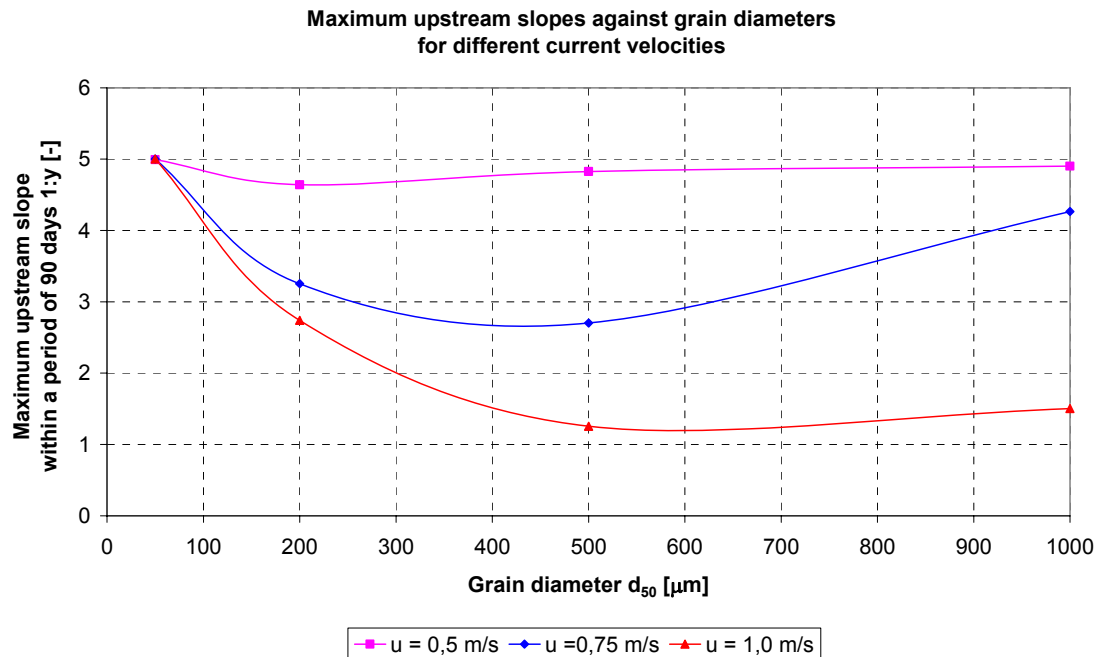


Figure 10-18: Maximum upstream slopes within a period of 90 days against grain sizes

While sediment transport is increasing, the fall velocity remains the same, resulting in large sedimentation as soon as the current enters the channel and slows down, and consequently building up steep slopes, see Figure 10-19 for the large difference in sediment transport. The gradient of this graph is a measure for erosion and sedimentation. The sediment transport capacity is reduced to zero at the upstream side slope for the largest sediment (MCS), because suspended transport has a negligible contribution to total sediment transport, which therefore reacts instantaneously to the changing bed level. MFS shows a rapid drop in total sediment transport at the upstream side slope, caused by changing bed load and suspended transport, and a more gradual decrease at the channel bottom, caused by the remaining suspended sediment transport.

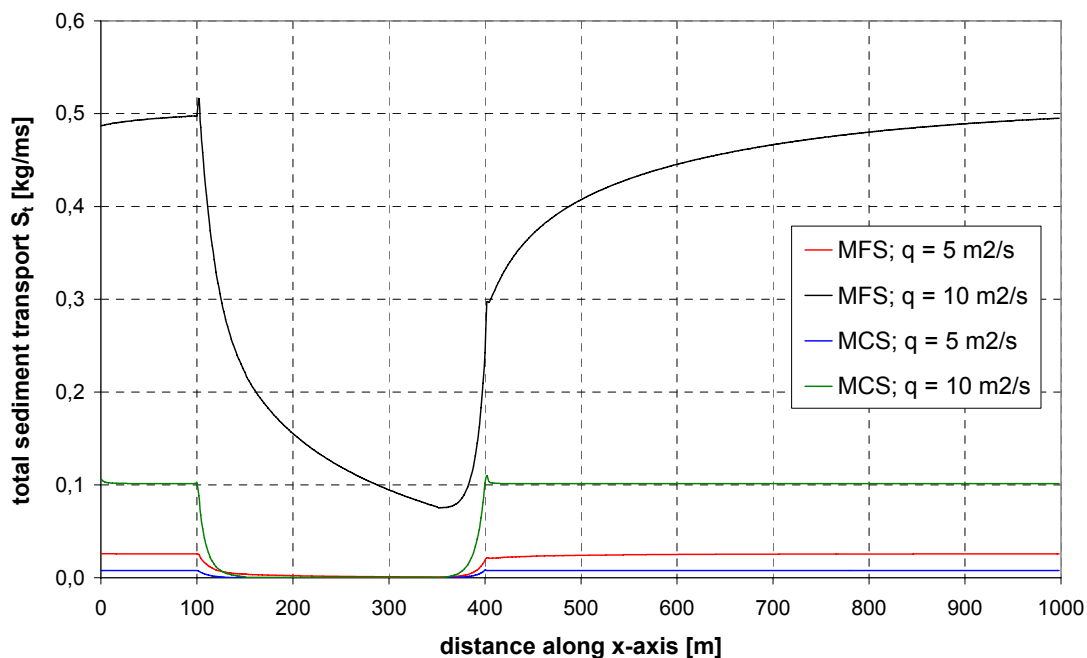


Figure 10-19: Total sediment transport for different current discharges in MCS and MFS

10.2.4 Variation of significant wave height

The same procedure is followed for variation of the wave height. When wave stirring increases, sediment transport increases. As can be concluded from Figure 10-20, Figure 10-21 and Figure 10-22 downstream slopes for all sediments become flatter as the wave height increases.

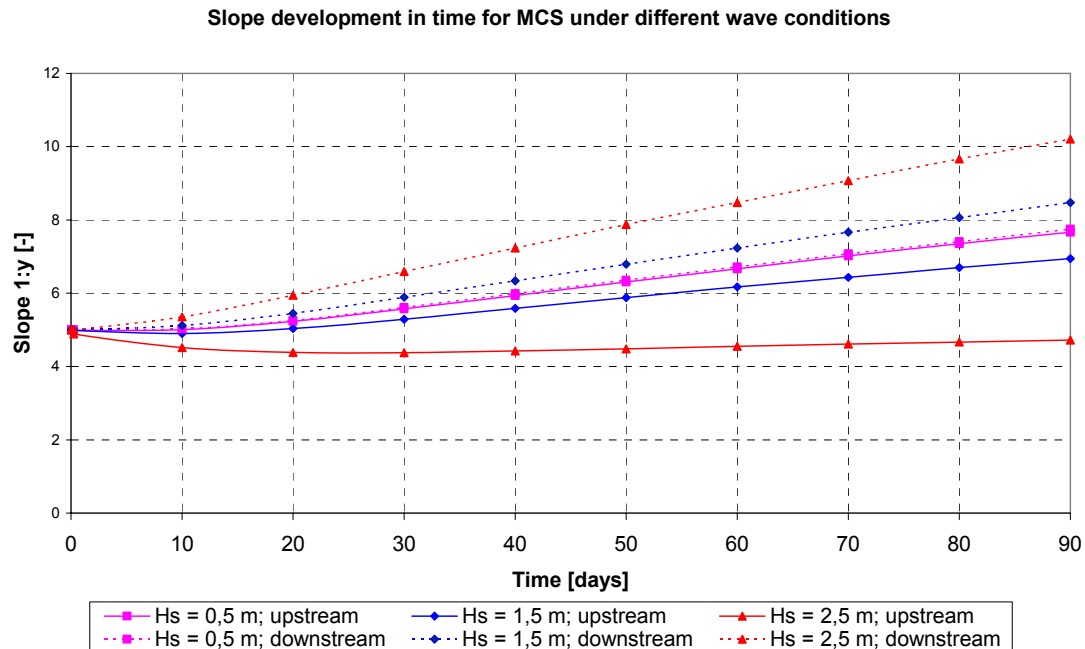


Figure 10-20: Slope development in unidirectional flow in MCS for different wave heights

The development of the upstream slopes depends on the wave height and soil type. Wave heights larger than approximately 2 m cause significant slope steepening, especially for grain sizes between 200 and 500 μm .

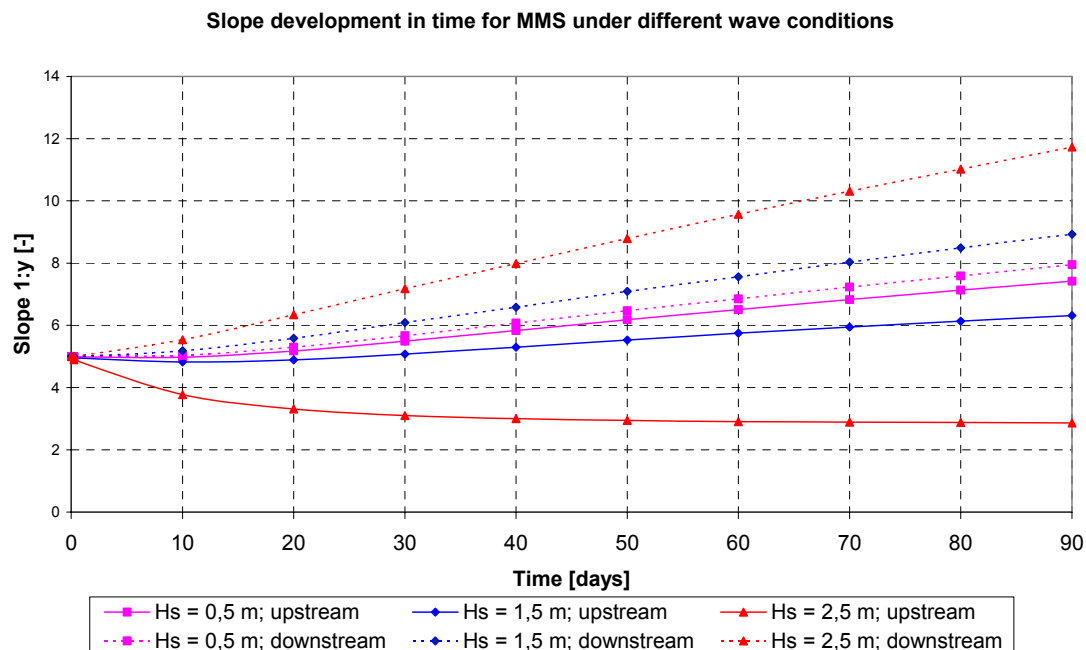


Figure 10-21: Slope development in unidirectional flow in MMS for different wave heights

Slope development in time for MFS under different wave conditions

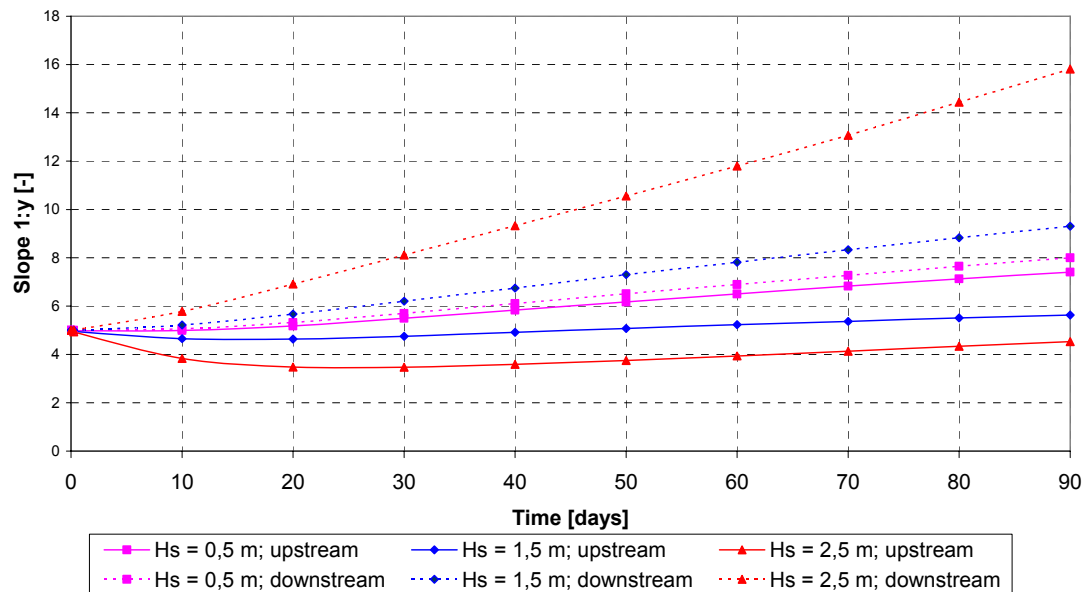


Figure 10-22: Slope development in unidirectional flow in MFS for different wave heights

For MSI opposite behaviour is again observed: larger wave heights cause rapid siltation and therefore flatter slopes. Another remarkable conclusion is that in absence of large waves, the development of upstream and downstream slopes is very similar for all soil types under consideration: all side slopes tend to develop to a slope of about 1:7,5. These phenomenon is coincidental, however can be explained. In absence of waves slope development is only governed by the unidirectional current. The upstream slope develops slowly to a relative flat slope, because of sedimentation and trench migration, while the downstream slope is hardly eroded, because the grains on the downstream slope experience a constant favourable influence of gravity without the cyclic wave-induced bed-shear stresses.

Slope development in time for MSI under different wave conditions

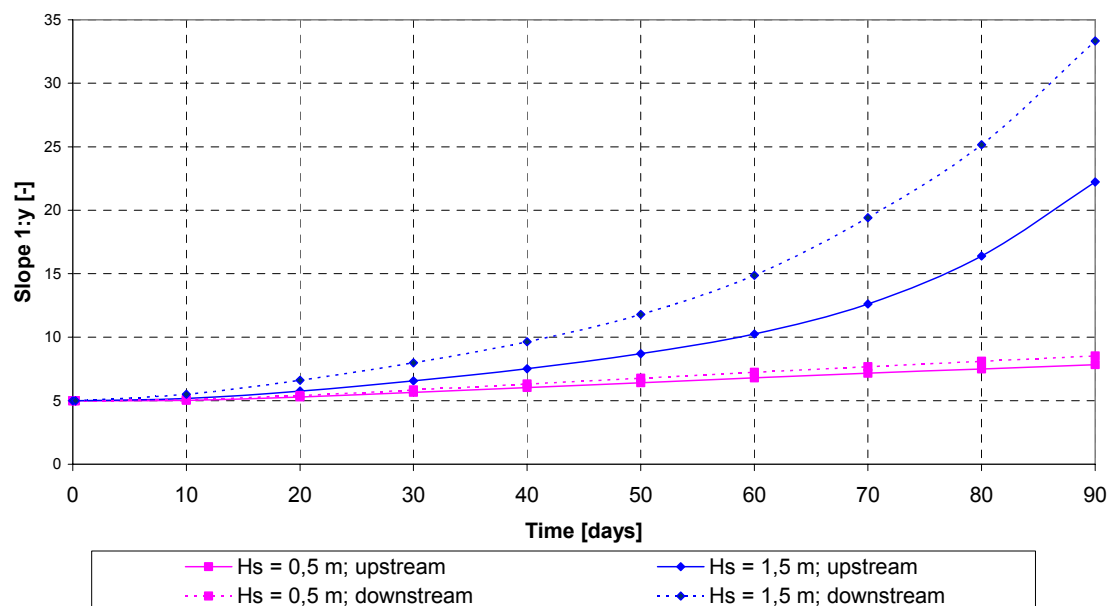


Figure 10-23: Slope development in unidirectional flow in MSI for different wave heights

The above four graphs are again summarized in one graph, see Figure 10-24. For most wave heights a slow flattening of the side slopes occurs (please note that the drawn lines represent the steepest upstream slope, which can be the initial slope of 1:5), but larger wave heights can result in steeper slopes. In the case of wave heights larger than 2,5 m, probably even steeper slopes can develop in sediments coarser than 500 μm , although the impact of bigger waves will become deeper and deeper, resulting in erosive activities along the side slopes.

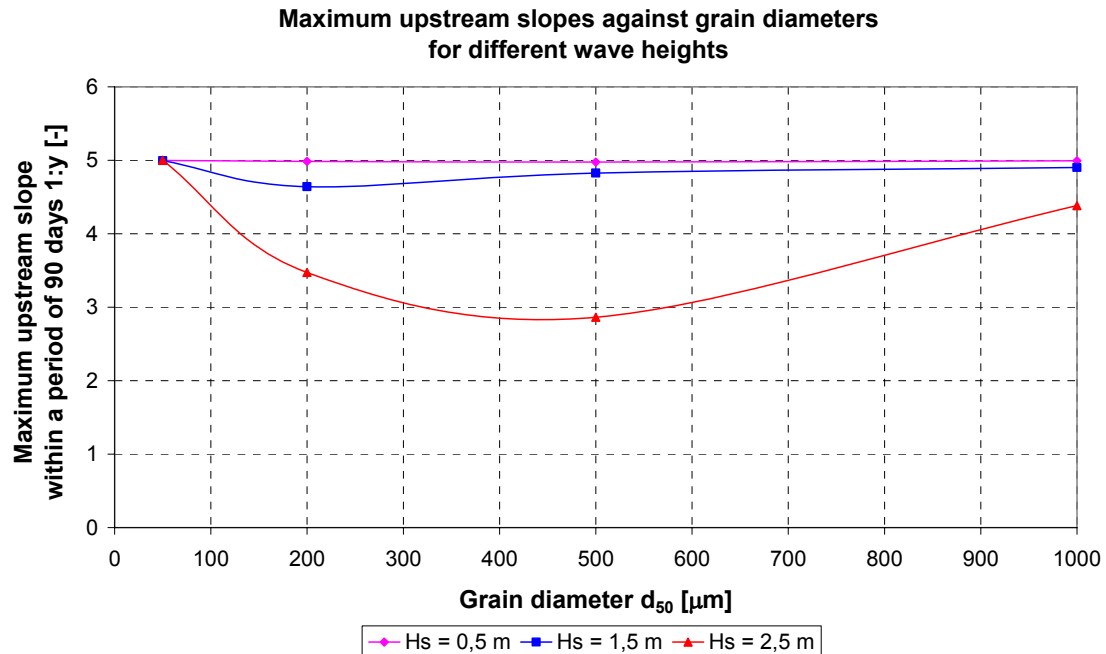


Figure 10-24: Maximum upstream slopes within a period of 90 days against grain sizes

10.3 Simulations with simple tidal flow

10.3.1 Introduction

In most natural situations tidal flow will occur instead of unidirectional flow. In this case trench migration will be much smaller and the difference between the so-called *upstream* and *downstream* slopes will be less significant and can be zero if the tidal flow is completely symmetrical. In this paragraph the tidal flow will be schematized to a simple block function, see Figure 10-25.

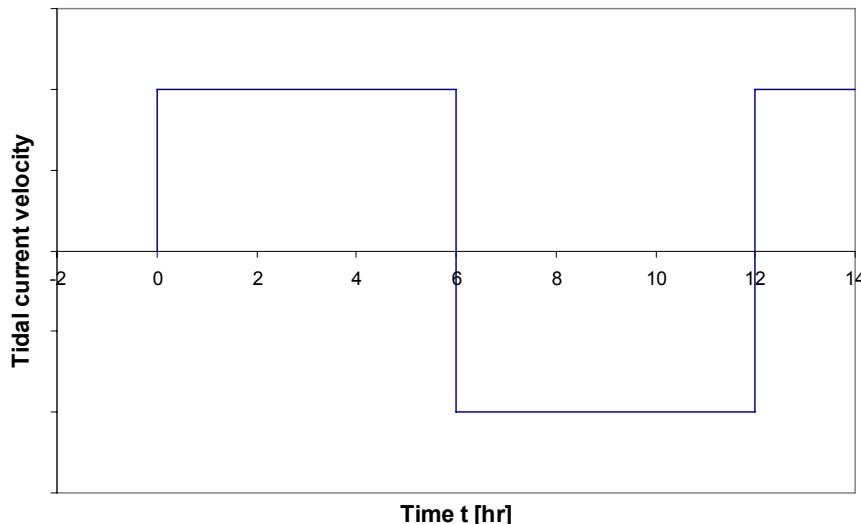


Figure 10-25: Block function of tidal current velocity [dimensionless]

Because of the equal absolute ebb and flood current velocities, there will be no difference between upstream and downstream slopes and no trench migration. The morphological changes will also be smaller under tidal conditions due to the turning current and sediment transport; therefore the simulation period is extended to 300 days (which is almost the entire period of interest of this thesis). The slope development during this period will be observed every 30 days to find out what the critical stage for every condition and sediment is.

Again calculations for different side slopes (1:3, 1:5 and 1:7), different (tidal) current velocities ($u = \pm 0,5, 0,75$ and $1,0$ m/s) and different wave heights ($H_s = 0,5, 1,5$ and $2,5$ m) will be done for four non-cohesive reference soils (MCS, MMS, MFS and MSI).

As was explained in the sensitivity analysis, 'numerical smoothening' is important in case of very steep profiles or sharp discontinuities; otherwise instabilities will occur in the bottom profile or sediment transport, which can hardly be prevented by decreasing the time step Δt . In unidirectional flow, the problem of 'oversteepening' of the upstream slope asked for a rather large smoothening coefficient ($\alpha = 0,5$). In tidal flow the morphological development of the trench is much more symmetrical, allowing the application of less numerical smoothening. A lot of calculations with different space and time steps and smoothening coefficients showed that with space step $\Delta x = 2$ m, time step $\Delta t = 3$ hr and $\alpha = 0,01$ no instabilities occur and the representation of the development of the trench profile is sufficient accurate: smaller numerical smoothening causes instability, while the profile shows hardly any changes, when the 'instability peaks' are neglected. The main part of the side slopes is not very sensitive to a (further) decrease of the smoothening coefficient; the steepness also remains the same. Only at the top and toe of the slope small differences can be observed, which are not relevant when considering slope stability.

Table 10-5: Variation of different parameters in simulations with simple tidal flow

	Name	Initial Slope	Current Discharge	Wave Height	Wave Period
		-	m ³ /s/m	m	s
Medium packed Coarse Sand	DC1	1:5	5	1,5	6
	DC2	1:3	5	1,5	6
	DC3	1:7	5	1,5	6
	DC4	1:5	7,5	1,5	6
	DC5	1:5	10	1,5	6
	DC6	1:5	5	0,5	5
	DC7	1:5	5	2,5	7
Medium packed Medium Sand	DM1	1:5	5	1,5	6
	DM2	1:3	5	1,5	6
	DM3	1:7	5	1,5	6
	DM4	1:5	7,5	1,5	6
	DM5	1:5	10	1,5	6
	DM6	1:5	5	0,5	5
	DM7	1:5	5	2,5	7
Medium packed Fine Sand	DF1	1:5	5	1,5	6
	DF2	1:3	5	1,5	6
	DF3	1:7	5	1,5	6
	DF4	1:5	7,5	1,5	6
	DF5	1:5	10	1,5	6
	DF6	1:5	5	0,5	5
	DF7	1:5	5	2,5	7
Medium packed Silt	DS1	1:5	5	1,5	6
	DS2	1:3	5	1,5	6
	DS3	1:7	5	1,5	6
	DS4	1:5	7,5	1,5	6
	DS5	1:5	10	1,5	6
	DS6	1:5	5	0,5	5
	DS7	1:5	5	2,5	7

10.3.2 Variation of initial side slopes

A number of 12 simulations have been done with initial side slopes of 1:3, 1:5 and 1:7 and a constant depth-averaged tidal flow velocity of 0,5 m/s and a wave height of 1,5 m. Because of the turning current the side slopes also will undergo a turning development, for example during flood steepening and during ebb flattening of the slope. Slope geometries are therefore expected to develop slower compared to unidirectional flow and slope angles will be somewhere between the upstream values (upper boundary) and downstream values (lower boundary) of the previous paragraph.

All simulations were done with $\Delta x = 2$ m, $\Delta t = 3$ hr and $\alpha = 0,01$, except for the 1:3 and 1:5 side slopes in MMS. These simulations showed small signs of instability at the top of the slope. Therefore the time step was halved to 1,5 hr. The results then showed slopes very similar to the initial simulations with the larger time step, but the instabilities disappeared.

It appears that the channel development within one year remains limited to the side slopes for sediments larger than 200 μ m (MCS and MMS) under these hydrodynamic conditions. The channel depth remains constant, because adaptation of the sediment profile is restricted to the slopes because of the large particle fall velocity and the small total sediment transport, which is mainly in the form of bed load transport (about 80% for MMS and over 99% for MCS).

In Figure 10-26, it can be seen that there is some sedimentation (± 16 cm) on the channel bottom in MFS and only minor slope development. At first sight, it can be observed that all initial slopes are moving towards a *dynamic equilibrium slope*: the 1:3-slope is flattening, the 1:7-slope is steepening and the 1:5-slope stays more or less the same, indicating that a side slopes of 1:5 is approximately the *dynamic equilibrium slope* for these conditions (water depth, tidal current and wave height). This morphological behaviour can completely be explained by the distribution of the total sediment transport into bed load and suspended transport. The bed load transport diminishes when passing over the slope. At the toe of the slope, there is only little bed load transport; when the seabed becomes flat at the channel bottom, there is only suspended sediment transport left, resulting in a small sedimentation on the channel bottom.

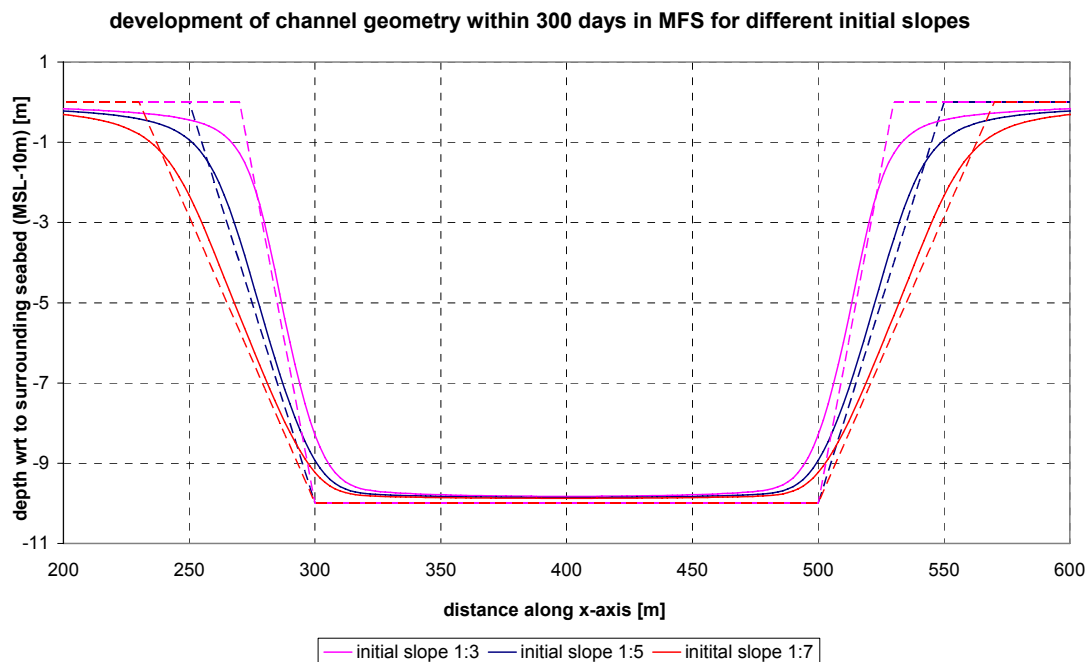


Figure 10-26: Development of channel geometry for three initial slopes in tidal flow in MFS

Smaller grain particles like MSI show complete silting up of the channel within one year. Because of the small particle fall velocity, slopes start to flatten instantaneously and the sedimentation pattern extends over the entire width. Sediment transport is almost 100% in the form of suspended transport.

When varying the side slopes in MCS, very slow slope development was observed due to the rather small sediment transport capacity ($S_{\text{tot}} = 0,69 \cdot 10^{-2} \text{ kg/ms}$), see Figure 10-27. Extrapolating the three lines results in an equilibrium slope of about 1:4,5 to 5. Required channel depth is guaranteed for years; slope stability problems are not to be expected. After all, morphological timescales are large, sedimentation remains limited, so good wave compaction can be reached and liquefaction problems are not to be expected, because of the large grain sizes, good permeability and consolidation properties and large relative densities. Rapid *slope steepening* does not occur and shear failure is not to be expected. Steep initial slopes like the 1:3-slope will slowly be flattened by waves and currents, so the initial situation will be normative.

Development of side slopes in time for different initial slopes in tidal flow in MCS

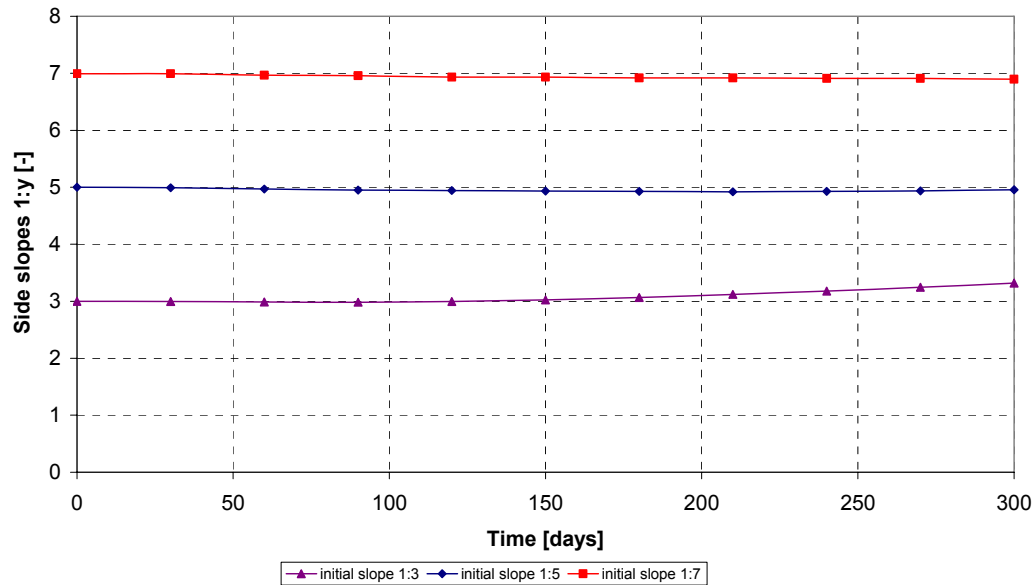


Figure 10-27: Slope development for initial slopes 1:3, 1:5 and 1:7 in tidal flow (MCS)

In medium sands (MMS) the situation becomes slightly different:

- sediment transport capacity is almost doubled ($S_{tot} = 0,013 \text{ kg/ms}$);
- not only bed load transport, but also suspended transport ($\pm 20\%$ at the surrounding seabed) occurs;
- particle fall velocities become smaller.

This results in a faster morphological development judging by the faster converging lines in Figure 10-28. Also the 'width of influence' extends: the sediment concentration vertical is 'emptied' over the entire slope. The main consequence is a small *flattening* of the slope to values of 1:6 to 7. The sedimentation on the channel bottom remains limited to a few dm's on the sides to zero in the middle. Again the normative situation is the initial slope. Medium sands become more susceptible to (cyclic) liquefaction, primarily depending on relative density.

Development of side slopes in time for different initial slopes in tidal flow in MMS

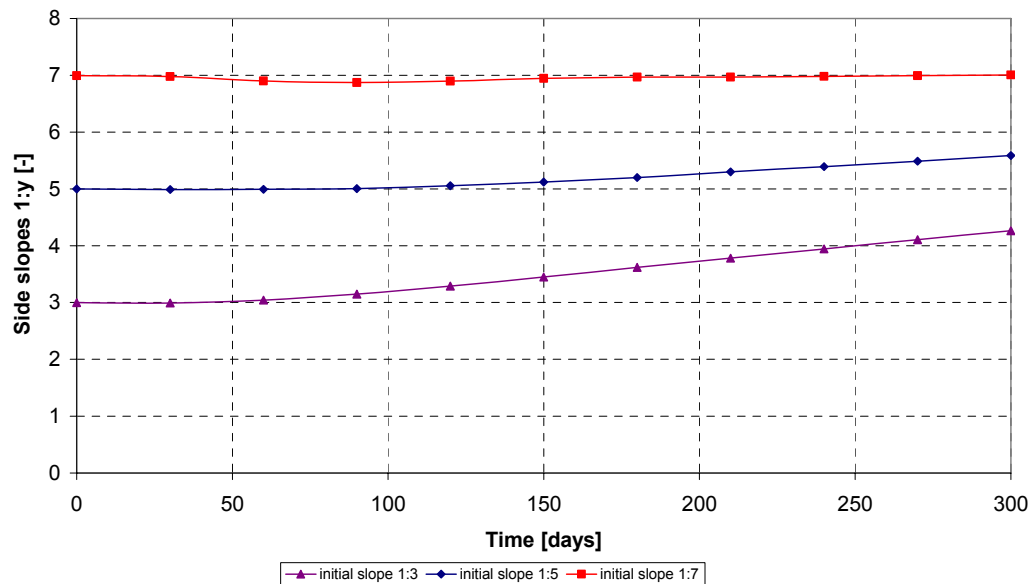


Figure 10-28: Slope development for initial slopes 1:3, 1:5 and 1:7 in tidal flow (MMS)

When the grain sizes become smaller (MFS), the following can be observed:

- sediment transport capacity is increased ($S_{\text{tot}} = 0,024 \text{ kg/ms}$);
- particle fall velocities become smaller;
- a significant part of the total sediment transport is suspended transport (72%).

Following the same line of thought, a further slope flattening would be expected. The simulations, however, show a different behaviour: slopes in MFS are steeper than in MMS. Equilibrium slopes are in the order of 1:4, even steeper than in MCS. Slope development is dependent on the gradient of sediment transport. So, although the sedimentation width extends over the entire upstream side slope and the channel bottom, due to the high contribution of suspended transport, the local gradient at the upstream slope still is larger than for MCS and MMS because of the larger sediment transport at the surrounding bed.

Development of side slopes in time for different initial slopes in tidal flow in MFS

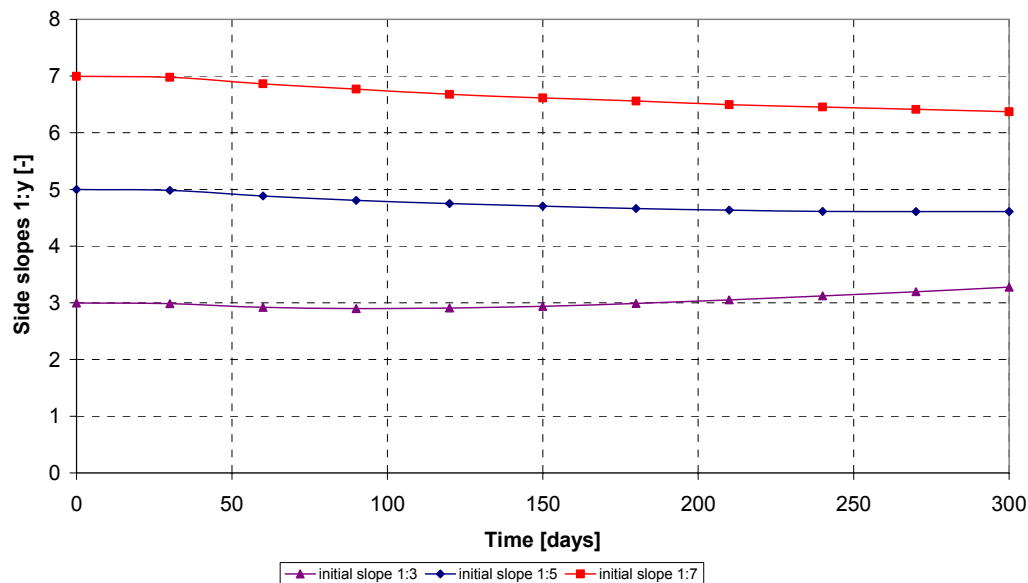


Figure 10-29: Slope development for initial slopes 1:3, 1:5 and 1:7 in tidal flow (MFS)

A further decrease of the grain size to 50 μm (MSI) again shows different behaviour, see Figure 10-30. The silting up proceeds so fast that it is almost impossible to draw conclusions on autonomous slope development. Due to the high contribution of suspended transport to total sediment transport, the actual angle of inclination of the side slopes hardly affects sediment transport.

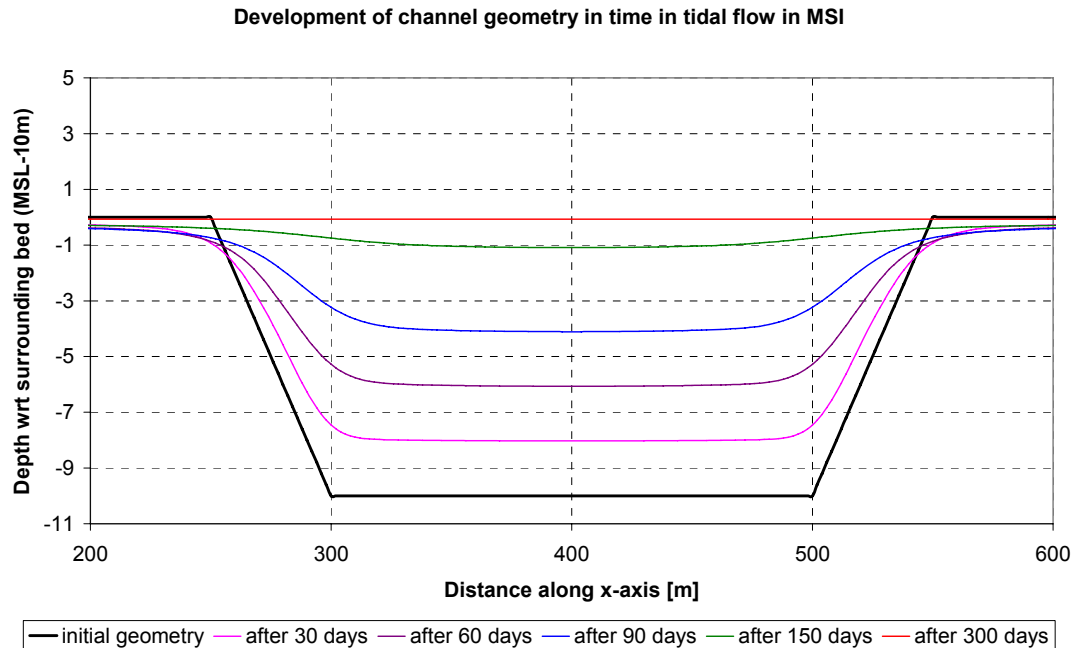


Figure 10-30: Morphological development of channel geometry in MSI

In Figure 10-31 (please note the different y-scale compared to the other reference soils) can be seen that during the first three months, the simulations with different side slopes show some differences, but after about 90 days an accelerated slope flattening takes place. From that moment slope heights are drastically reduced and both side slopes start to interfere with each other.

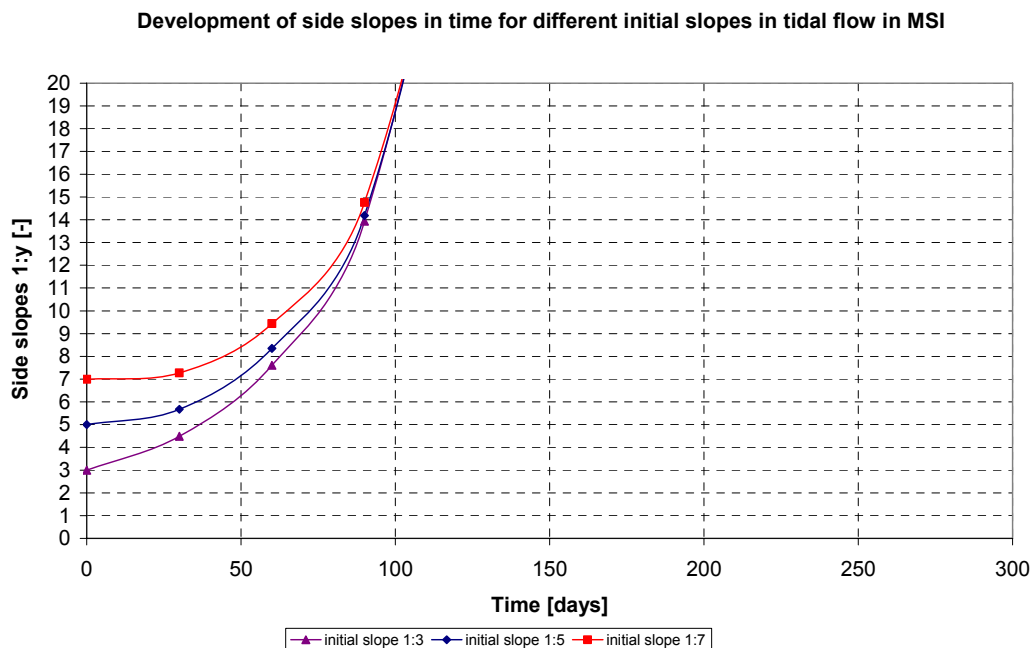


Figure 10-31: Slope development for initial slopes 1:3, 1:5 and 1:7 in tidal flow (MSI)

In MSI, it is completely irrelevant to speak of *dynamic equilibrium slopes* and the design of an appropriate slope should be based on other considerations than morphological development of the side slopes:

- which slope angle is safe from geotechnical point of view, while taking into consideration the rather short period this slope will have to persevere under local hydrodynamic conditions?
- which channel geometry is able to reduce overall sedimentation within the channel boundaries and to maintain required navigable depths as long as possible?

In this respect it is useful to consider trapping efficiencies of the three channel geometries, see Table 10-6. Flatter slopes result in larger upper widths of the trench, if the bottom width remains the same, and therefore larger trapping efficiencies and initial capital dredging volumes; starting from side slopes of 1:5, an increase of the slope steepness to 1:3 brings along a decrease of 8% in capital dredging volume, a decrease of the slope steepness to 1:7 results in an increase of 8%. These percentages of course rise if required channel bottom widths get smaller. So, designing flatter slopes improves slope stability, but increases construction costs.

Table 10-6: Sediment transport and trapping efficiency of three channel geometries

initial slope	sediment transport at surrounding seabed			minimum sediment transport inside channel			trapping efficiency	average rise of channel bottom after 30 days	sedimentation on channel bottom after 30 days
	$S_{tot}(0)$	$S_b(0)$	$S_s(0)$	$S_{tot}(0)$	$S_b(0)$	$S_s(0)$			
$\tan(\beta)$	[kg/ms]	[%]	[%]	[kg/ms]	[%]	[%]	[%]	[m]	[%]
1:3	2,12E+00	0,06%	99,94%	1,87E+00	0,00%	100,0%	11,9%	2,84	94,0%
1:5	2,12E+00	0,06%	99,94%	1,81E+00	0,01%	99,99%	14,8%	2,55	86,3%
1:7	2,12E+00	0,06%	99,94%	1,76E+00	0,02%	99,98%	17,1%	2,37	80,5%

From this table and also from Figure 10-32 can be concluded that a larger trapping efficiency does not inevitably result in a larger rise of the channel bottom. This can be explained by the fact that gentle slopes, which extend over a larger width, act themselves as *sediment trapping reservoir*: the actual sedimentation on the channel bottom as a percentage of the total sedimentation within the channel boundaries can be reduced from 86,3% to 80,5%, when flattening the side slopes from 1:5 to 1:7. On the other hand, the capital dredging volume is increased by 8% and the real advantage in time is only about 4 days. Variation of the side slopes is therefore not recommended for fine sediments, see Chapter 11. Keeping the upper width as small as possible and dredging of an overdepth seems the best construction method for sediments smaller than 200 μ m.

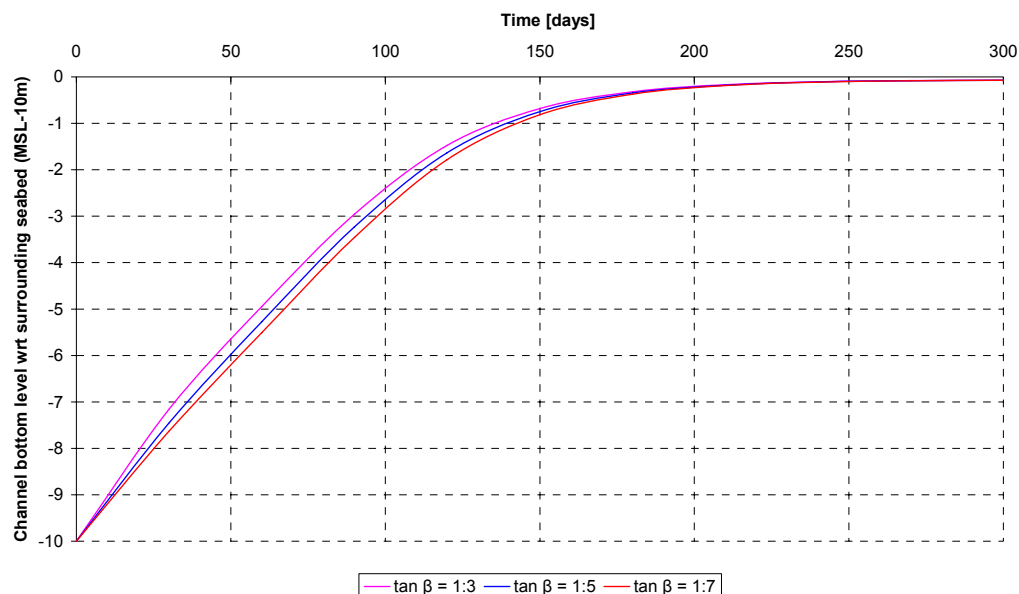


Figure 10-32: Average location of channel bottom for different initial side slopes

It can be seen that sediment transport in MSI is mainly suspended transport. Inside the channel there is hardly any bed load transport. The fact that bed load transport on more gentle slopes has a larger share in total sediment transport depends on the actual water depth where this minimum transport takes place; this depth is smaller for more gentle slopes, so current and wave impact on the seabed is felt better.

10.3.3 Variation of tidal flow velocity

The simulations of the previous paragraph were done for only one tidal current velocity. Now, the influence of the current velocity is investigated by varying the constant tidal current of Figure 10-25: $u_{\text{tidal}} = 0,5; 0,75; 1,0$ m/s.

The coarse sand (MCS) shows an interesting behaviour. For a tidal current velocity of 0,75 m/s, a small bend is located at a channel depth of 5 m (water depth of 15 m), see Figure 10-33; when the tidal current velocity is further increased this bend has disappeared and the bed geometry is smooth again (straight middle part and rounded top and toe of the slope). This phenomenon probably has something to do with depths of influence of both current and waves. In comparison to a depth-averaged tidal current of 0,5 m/s, bed-load transport is tripled and suspended transport is even multiplied by 142! Nevertheless, all morphological development occurs on the side slopes due to the still high particle fall velocity. It is likely that the immediate reacting bed-load transport and the somewhat lagging suspended transport create such an interesting profile.

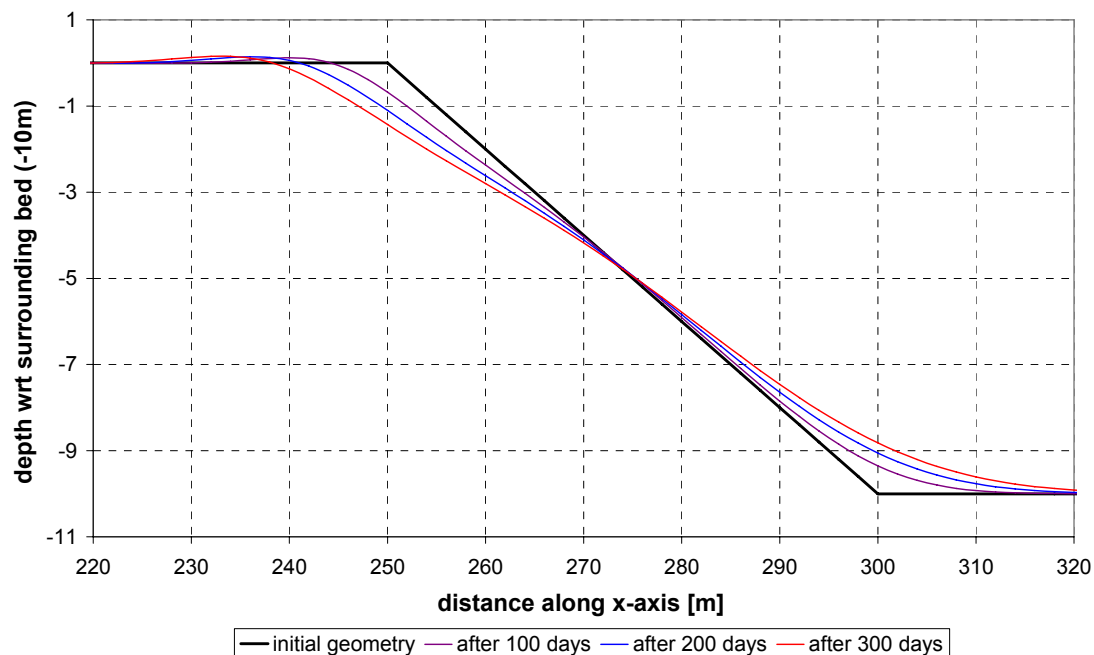


Figure 10-33: Slope development in time of MCS with a tidal velocity of 0,75 m/s

A further increase of the tidal current velocity up to 1,0 m/s raises the share of suspended transport to 50%, while the bed load transport reaches to the channel bottom. The transition from hardly any sediment transport to a considerable transport complicates drawing conclusions. However, it is certain that slope angles will not alter too much, see Figure 10-34.

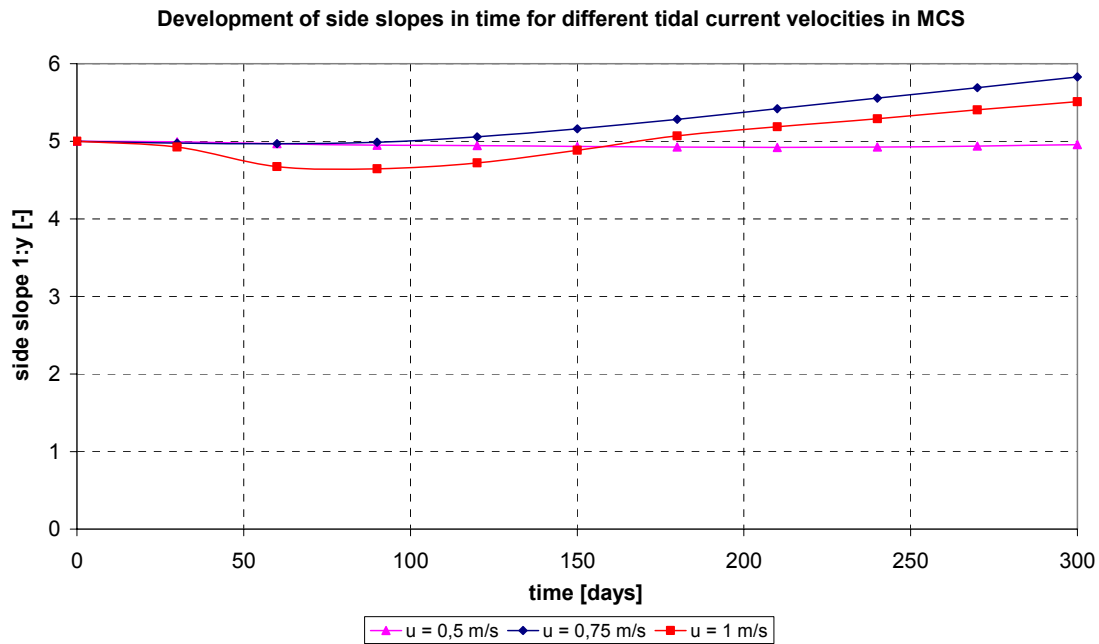


Figure 10-34: Slope development in tidal flow in MCS for different current velocities

The medium sand (MMS) shows a familiar behaviour: an increase of the current velocity results in steeper side slopes, see Figure 10-35.

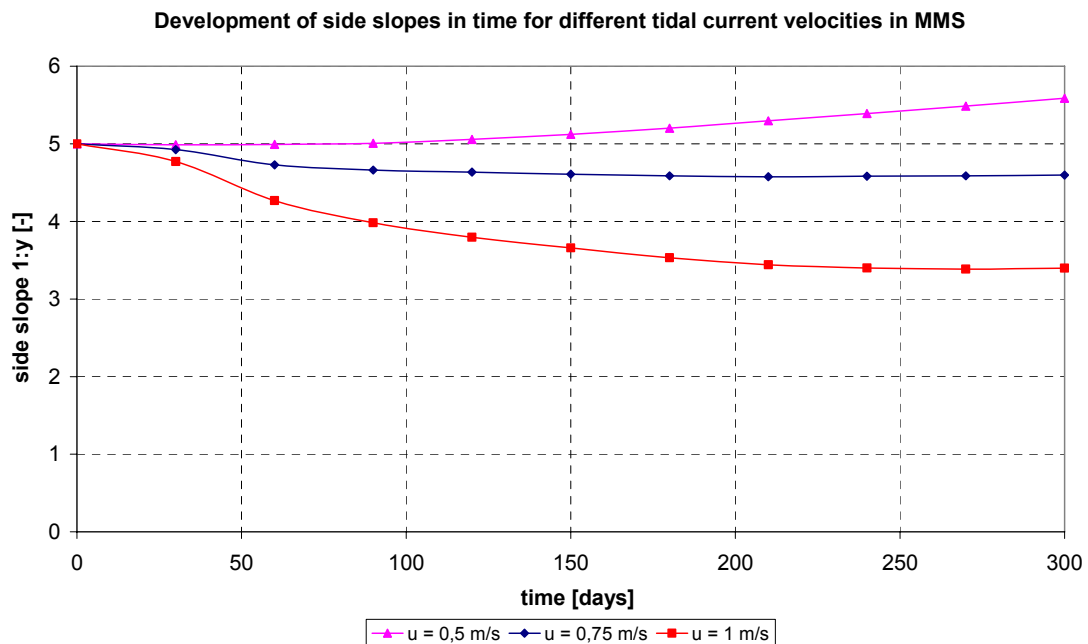


Figure 10-35: Slope development in tidal flow in MMS for different current velocities

In MFS the same slope steepening process is observed, until the channel starts to silt up. In the largest tidal current (1,0 m/s), this *silting up-process* is already noticeable after 60 days, see Figure 10-36, resulting in a rapid slope flattening.

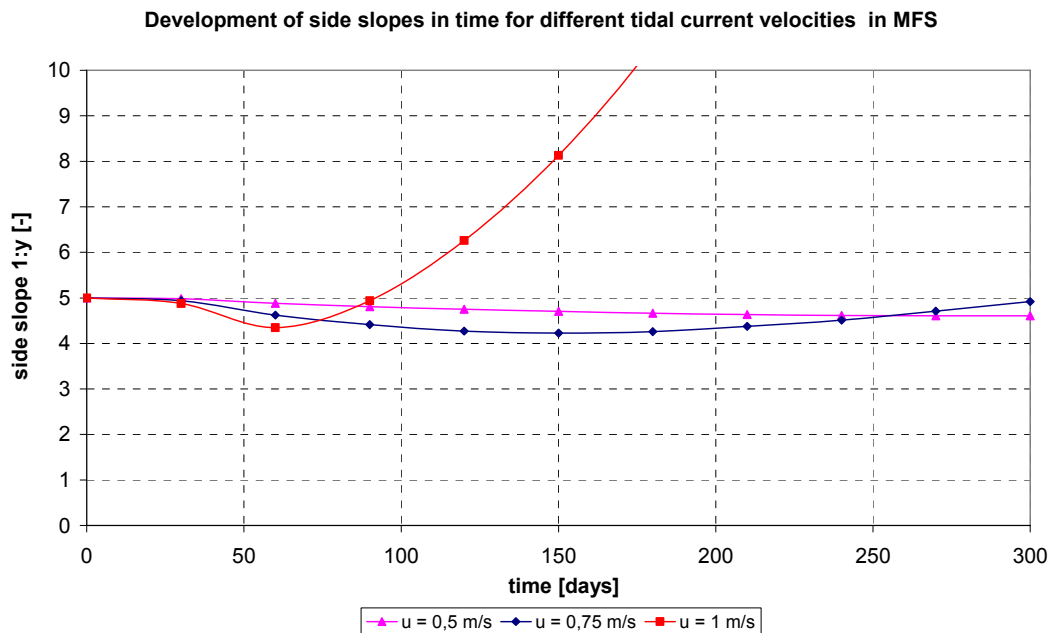


Figure 10-36: Slope development in tidal flow in MFS for different current velocities

As was mentioned before, homogeneous silts are that easily movable by even mild currents that one cannot speak of autonomous slope development. After respectively 30, 60 and 90 days, the channel is almost completely silted up, see Figure 10-37.

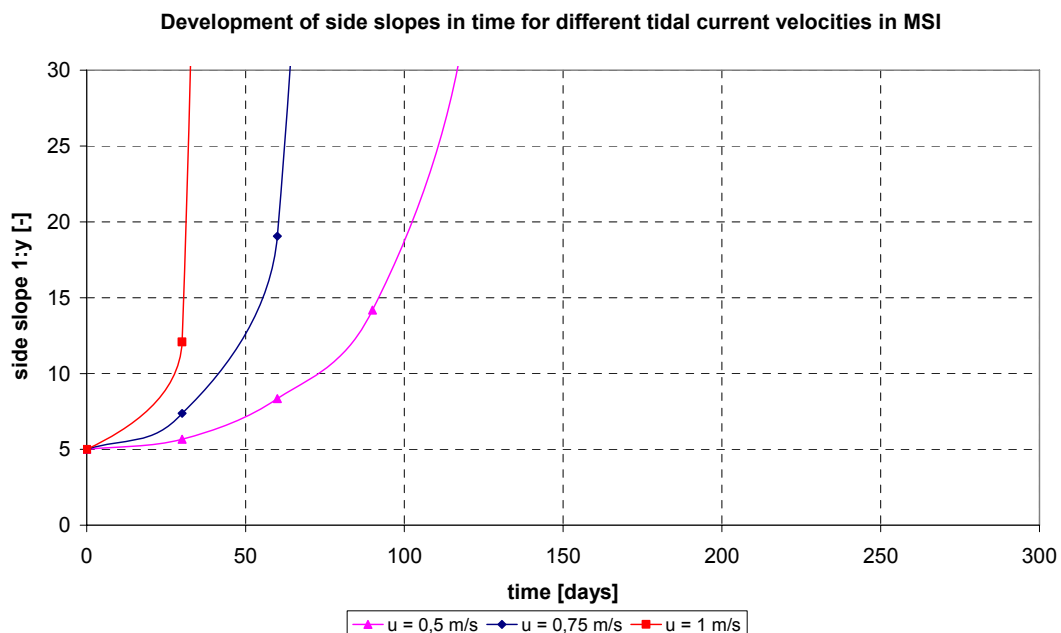


Figure 10-37: Slope development in tidal flow in MSI for different current velocities

Summarizing all of the above simulations leads to the following conclusions, see Figure 10-38:

- It is demonstrated that larger tidal current velocities cause larger *slope steepening* just like in unidirectional flow, although this steepening process progresses more slowly and is less extreme than in unidirectional flow;
- Fine to medium sands are most susceptible to this slope steepening. Coarse sands are hardly transported and very fine sands and silts are transported in large amounts, primarily as suspended transport. The sedimentation pattern is regularly spread out over the entire channel bottom and one cannot speak of autonomous slope development.

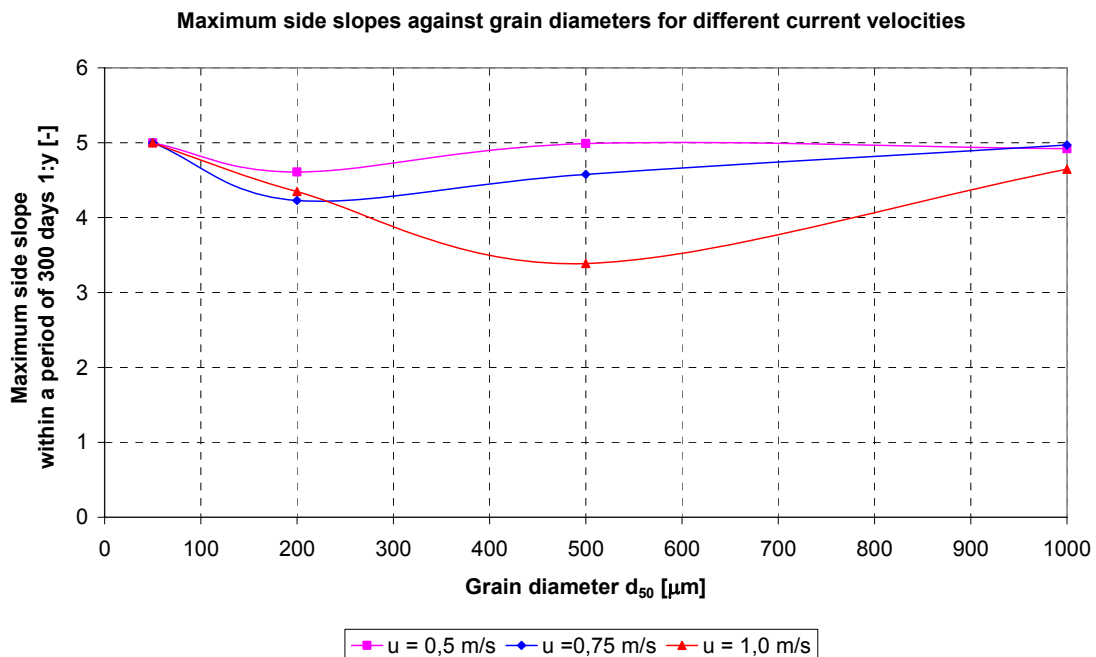


Figure 10-38: Maximum side slopes within a period of 300 days against grain sizes

10.3.4 Variation of significant wave height

Also the influence of the (constant) significant wave height has been examined for all four reference sediments. The wave height was varied as follows: $H_s = 0,5; 1,5; 2,5$ m. The morphological development in coarse sand proceeds very slowly, unless the wave height is increased to 2,5 m, see Figure 10-39. Instead of a small *slope steepening* as was the case in unidirectional flow, a little slope flattening occurs. However, slope development remains limited and shouldn't have to be taken into account in slope design.

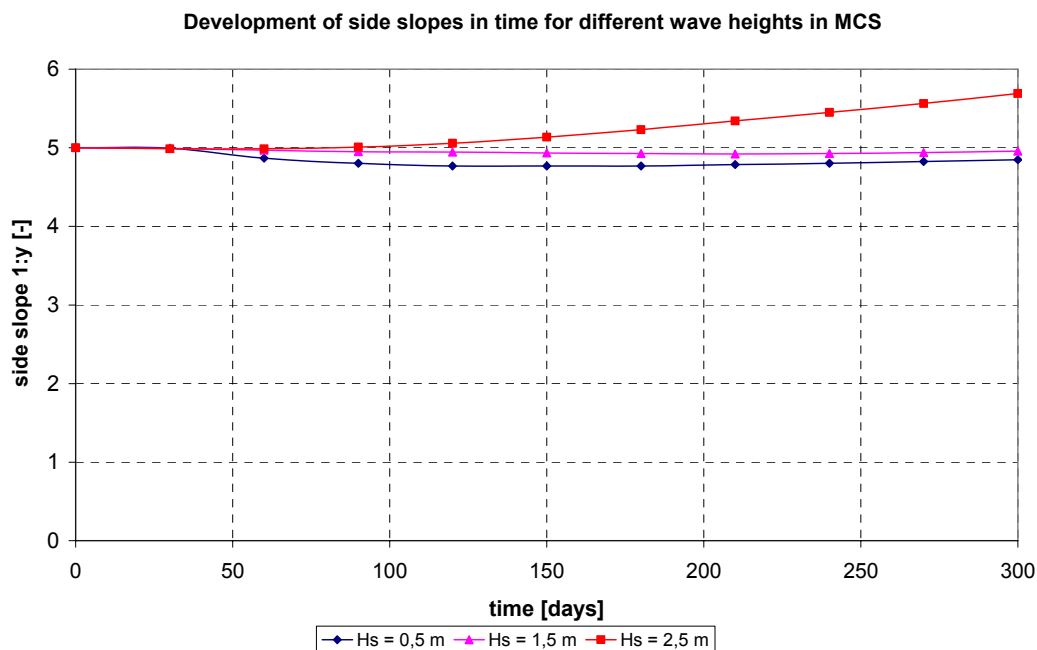


Figure 10-39: Slope development in tidal flow in MCS for different wave heights

Increasing the wave height to 2,5 m in MMS has a *steepening* effect (Figure 10-40). Strange enough, little waves show a small slope steepening, while medium waves create more gentle slopes, at least for this tidal current. In the previous paragraph it was already shown that larger tidal currents cause steeper slopes.

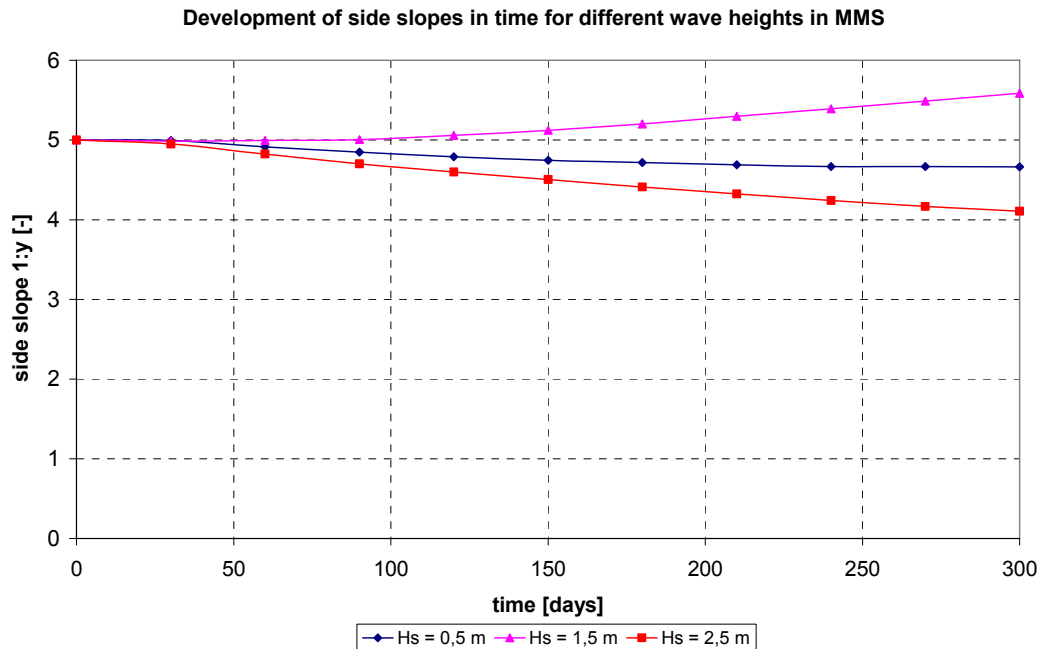


Figure 10-40: Slope development in tidal flow in MMS for different wave heights

In fine sands, the side slopes become steeper if waves become larger, although this effect remains small, observe Figure 10-41. Due to the increasing share of suspended transport with increasing wave height, rapid sedimentation occurs, resulting in flatter slopes.

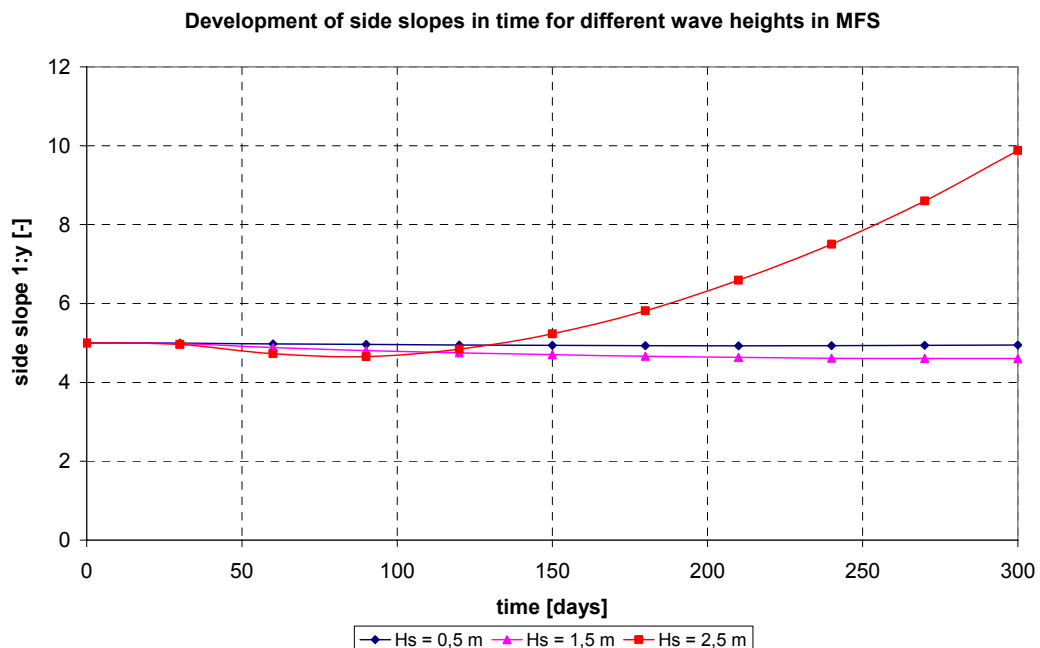


Figure 10-41: Slope development in tidal flow in MFS for different wave heights

In MSI for the first time autonomous slope development is observed, if the wave stirring is very small ($H_s = 0,5$ m), see Figure 10-42. In sediment like silt almost 100% of the sediment transport moves in suspension. Grains that are picked up by currents and waves immediately are mixed through the vertical.

Trapping efficiencies are small, even for this wide navigation channel. So sedimentation takes place over the entire channel and isn't confined to the side slopes. To maintain the initial side slopes, this sedimentation needs to be distributed evenly over the entire channel profile. However, the hydrodynamic forcing diminishes with increasing depth. Especially wave impact is reduced on the channel bottom. It is therefore not strange that particularly small waves ($H_s = 0,5\text{m}$) cause a more regular sedimentation pattern, because the influence of the water depth will be less. Also more gentle side slopes will result in a more regular sedimentation pattern, as was already concluded in Paragraph 10.3.2: the sedimentation on the more gentle side slopes (1:7) was far more than on the steeper side slopes (1:3).

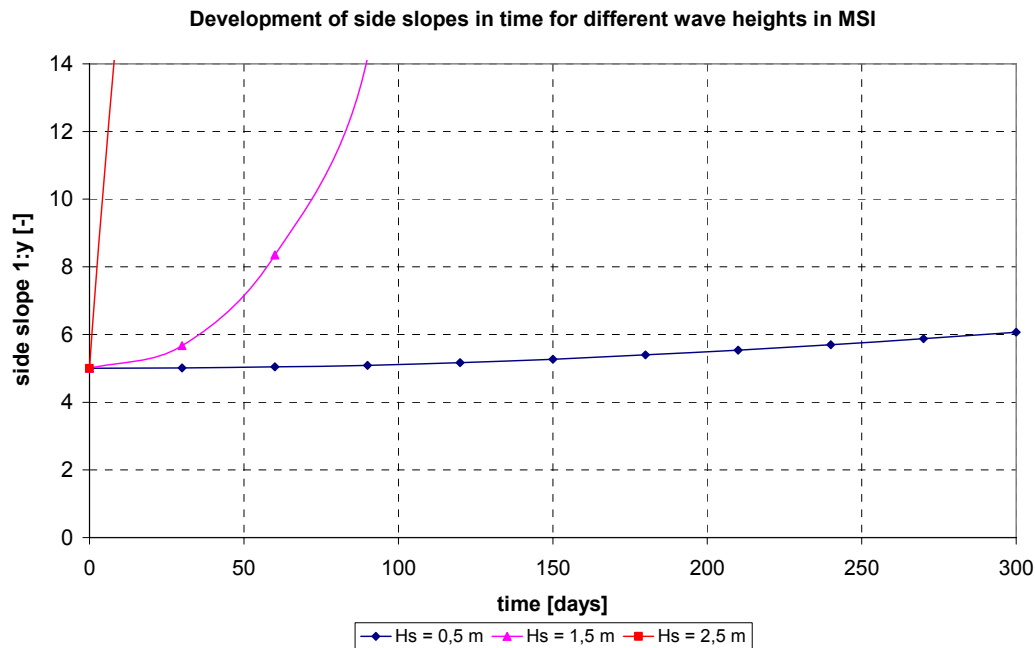


Figure 10-42: Slope development in tidal flow in MSI for different wave heights

So, channel development in MSI is most influenced by variation of the wave height. Large waves are able to almost completely silt up this navigation channel within a few days, while small waves cause a gradual siltation. If protection against waves isn't possible, the alignment of the channel itself seems the only appropriate measure to withstand storm waves. A small angle between current and channel axis generates the before mentioned self-cleansing effect. Sediment trapping reservoirs are only useful for sediments in the range of 100-200 μm ; smaller sediments remain too long in suspension due to a small particle fall velocity and therefore won't settle in such reservoirs.

Again summarizing all of the above simulations with different wave heights, the following conclusions can be drawn:

- The influence of varying wave height is less pronounced in tidal flow than in unidirectional flow. This can be explained by the fact that the flow is turning and *slope steepening* only proceeds when a side slope is acting as an upstream slope;
- The influence of varying wave height is less obvious than the influence of varying current discharge. This can be explained by the fact that an increase of the wave height not only increases wave stirring at the surrounding seabed, but also increases the 'depth of impact'. Bigger waves start reshaping the side slopes and disturb the gradual sedimentation process by the current;

The above explanation can be observed in Figure 10-43. Small waves ($H=0,5\text{m}$) only stir up sediment at the surrounding seabed, which is transported by the tidal current to the upstream slope. Medium waves also stir up sediment (in larger amounts), but have a deeper impact and interact with the side slopes at intermediate depths, resulting in somewhat flatter side slopes. With big waves the first described process is dominant.

The most important conclusion should therefore be: waves affect the morphological development of *the channel as a whole* to a large extent, but the consequences for the *side slopes* are not that obvious. Fine to medium sands are most likely to show some slope steepening, but due to the turning character of the flow, side slopes probably would not become steeper than 1:4.

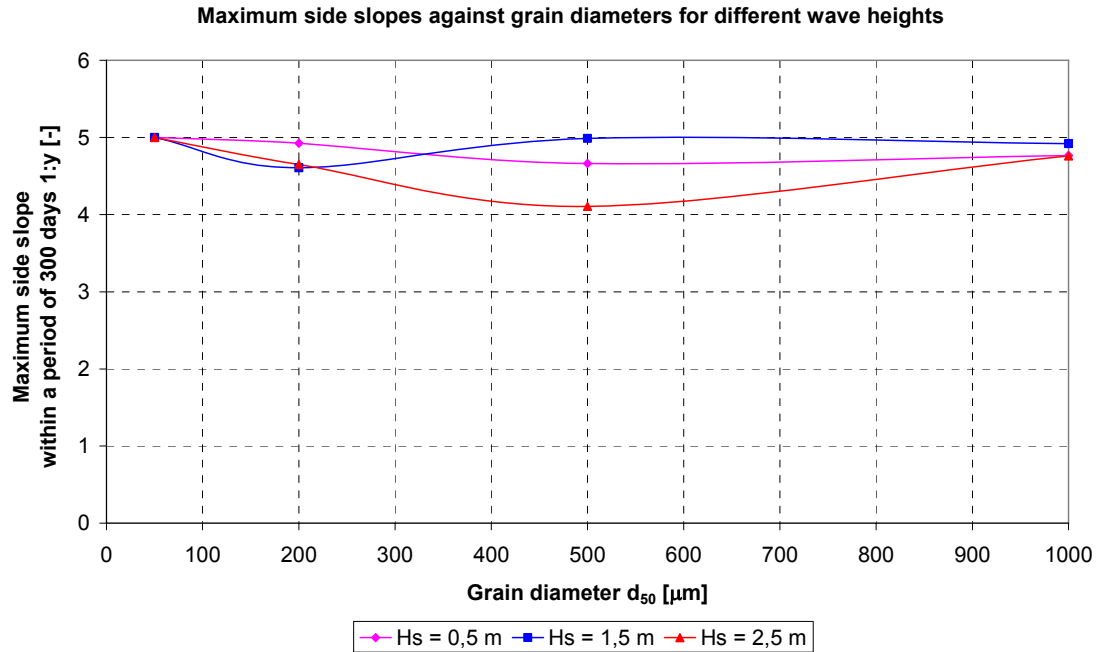


Figure 10-43: Maximum side slopes within a period of 300 days against grain sizes for different wave heights

10.4 Simulations with a real tidal and wave climate

10.4.1 Hydrodynamic input data

In order to resemble nature more and more, now some simulations with real (schematized) tidal flow and wave climate will be done. In fact any location can be chosen, as long as all hydrodynamic input data can be obtained at that location. These hydrodynamic input data consist of the vertical tidal elevation, the horizontal tidal velocity and the wave data over a complete year. The data are obtained from Noordwijk Meetpost (52°16'26" Northern Latitude, 04°17'46" Eastern Longitude). This measuring station is located about 10 km off the mainland coast in a water depth of MSL-18m, see Figure 10-44.

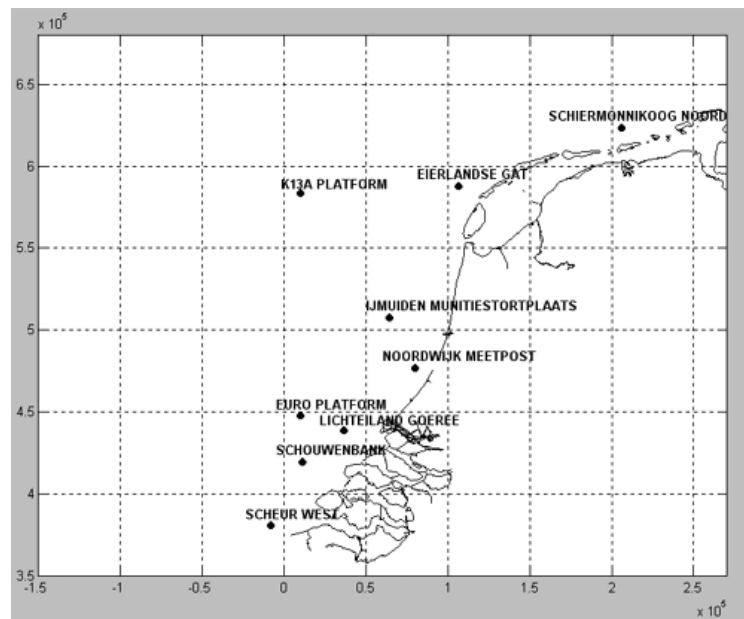


Figure 10-44: Measuring stations in the North Sea

The (vertical) tidal elevations of the year 2003 at Meetpost Noordwijk are obtained from www.getij.nl and plotted in Figure 10-45. In blue the highest water levels in a tidal cycle, in red the lowest water levels are plotted. The horizontal lines represent the year-average values.

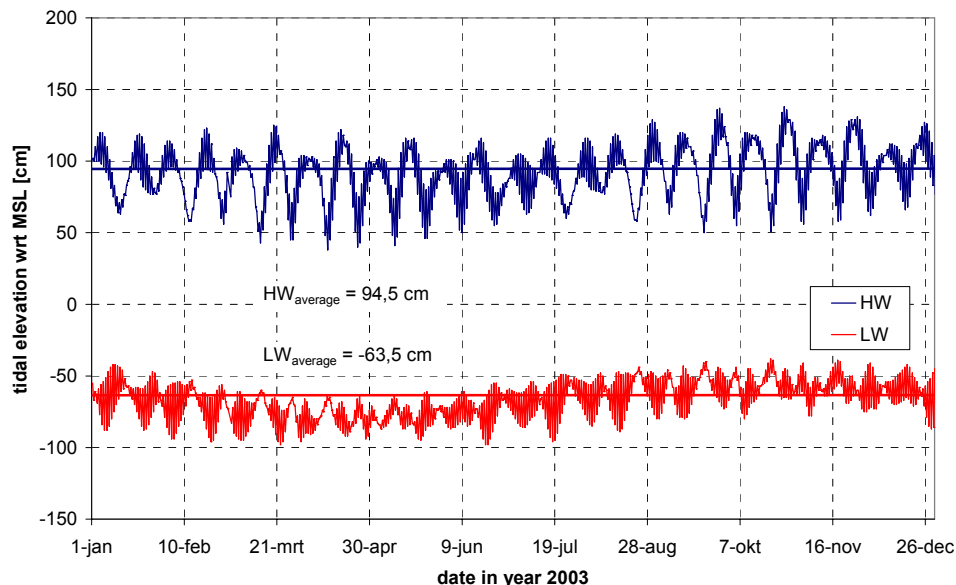


Figure 10-45: Tidal elevations at Meetpost Noordwijk in the year 2003

Because in SUTRENCH only one representative tidal cycle can be entered (and not a full year of tidal data), a tidal cycle has been selected with the average high and low water: December 6th from 12:03u, plotted as the 'blue' line in Figure 10-46. In this way the small oscillations throughout the year due to the interaction between moon and sun (like neap and spring tide) are neglected.

The asymmetrical tide (longer ebb period) is partially caused by the small distance to the amphidromic point (about 60km), where the influence of the moon is weakened and other higher order components become more significant.

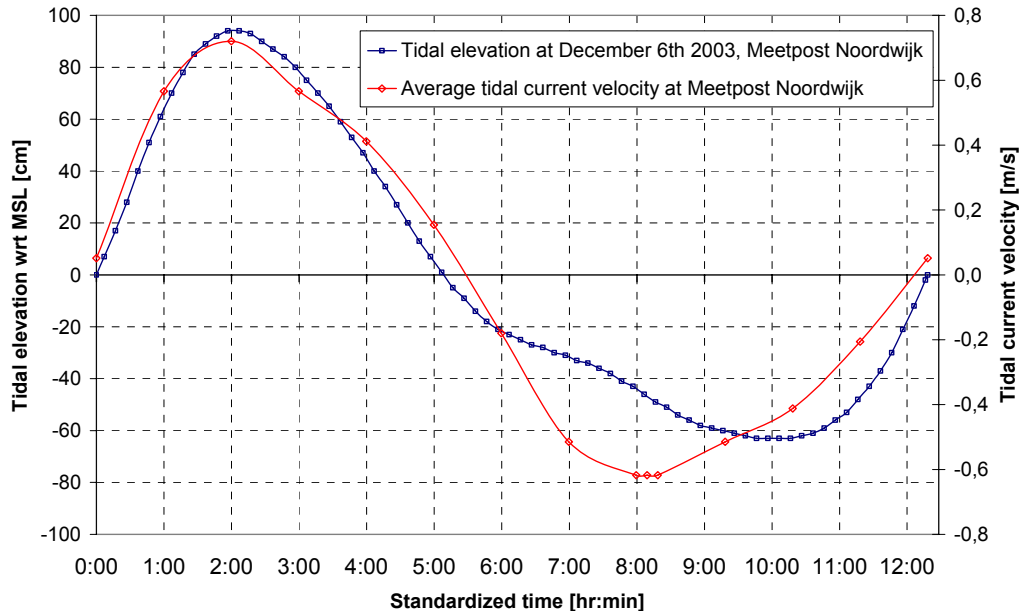


Figure 10-46: Representative tidal cycle and average current velocity against normalized time

The Nautilus Digital Current Atlas (Dutch: *Nautilus Digitale Stroomatlas* by RIKZ) supplies the horizontal tidal data (current velocities). The data are related to the time of high water (HW) and have to be shifted in time to be compatible with the vertical tidal data, see the 'red' line in Figure 10-46. It appears that horizontal and vertical tide are almost in phase, especially during flood.

The representative tidal cycle, based on the above vertical and horizontal tidal data, has to be further schematized into discrete periods, see Figure 10-47.

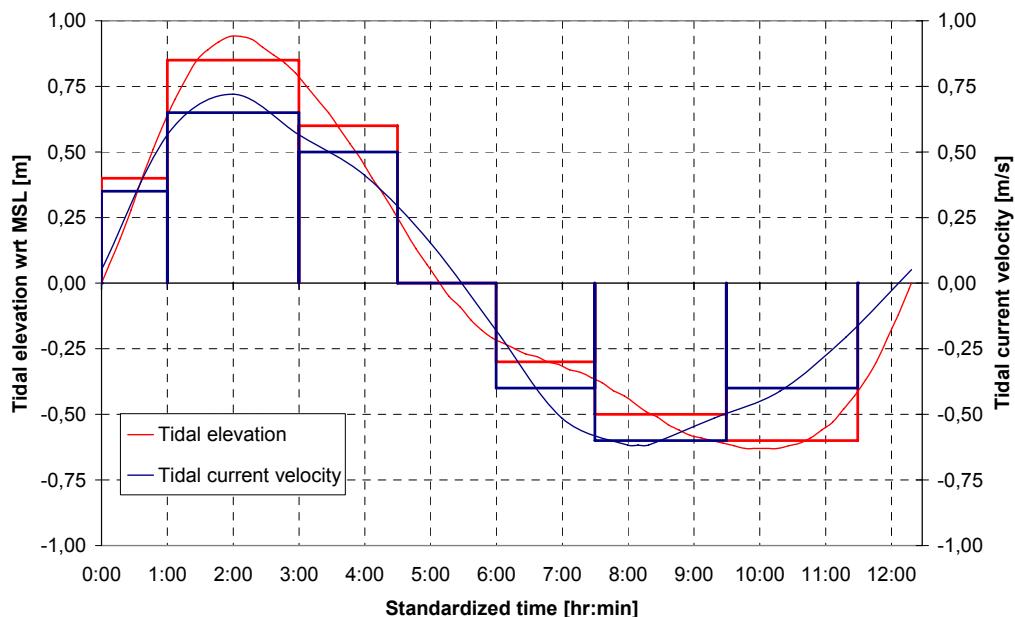


Figure 10-47: Schematizations into 6 discrete periods for tidal elevation and current velocity

In this way a full (representative) tidal cycle can be represented by 6 discrete periods with a certain water level and discharge and 2 discrete 'transition' periods without any tidal elevation or current discharge between every flood and ebb period to ensure that the simulation time and actual time are equal, see Table 10-7.

Table 10-7: Schematization of representative tidal cycle

		water level	discharge	time step
		h_0 [m]	Q [m ³ /ms]	dt [s]
flood	1	10,40	3,64	3600
	2	10,85	7,05	7200
	3	10,60	5,30	5400
	transition	10,00	0,00	5400
ebb	4	9,70	-3,88	5400
	5	9,50	-5,70	7200
	6	9,40	-3,76	7200
	transition	10,00	0,00	2940
total tidal cycle			12hr19min =	44340

The wave data at Meetpost Noordwijk are obtained from www.golflklimaat.nl for a period of ten years (1992-2001) and plotted in Figure 10-48. As wave height ' $H_{1/3}$ ' has been used, because $H_s \approx H_{1/3}$. From now on, we will only speak of significant wave height, although the real wave data were obtained as ' $H_{1/3}$ '.

Because SUTRENCH cannot process large wave input files (restricted to 20 periods with constant (significant) wave height), a schematized set of wave input data has to be constructed. Although it is inevitable that some information is lost during this process, it is very important that in some manner the same amount of wave energy is put into the morphological model. Therefore the following approach has been adopted.

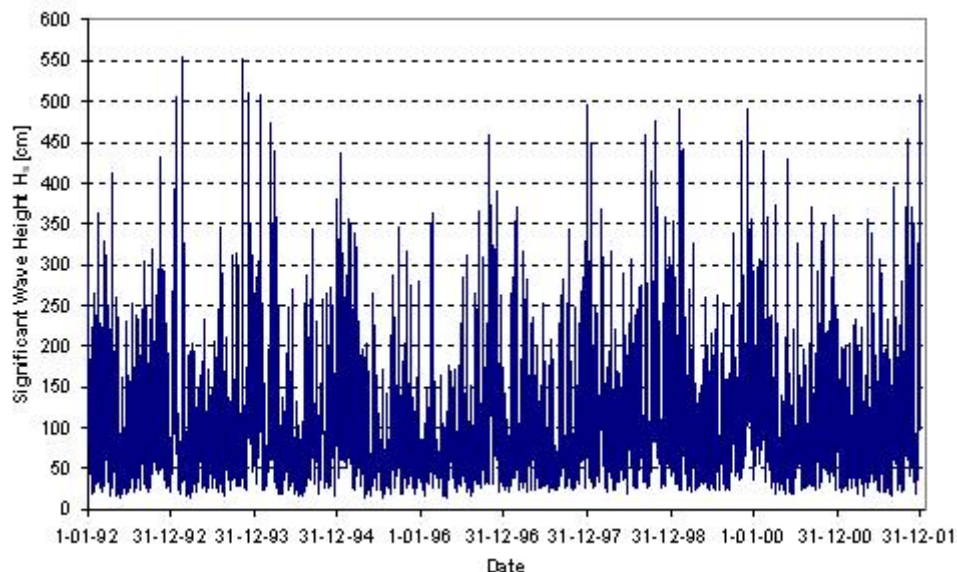


Figure 10-48: Significant wave heights in a period of 10 year (1992-2001) at Meetpost Noordwijk

First all waves of the wave data set over 10 years have been categorized into smaller wave bins. For each bin the significant wave period and the probability have been determined, see Figure 10-49. Now, it is questionable which value for the wave height has to be used for each wave bin. One could use the middle of the bin ($H = 0,25; 0,75; 1,25; \dots 5,75m$), but due to the distribution of the waves inside each bin towards the peak probability somewhere inside the second bin, the wave height would be slightly overestimated for most wave bins. This error of course can be reduced by decreasing the bin width, but then the resulting input becomes too extensive for SUTRENCH. Another option is to calculate the average wave height inside each bin for the 10-year period.

This method neglects the fact that increasingly large waves have a more than linearly increased impact. Therefore the 'average square wave height ($\overline{H_i^2}$)' has been calculated for each wave bin 'i'. This approach ensures *conservation of wave energy* inside each wave bin.

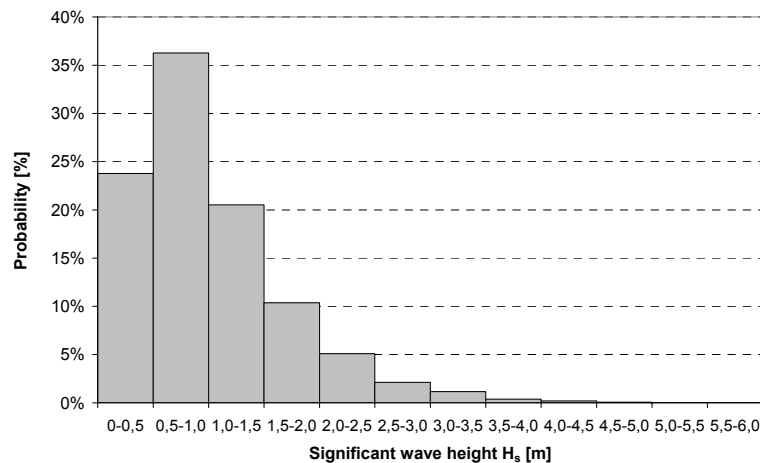


Figure 10-49: Probability distribution of wave bins

Together with the probability of each wave bin a number of 12 'storms' with different durations have been constructed, see Table 10-8. If this combination of storms is used in morphological simulations, approximately an amount of wave energy is involved that is equal to a 10-year-averaged wave climate at Noordwijk Meetpost. Please note that the wave heights were determined as the average of the square wave heights, while the wave periods were averaged in the usual way. This method is quite in line with the approach formula, deduced in Paragraph 10.1.2, based on the powers of most important input parameters. There a power of '1,7' for the wave height and '0,5' for the wave period was found.

Table 10-8: Schematization of wave height, wave period and 'storm duration'

Wave bin	H_s	T	Duration
cm	m	s	hr
0 - 50	0,36	4,6	2084,1
50 - 100	0,71	5,0	3176,8
100 - 150	1,17	5,5	1798,9
150 - 200	1,62	6,0	908,2
200 - 250	2,11	6,6	446,4
250 - 300	2,58	7,2	186,5
300 - 350	3,09	7,7	100,1
350 - 400	3,56	8,2	32,4
400 - 450	4,08	8,7	17,4
450 - 500	4,70	9,3	6,3
500 - 550	5,13	9,5	2,4
550 - 600	5,51	10,8	0,6

Now, the only remaining question is the order of the storms throughout the year. One of Postma's conclusions [1987], see Paragraph 2.3, was that the morphological development to an 'equilibrium profile' of a trench was strongly dependent on the weather conditions; the sooner storm conditions occurred, the faster the side slopes developed towards this 'equilibrium profile'. It seems evident that it makes a big difference whether a freshly dredged channel profile can slowly adapt to deteriorating weather conditions or when the same initial profile is instantaneously swept by a storm.

- But is morphological development really influenced by the wave distribution throughout the year?

When no extensive wave data are available or one just wants to make a quick computation, it will be valuable if one can apply just one wave height instead of a (schematized) full-year wave climate.

- But which wave height has to be chosen in a morphological simulation to be representative for a full-year wave climate?

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

Based on Soulsby's statement a wave height of about 1,5 to 2,0 m would be expected to be representative for the entire year.

To investigate both underlined questions the following wave distributions are proposed:

- I: an ever increasing wave climate, lower boundary of morphological development;
- II: an ever decreasing wave climate, upper boundary of morphological development;
- III: first increasing and then decreasing wave climate, most realistic case;
- IV: constant year-averaged wave height;
- V: constant year-averaged square wave height;
- VI: constant wave height that will be fitted to case III.

Cases I and II give insight in the maximum variation in morphological development due to the wave distribution throughout the year. Case II can be considered as a maximum for bed level changes, because the largest waves occur when the profile is still sharp, while in case I the storm waves occur when the channel profile is already more gentle. Case III is considered to be most realistic. Often dredging works are executed under calm weather conditions (summer). After about a half year the storms have intensified (winter) and then the wind will die down again. Cases IV, V and VI should demonstrate the validity of calculations with a constant wave height and should yield a value for this particular wave height that approximates case III best. The first three cases are plotted in Figure 10-50.

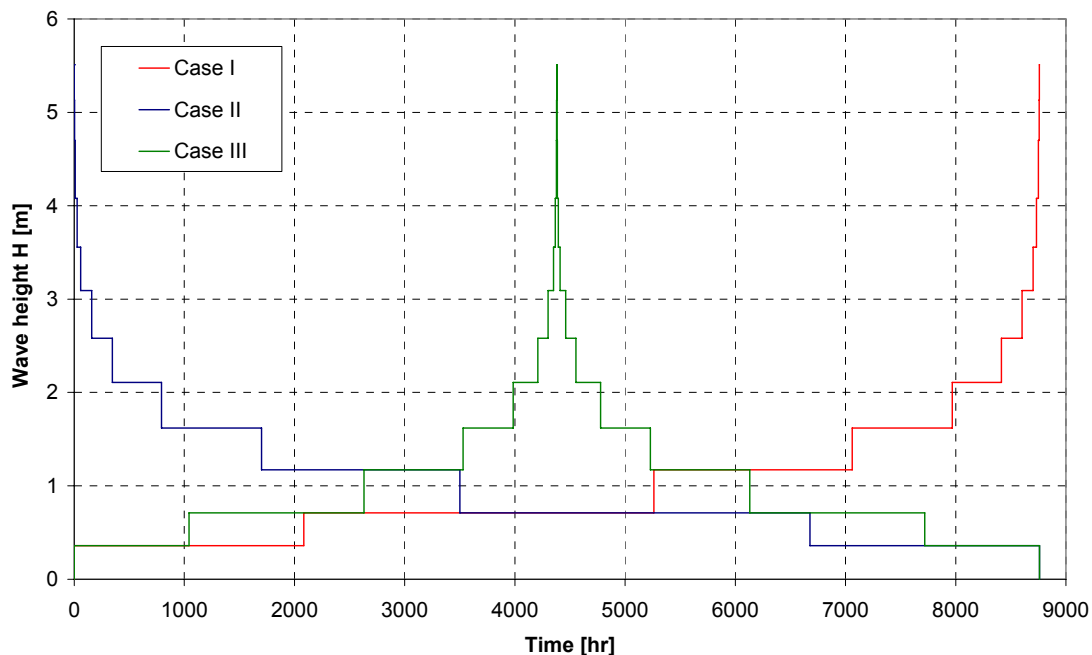


Figure 10-50: Schematization of first three cases of wave distribution

One should keep in mind that morphological development remains a probabilistic process. The applied schematization of the wave climate, however averaged over 10 years, is still just a random indication: there is a large variation in wave height between different years. For instance the averaged significant wave height in 1997 was 91 cm, while in 1998 this was 117 cm.

10.4.2 Different wave distributions

Before the final simulations were done, first the well-known problem of numerical diffusion was examined. Because the tidal flow was reversing and relatively mild, the need for numerical smoothening was expected to be limited. Indeed, it appeared that even with a very small value for coefficient ' α_s ' the simulations remained stable and showed natural behaviour. All simulations of these paragraph were executed with $\alpha_s = 0,001[-]$.

The first question of the previous paragraph that needs to be examined was: "is the morphological development significantly influenced by the wave distribution throughout the year?" Therefore the morphological development within 1 year is presented for cases I, II and III in Figure 10-51. It can be observed that after one full year of inserted wave energy, the difference between the cases is hardly noticeable and certainly much smaller than the uncertainty in the wave distribution, the sediment transport, the hydrodynamics et cetera. Furthermore it appears that both slopes show some *upstream* and *downstream slope behaviour*; the top of the slopes are flattened and rounded due to erosion (downstream slope), while the middle and lower part remained steep and shifted somewhat towards the centre of the channel (upstream slope). The channel width is only slightly reduced, while the depth reduction on the channel bottom does not exceed 1m.

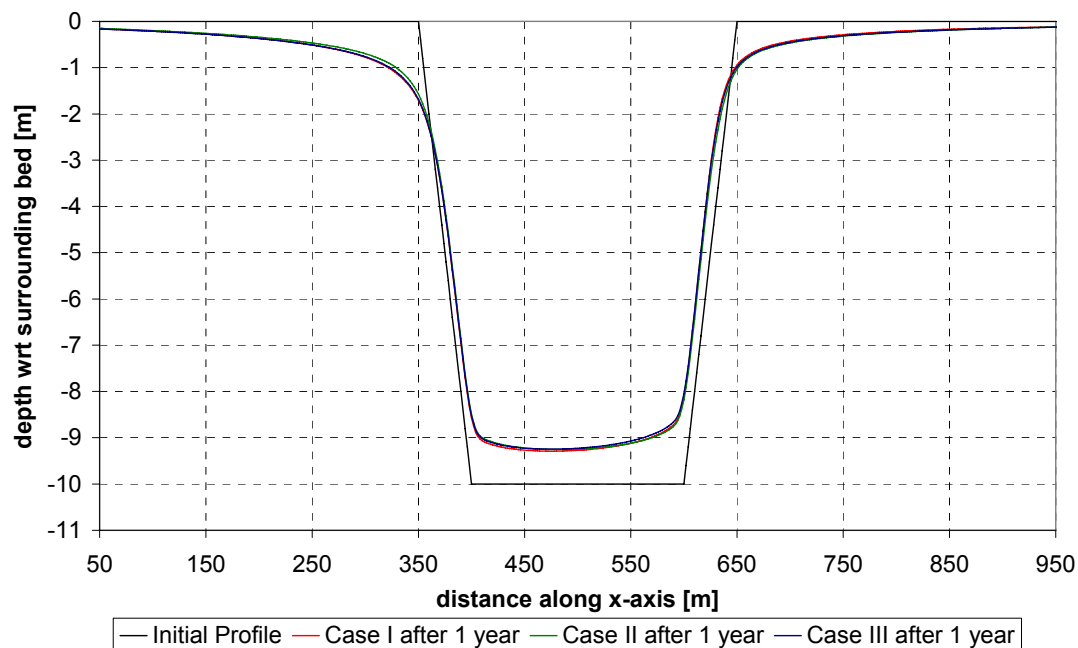


Figure 10-51: Morphological development within 1 year for cases I, II and III

Also it can be seen that the morphological development is slightly asymmetric. Obviously the ebb current is slightly dominant over the flood current, although this is also dependent on the tidal period in which the largest storm waves occur. Case II with the decreasing wave height has its peak storm during flood (at $t = 0$), which can be observed from the better developed left slope (acting as the upstream slope during flood). For case I holds the opposite: storm occurs during ebb (at $t = 365$ days) and the right slope has accreted more compared to case II. The described differences are however very hard to notice on this scale.

So, now let us look at the slopes in more detail. In Figure 10-52 the slopes of the initial seabed and the seabed after 1 year are plotted for the three cases. The rectangular bumps represent the initial situation; the more slender peaks represent the situation after 1 year. The most important conclusions are:

- maximum slope steepness does not change much and remains in the order of 1:5;
- the steepest parts occur over a smaller height which is favourable for stability, although the density of the accreted sediment at both slopes is very hard to predict;
- both side slopes can develop autonomously, until the slopes have shifted so close to each other that the toes of the slope start to intersect. If the side slopes become steeper and flow separation occurs, the location of the reattachment point of the flow determines whether side slopes can develop autonomously.

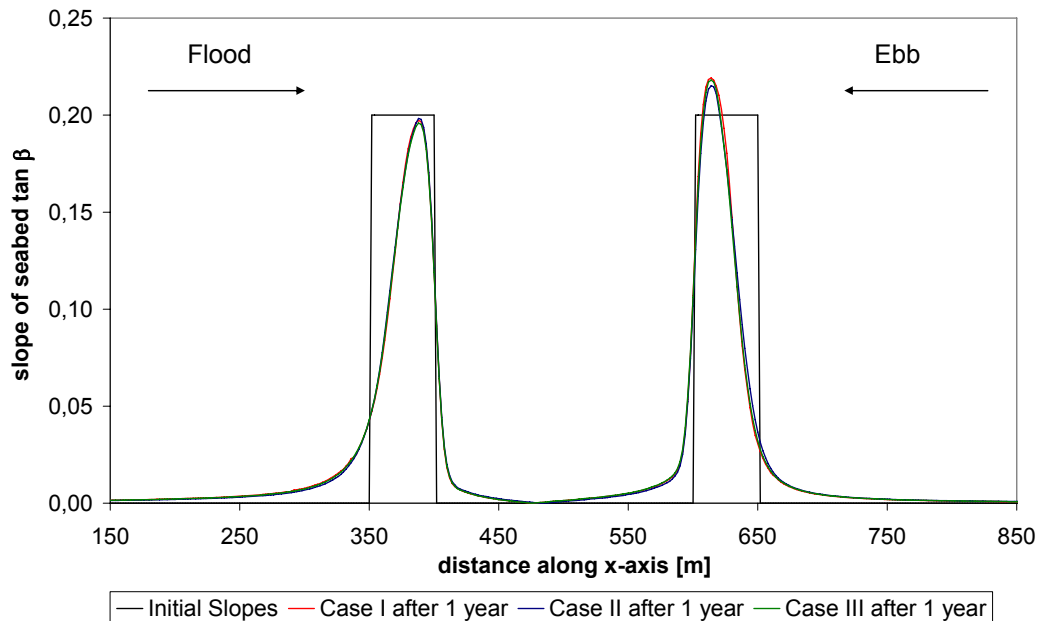


Figure 10-52: Slope of seabed ($\tan \beta$) along the x-axis

So, it is proved graphically that the differences in morphological development due to different wave distributions throughout the year remain limited and the small deviations could be explained. But what about Postma's statement [1987] about the time that the side slopes needed to reach their (dynamic) equilibrium and how this time was dependent on the time that elapses till the first storm? To explore this question slope development throughout the year is presented in Figure 10-53. This graph looks rather chaotic at first sight, a maze of apparently arbitrary lines, but they can be explained.

The largest slope steepening or flattening occurs during the *stormy period*. The steep parts in Figure 10-53 correspond to the cases with different wave distribution. For instance, case III has the stormy period halfway the year.

The slope development is most pronounced, when a slope is acting as an *upstream slope*. This could also be concluded from the simulations with simple tidal flow; both side slopes remained rather steep and the downstream behaviour of slope flattening was hardly noticeable. So left slopes develop mainly during flood, while right slopes develop mainly during ebb. Because the water level is higher during flood, only the biggest waves are felt at the seabed. During ebb, the water level is lower and the fast slope development is caused by a larger number of waves. This can be observed in Figure 10-53: the quick changes of the left (flood) slopes occur in a period of 30 days, while these changes take about 60 days for the right (ebb) slopes. Comparing these periods with the wave distribution shows that the flood slopes are mainly developed by wave heights of 2,11 m and larger, while ebb slopes develop in a wave climate of waves with a minimum height of 1,62 m. These seemingly exact values only follow from the applied classification in wave bins, as was explained above and are therefore only indicative.

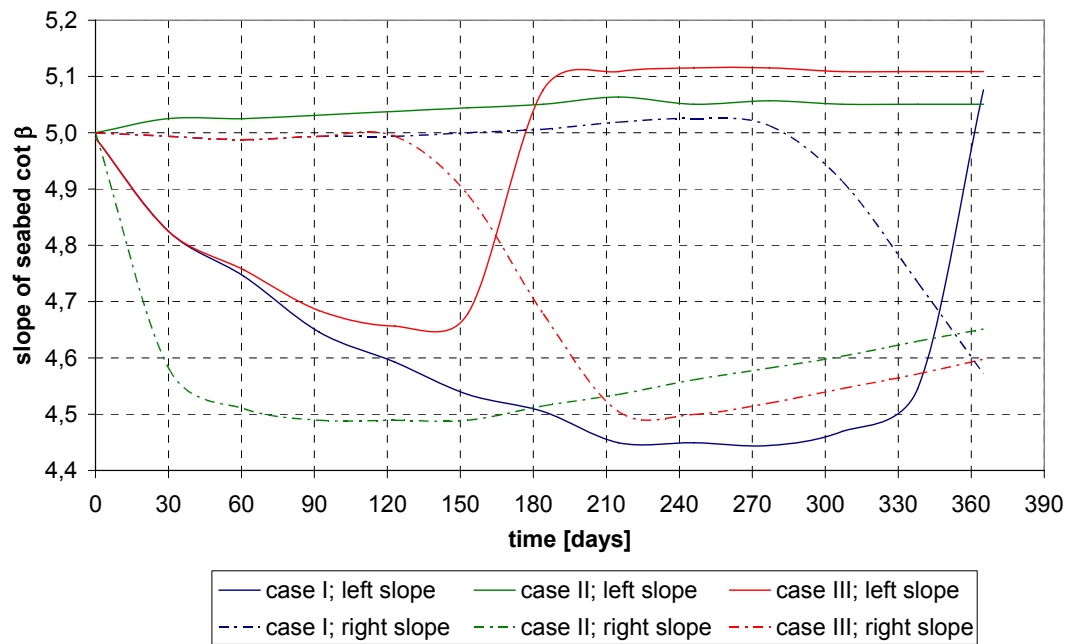


Figure 10-53: Slope development of three cases throughout the year

Both the flood slope and the ebb slope develop towards their own dynamic equilibrium value. For the flood slope this dynamic equilibrium slope (DES) is about 1:5,1, while the ebb slope steepens somewhat to 1:4,6.

Remarkable is the fact that flood slopes steepen under smaller waves ($H < 2,11\text{m}$), whereas ebb slopes tend to remain approximately 1:5 for smaller waves ($H < 1,62\text{m}$). Then for the larger waves flood slopes flatten and ebb slopes steepen. This effect is probably caused by the combination of wave height and water depth. During flood (large water depth) the smaller waves do not affect the slopes, but only provide a considerable amount of sediment, while during ebb these waves are involved in the slope shaping process.

So, it seems that there is a certain *wave height of influence*, but a sound formulation of this wave height is difficult: the water level is constantly changing, the wave period is not constant, the wavelength is adapting at the side slopes et cetera. A rough criterion for the wave height of influence can be obtained from the maximum orbital velocity at half-depth $((h_0 + h_1)/2)$. The minimum wave heights that influence slope development induce orbital velocities at the seabed of 0,32 - 0,44 m/s at a water depth of 15 m. This results in the following design criterion:

$$H_{\text{influential}} \geq 0,13T \sinh\left(\frac{1}{2}k(h_0 + h_1)\right) \quad \text{Eq. 10-5}$$

Of course, this is a very rough relation, not based on many simulations, but it explains the observed behaviour. The location of 'half-depth' is based on the idea that the steepest part of the side slope usually is situated at half-depth.

The observed values of the steepness of both side slopes should only be interpreted in a qualitative way; the quantitative values are strongly dependent on the combination of wave height, water depth and (as will be shown in the next paragraph) the wave distribution.

10.4.3 Towards a representative wave height

The second question that rose in Paragraph 10.4.1 concerned the idea whether a full-year wave climate could be represented by a single representative wave height. It was suggested to run simulations with the average wave height ($H=1,02\text{m}$), the average of the square wave height ($H=1,23\text{m}$) and finally to search for a representative wave height by trial and error. This last simulation will be compared with case III which was considered to be the most realistic case (construction works during summer, storms during winter).

What strikes one first, when observing Figure 10-54, is the fact that both the simulation with the average wave height (case IV) and the average square wave height (case V) result in too little morphological development.

After the conclusion of the previous paragraph that the major morphological development occurred for waves larger than 1,62 m (ebb slopes), this was to be expected.

In case VI the wave height was varied until an acceptable graphical resemblance to case III was obtained. This constant wave height is 1,8 m. This wave height falls in the wave bin of 1,5-2,0 m, which has an occurrence of 10%. Wave bins representing larger wave heights together account for 9%. This means that the representative significant wave height of 1,8 m is not exceeded in 81% to 91% of the time.

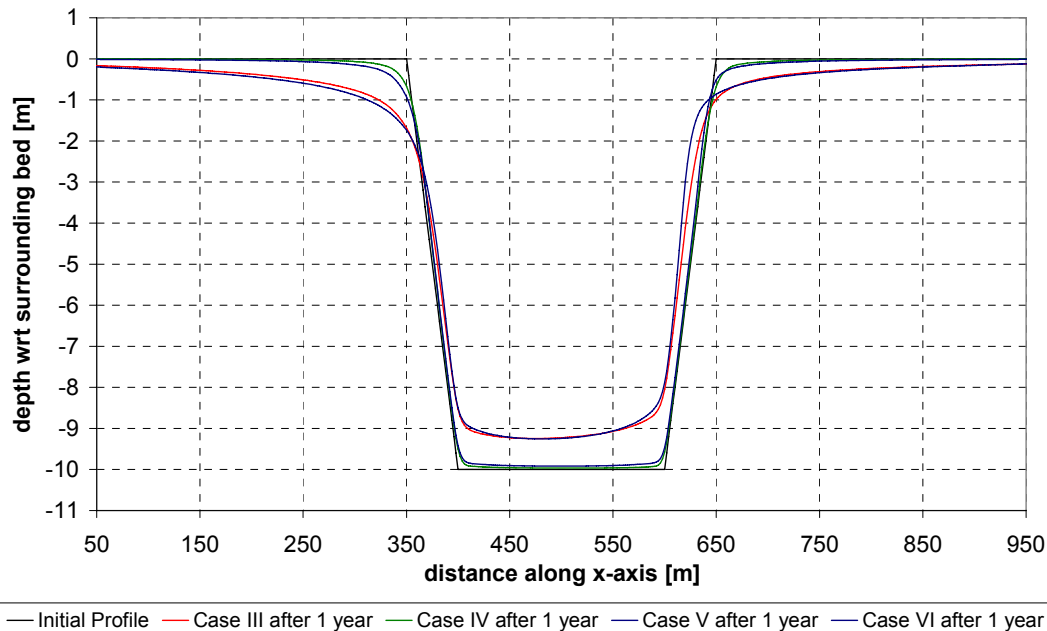


Figure 10-54: Morphological development within 1 year for cases III, IV, V and VI

To get a better estimate of the exceedance in time, we have to return to the rough data of the wave heights during 10 year at MPN. It turns out that in 87,4% of the time, the significant wave height (in fact $H_{1/3}$) does not exceed 1,8m. So, indeed a rather large wave height has to be applied to represent a full-year wave climate. On the other hand, this wave height is still far smaller than the design wave height for dykes or structures.

The next question that arises is whether morphological simulations with a representative wave height yield similar results for slope development. Simulating with one wave height instead of a complete wave distribution may be practical for the prediction of backfilling, but inevitably some information is lost during this process. How significant this loss in geometrical accuracy is, can be observed in Figure 10-55. The location of the side slopes is quite similar for cases III and VI, but case VI yields steeper side slopes, especially the ebb slope (1:3,4).

In earlier simulations was already found that side slopes can steepen significantly for a large wave height, depending on the sediment size. Now, it turns out that a varying wave height suppresses this behaviour. This can be explained by the fact that the very influential, large waves reshape the side slopes due to the deep impact of these waves.

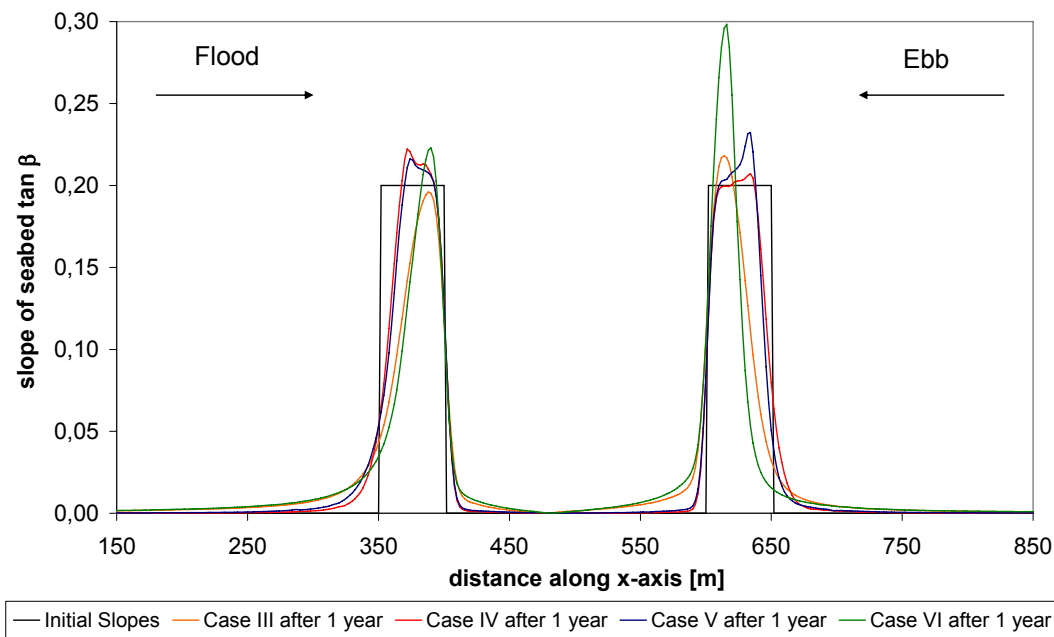


Figure 10-55: Slope of seabed ($\tan \beta$) along the x-axis

Finally, slope development in time for the different cases will be studied, see Figure 10-56. Cases IV, V and VI show familiar slope behaviour: slopes steepen in time and only the right slope of case IV then flattens again. For larger wave heights the steepest slope will not be reached within a year. The ebb slopes become somewhat steeper than the flood slopes. Case IV (smallest wave height) only shows steeper ebb slopes in the first 90 days.

Anyhow, the cases with a uniform wave height (IV, V, VI) show a gradual morphological development, which can certainly not be said for case III. Both the flood and ebb slope change rather suddenly during respectively 30 and 60 days, i.e. 8 and 16% of the time. As a result the side slopes of cases III and VI (representative wave height) after one year are rather different. Geotechnical instabilities are for this situation not likely to occur in reality (full-year wave distribution; case III), whereas the simplified procedure of simulating morphological development (representative wave height; case VI) predicts a significant slope steepening of especially the ebb slope.

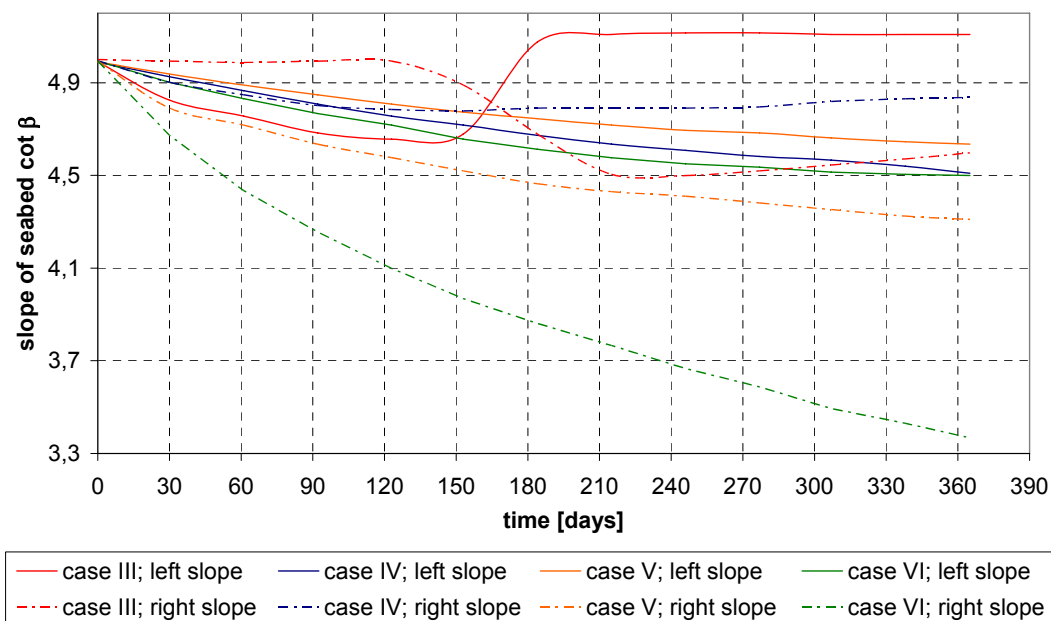


Figure 10-56: Slope development of cases III, IV, V and VI throughout the year

From this paragraph some important conclusions can be drawn:

- SUTRENCH can only handle a schematized hydrodynamic climate. Both waves and tidal current have to be discretized. Although some information is lost during this process, there is some confidence that the input data are a reliable reflection of the field conditions.
- If one is particularly interested in the morphological development *over* one or more years, the sequence of the wave heights throughout the year (case I, II and III) is not that important, as long as the inserted wave energy is conserved and the wave distribution function is guaranteed. In other words, it does not make that much difference if storm waves occur early or late in a year, although it was expected that large waves would have a larger impact on a newly dredged profile.
- However, if one is interested in the morphological development *throughout* the year, it is extremely important at what time during the year storm waves occur. The largest waves that only occur about 10-20% of the time govern slope development. This 'time of influence' is dependent on the combination of wave height and water depth. During ebb more waves are felt at the side slopes, so ebb slopes will develop during a longer period of time.
So the slope development can well be approximated if one only considers the largest waves (satisfying the wave distribution). In fact a full-year simulation is then replaced by a simulation over a shorter period of time with only the largest waves. Equation 10-5 gives an idea for a minimum value for the wave height. It is then easy to determine which part of the wave distribution in time has to be applied. One should note that in this approach the backfilling is strongly underestimated, because the smaller waves, that do transport some sediment, are neglected. Besides, this approach is only permitted if one knows for sure that both slopes can develop independently throughout the 'real' simulated time.
- The application of a uniform wave height, that is representative for a full-year wave climate, yields good results, if one is particularly interested in the backfilling within the channel boundaries, but less interested in the exact locations of the sedimented material. A wave height based on significant wave heights that are exceeded only 87,6% of the time ($H=1,8$ m in this case) showed best graphical resemblance.

So, hopefully the difference in determining total backfilling and slope steepness is made clear.

Backfilling: *full-year* simulation with *uniform, representative* wave height

Slope steepness: simulation over only the *influential period* of the *wave distribution*

10.5 Qualitative comparison of results with full 3D-model

Although it wasn't possible to run a full 3D-model that solves the complete Navier-Stokes equations for flow, such as was described in Paragraph 6.1.2, a qualitative comparison can be made between both results to gain insight in the most spectacular differences and the limits in between the SUTRENCH-results of this thesis are valid. First, the results of the comparison of different sediment transport models, as was executed by Jensen et al. [1999], are presented, see Figure 10-57. The water depth and the sediment transport are made dimensionless; the side slope is $\pi/6 \approx 1:1.9$, which is rather steep. The most clear distinction between their sophisticated 3D-model and the other models is the (total) sediment transport at the upstream slope. The relative sediment transport first increases at the top of the upstream slope and then approximates the quasi-3D model. When one only considers bed-load transport, it can be observed that the bed-load transport shows a rapid decay on the upstream slope. This effect becomes larger for steeper slopes, because the flow loses contact with the seabed. The model of Mayor-Mora is based on a gradual decay of the sediment transport and only suitable for calculations of total sedimentation inside a trench or channel.

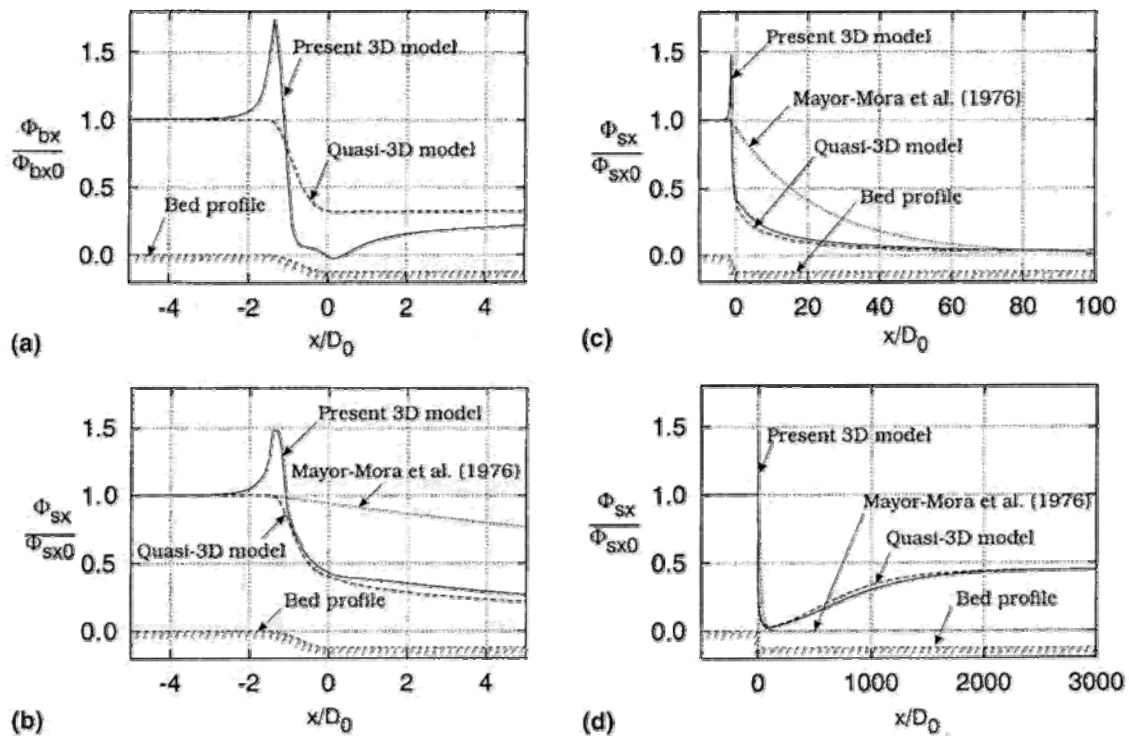


Figure 10-57: Results of different sediment transport models [Jensen et al., 1999]; (a) relative bed-load transport, (b) relative total sediment transport from $-5x/D_0$ to $5x/D_0$, (c) relative total sediment transport from $-10x/D_0$ to $100x/D_0$, (d) relative total sediment transport from $-500x/D_0$ to $3000x/D_0$

The differences between the sophisticated 3D-model and the quasi 3D-model become smaller when the side slopes become flatter. It should be noted that these models do not include the effect of waves.

The appearing inconsistent increase of sediment transport at the channel bottom is caused by the incoming flow angle of 60° . As was explained in Paragraph 6.1.2, the slow longitudinal acceleration of the refracted flow increases sediment transport.

In Figure 10-58 the dimensionless transport of four reference soils is presented. Whereas the relative transport in MSI is only reduced by 15% in the navigation channel, the sediment transport in MMS and MCS is reduced to zero, when the toe of the slope is reached (at $x/h_0=15$). This of course is caused by the very low content of suspended transport and the large particle fall velocity. The reduction of sediment content of the flow when the seabed consists of MFS is about 85% on the upstream slope.

The suspended transport further reduces over the channel bottom; at the toe of the downstream slope only a few percent of the equilibrium sediment transport at the surrounding bed is left.

Please note that all sediment transport rates are relative to their own equilibrium value: transport in MSI is a factor 320 larger than transport in MCS.

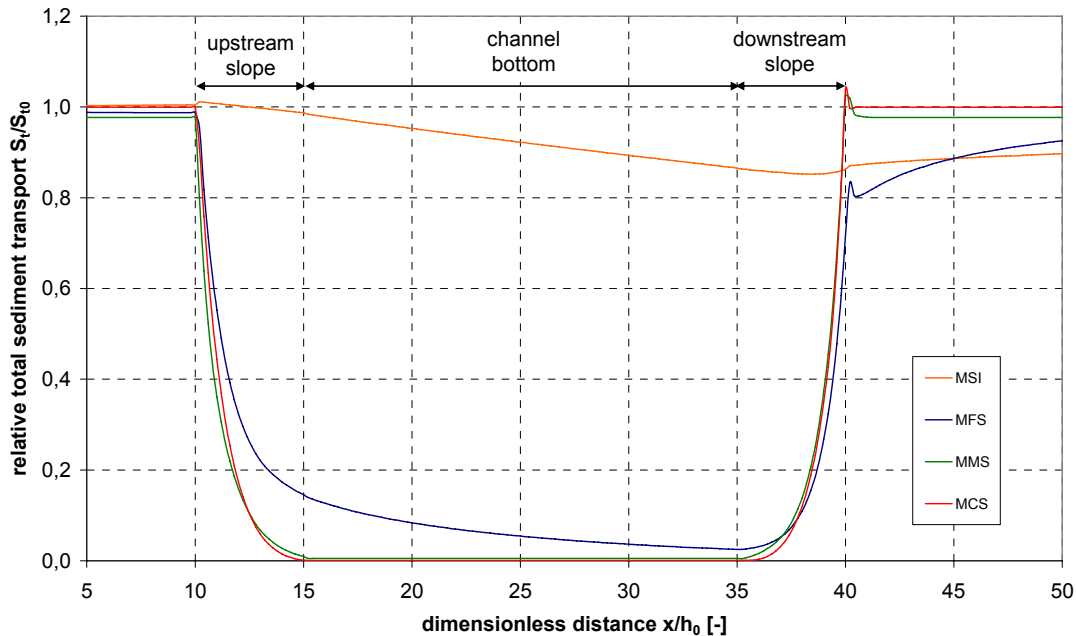


Figure 10-58: Relative total sediment transport for MSI, MFS, MMS and MCS

When the gradient of sediment transport is determined, it becomes clear that large sedimentation occurs at the top of the upstream slopes at $t=0$ days. This is partially caused by the sharp upper corners of the side slope. When the channel geometry changed into a more gentle profile after 90 days, the transport gradient is more gradual. Also can be seen that especially in MFS a downstream shift of the upstream slope has occurred. The downstream slope has flattened and shifted more, which can be concluded from the 'wide-spread' and flat gradient.

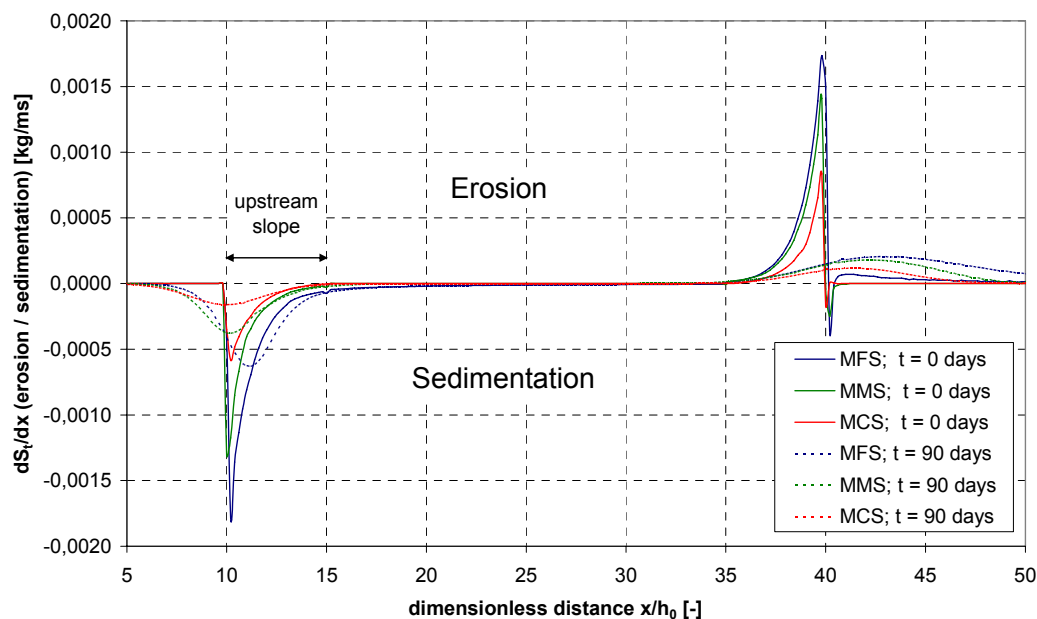


Figure 10-59: Sedimentation/erosion pattern inside channel

Summarizing can be concluded that the shape of the graphs of sediment transport primarily deviate at the top of the upstream slope. As long as side slopes become not too steep (steeper than 1:5), the errors remain limited.

11 Improvement of trench and channel design

11.1 Considerations on soil mechanics and morphology

As was mentioned earlier, channel geometries could well be adapted to expected morphological changes and possible instability problems. With the knowledge gained in the previous chapters one can easily cope with the relevant instability problems or backfilling behaviour. In this thesis the focus will be on the optimization of the side slopes. The most elegant slope design creates a balance between increasing slope stability and reducing backfilling volumes. Advantageous side-effects can be a reduction of the capital dredging costs (for a great deal dependent on dredging volumes) or a reduction of the channel upper width in case of restrictions in space. A disadvantageous side-effect can be the more difficult dredging method. The ability to control the breaching mechanism, when box-cutting (see Paragraph 2.3), determines to a large extent the applicability of variation in slope steepness. Besides, such measures are only profitable if the slope height is considerable and the dredging is performed in multiple layers.

When coping with backfilling problems, the most common solution is overdredging. It is very important to distinguish between two definitions. Some consider overdredging as a tool to deal with uncertainties inherent in channel dredging operations, but in this thesis by overdredging is meant "an effective strategy to increase side slope stability or to reduce dredging frequency and total maintenance costs". Both types of overdredging can be applied simultaneously. Also, overdredging does not necessarily have to be performed at the channel bottom. Dredging of flatter side slopes than necessary is in fact also a form of overdredging. Sometimes dredging of pocket holes in the side slopes at intermediate depths is considered. This measure can be extremely efficient if large travelling sand waves occur.

The appropriate method to reduce backfilling is strongly dependent on hydrodynamic conditions and sediment properties, of which grain size and particle fall velocity are most important. Sediment sizes larger than 500 μm turned out to stay very close to the side slopes and the morphological development is characterized by slope migration, while hardly any slope flattening occurs. The channel bottom remains unaffected, so no significant reduction in depth takes place. Finer sediments ($100 \mu\text{m} \leq d_{50} \leq 500 \mu\text{m}$) are more and more transported in suspension, so sediments spread over a larger width. Often a small depth reduction is observed as well as some accretion at the side slopes. Silty, but still non-cohesive sediments ($50 \mu\text{m} \leq d_{50} \leq 100 \mu\text{m}$) are almost solely transported in suspension. As a result the sediment will settle down over the entire channel width and the current will blow still a rather large amount of sediment over the channel. The above considerations lead to the following proposition of slope improvement with respect to morphology:

$500 \mu\text{m} \leq d_{50} \leq 1000 \mu\text{m}$: (small) side slope shift, no overdredging at channel bottom;
 $100 \mu\text{m} \leq d_{50} \leq 500 \mu\text{m}$: side slope shift or mid-depth pocket-holes, little overdredging at channel bottom;
 $50 \mu\text{m} \leq d_{50} \leq 100 \mu\text{m}$: minimum channel upper width to reduce trapping efficiency, significant channel-wide overdredging, in fact over-dimensioning of the entire channel, because sedimentation is almost evenly distributed over the width.

The above measures need to take possible tidal asymmetries into account. One could think of shifting just one side slope or shifting one side slope over a larger distance than another. Typical downstream behaviour is difficult to reduce, but also rarely unfavourable for slope stability or navigable depth.

Besides these measures for slope improvement, it can be interesting to observe the morphological behaviour of an initial channel geometry that is more or less identical to its *dynamic equilibrium shape*. Will this lead to a more gradual and widely spread sedimentation?

Although measures to increase geotechnical stability are to a lesser extent determined by grain size only, also some remarks can be made. The susceptibility to static liquefaction is reduced by flatter lower parts, while the susceptibility to dynamic liquefaction by waves is reduced by flatter upper parts. Slope failure due to unloaded micro-instability is most likely to occur at the steepest parts, while macro-stability (in non-cohesive soils) only comes into play, if weaker soil layers or unfavourable water levels are present.

Combined with the morphological considerations, the following improved slopes are proposed, see also Figure 11-1. All-embracing names could not be created, so one of the main characteristics of a certain slope design is chosen to name the alternative:

- | | |
|-----------------------------------|--|
| <i>I: slope migration:</i> | straight side slopes at some distance from the required profile to allow for some slope migration; |
| <i>II: static liquefaction:</i> | flatter lower part and steeper upper part, compared to slope I. This shape is implicitly equipped with a sediment trapping reservoir; |
| <i>III: volume reduction:</i> | a significant reduction in capital dredging volume is obtained and the part of the slope that is most attacked by storm waves is flattened; |
| <i>IV: dynamic equilibrium:</i> | a side slope with a steep middle part and a flatter toe and top resemble the dynamic equilibrium shape best; |
| <i>V: sediment trapping berm:</i> | this division into two smaller slopes is easy to dredge and satisfies the static liquefaction criterion of slopes not steeper than 1:3 and not higher than 5 m (see Paragraph 4.2.1). This shape also is equipped with a considerable sediment trapping reservoir. |

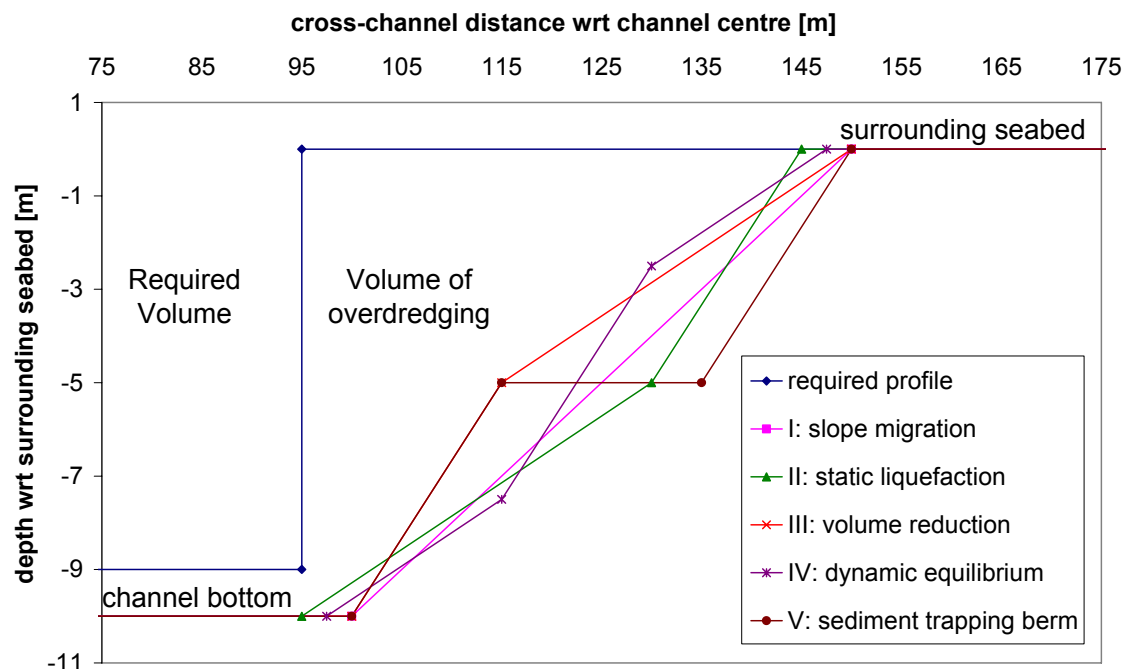


Figure 11-1: Schematization of required profile and improved side slopes

In Figure 11-1 also the required profile is plotted. This rectangular fairway forms the required space for navigation and is set to a minimum water depth of 19 m over a minimum width of 190 m. This navigable depth asks for a channel bottom 9 m below the surrounding seabed. This required profile needs to be guaranteed for a full year.

The hydrodynamic conditions are again obtained from MPN (see Paragraph 10.4.1). Because of the almost symmetrical morphological development, the same slopes are applied at both sides. Simulations will be done with MMS and MFS for all five improved slope designs. The coarser sand (MCS) is assumed to behave similar to MMS, only at a

slower rate. Silt will be transported in such large amounts that every channel profile would experience a rather fast backfilling.

11.2 Results of simulations with improved slope

11.2.1 Improved slopes in medium packed fine sand (MFS)

The first five simulations with improved slopes in MFS are presented in Figure 11-2; this figure zooms in on the ebb slope to be able to distinguish between the different slopes. It can be concluded that the shapes of the initial profiles are smoothened by current and waves, but some characteristics are still visible.

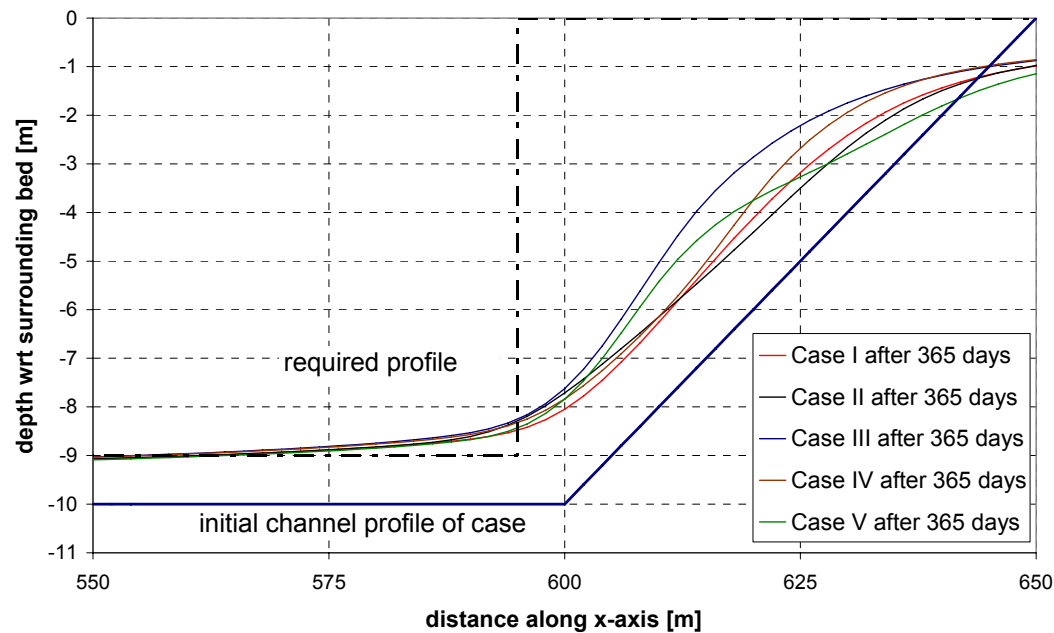


Figure 11-2: Morphological development of improved slopes; zoomed in on right (= ebb) slope

To come to a comparison of all alternatives first some aspects will be investigated in more detail. In Paragraph 10.4 wave-induced slope development throughout the year was discussed. The largest waves primarily contribute to slope development. Which 'improved profile' will be most affected by storm waves?

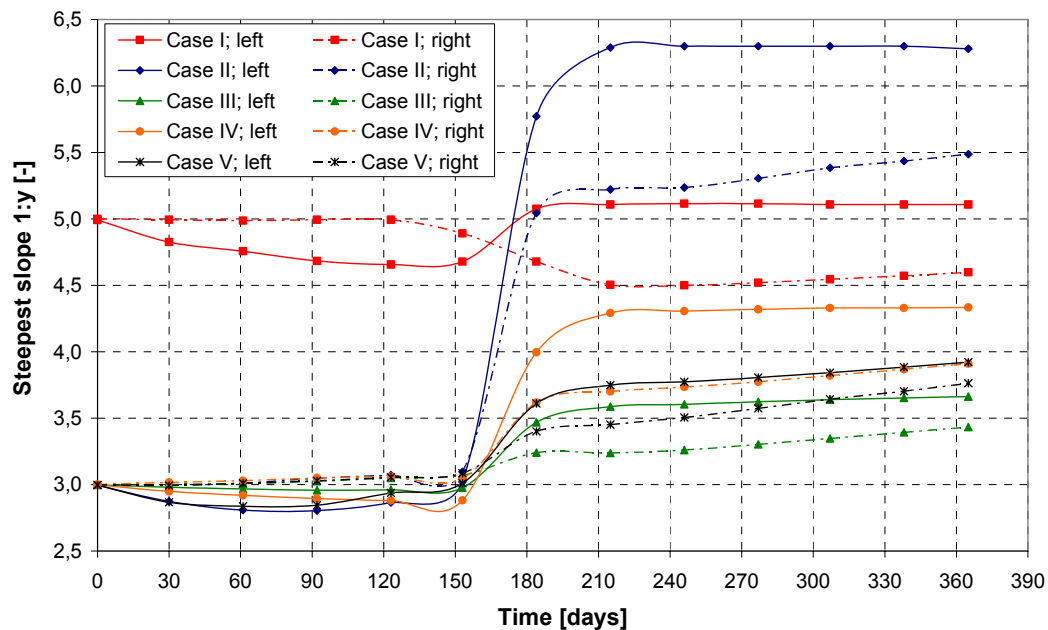


Figure 11-3: Steepest slopes of improved profiles throughout the year

When observing the (again) rather chaotic Figure 11-3, one can conclude that especially case II is sensitive to the stormy period of the year; case IV and case I to a lesser degree, while side slopes of profiles III and V only slightly flatten. This can of course be explained by the fact that steeper upper parts of the side slopes are more easily affected by larger waves. The more stable profile III has a relative flat upper part; the steeper lower part is situated deep enough to be out of reach of the larger waves. Profile V shows a more complex behaviour (Figure 11-4). The steep upper half is spread out over the berm (slope of 1:10), while the steep lower part can persevere (1:4). So Figure 11-3 shows an unjust favourable slope steepness of case V.

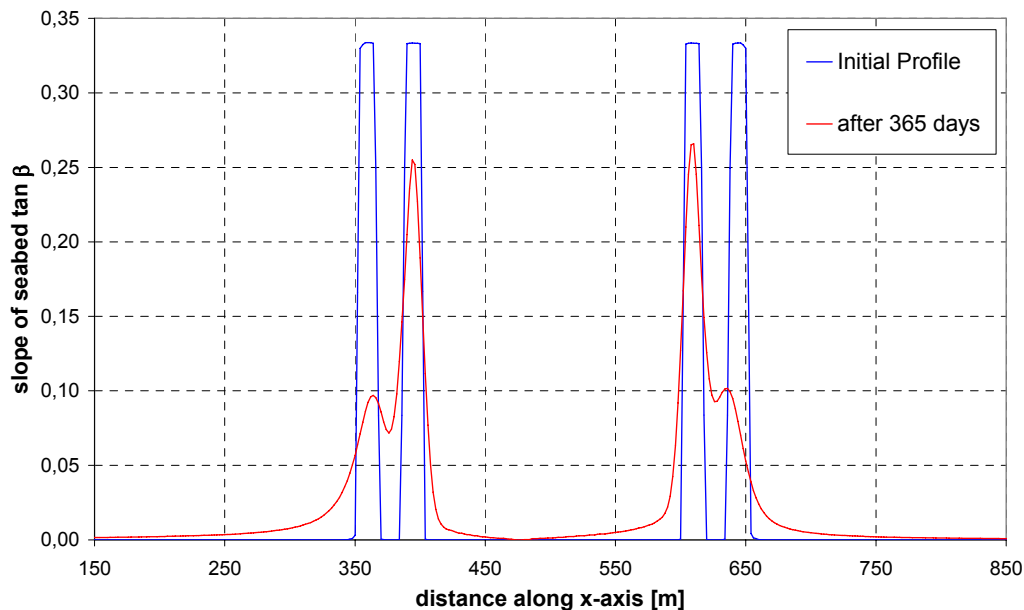


Figure 11-4: Slope development of improved profile V in one year

Another criterion is channel backfilling behaviour. Therefore some numbers of dredging volumes and backfilling rates are presented in Table 11-1.

Table 11-1: Backfilling behaviour of 'improved' profiles in MFS

		Profile I	Profile II	Profile III	Profile IV	Profile V
initially dredged channel volume	m ³ /m	2500	2500	2400	2450	2500
volume of overdredging	m ³ /m	790	790	690	740	790
sedimentation within initial channel boundaries	m ³ /m	284	294	273	282	291
sedimentation within required profile	m ³ /m	6,2	7,5	10,1	9,1	5,8
remaining channel volume	m ³ /m	2216	2206	2127	2168	2209
remaining volume of overdredging	m ³ /m	506	496	417	458	499
sedimentation/initial channel volume	%	11,4	11,8	11,4	11,5	11,6
sedimentation/volume of overdredging	%	36,0	37,2	39,6	38,2	36,8

Because this navigation channel has a rather large width and particle fall velocities are significant, almost all suspended particles will settle down inside the channel. Most time of the year, bed load transport will be practically zero at the channel bottom. This means that almost all incoming sediment will be trapped inside the channel. Measures to reduce the channel width can be very effective if grains are mainly transported as suspended sediment and have small particle velocities, but in this case these measures will not have any effect.

Therefore the amount of trapped sediment can best be reduced by reshaping the channel profile in such a way that little extra sediment is picked up *inside* the channel. The discussion on threshold profiles teaches that the upper part should not be too steep. Deeper inside the channel the side slopes can become steeper, because flow velocities are reduced. So, channel profiles II and V will induce most sedimentation, while profiles III and IV induce least sedimentation inside the channel. Sedimentation is thus determined by intraslope transport and not so much by the total channel geometry.

The criteria to distinguish between the various profiles are the following:

- capital dredging volume: this volume determines the initial costs;
- sedimentation within initial channel boundaries; this volume determines maintenance costs;
- slope development: which channel profile shows largest slope development during the 'rough' period of the year;
- static liquefaction: which profile runs the largest risk on liquefaction throughout the year? The stability of the toe of the side slope and the amount of sedimentation, which is generally loosely packed, at the lower half of the slope;
- cyclic liquefaction: the risk on cyclic liquefaction increases if the upper part of the side slope is steep;
- risk on reduction of navigable width: if the lower part of the slope approaches the required profile, there is a risk that the navigable width cannot be guaranteed.

The scores can be '+', '-' or '0' and are relative to each other. Depending on other soil properties and specific goals, the appropriate slope design can be chosen. For instance, if the soil is susceptible to static liquefaction, the fourth criterion will be of more significance. Otherwise profile II is not recommended, see Table 11-2.

Table 11-2: Multi-criteria analysis of improved profiles in MFS

Criteria	Profile I	Profile II	Profile III	Profile IV	Profile V
capital dredging volume	-	-	+	0	-
sedimentation within initial channel boundaries	0	-	+	0	-
slope development	0	-	+	0	0
static liquefaction	0	+	-	0	0
cyclic liquefaction	0	-	+	+	0
risk on reduction of navigable width	+	0	-	0	-
total	0	---	++	+	---

Profiles III and IV score well because of their morphological behaviour, while profile I is still an attractive, simple alternative. If the channel depth is only limited and slope heights remain small, the risk on slope instabilities, like static liquefaction, is reduced.

11.2.2 Improved slopes in medium packed medium sand (MMS)

The same procedure can be followed for improved slopes in MMS, see Figure 11-5.

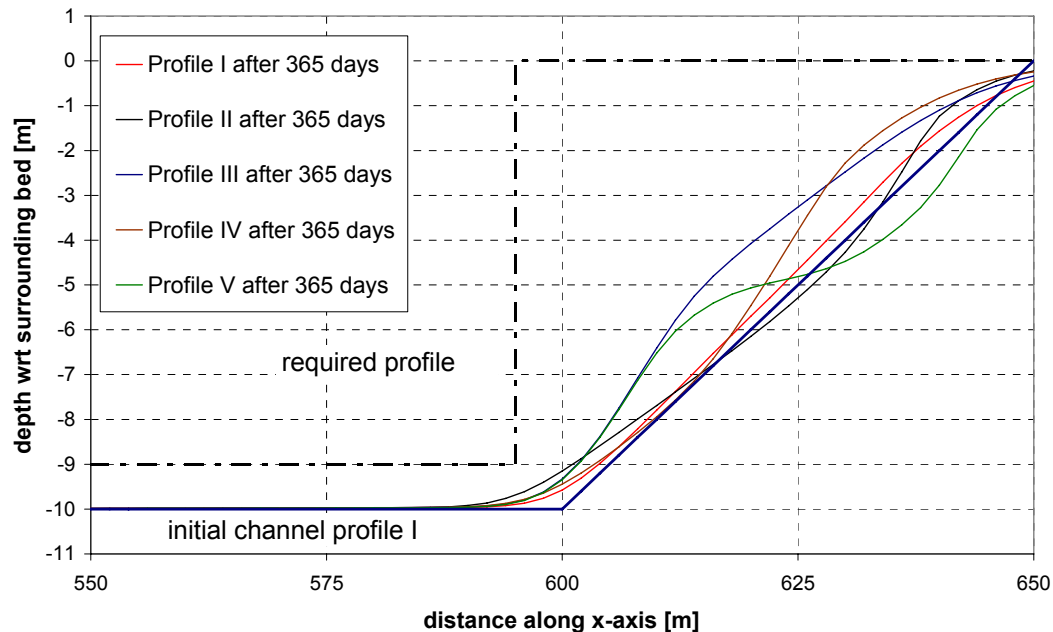


Figure 11-5: Morphological development of improved slopes; zoomed in on ebb slope

In medium sand the channel depth remains 10 m and morphological development is restricted to the side slopes. Sediment transport is primarily bed load transport, which adapts instantaneously to the depth expansion at the side slopes. The improved slopes are still visible after 365 days. Slope development is less pronounced than in MFS, see Figure 11-6. Profiles I and II react strongly to the midyear storm, while profiles III and IV show least variation in slope steepness.

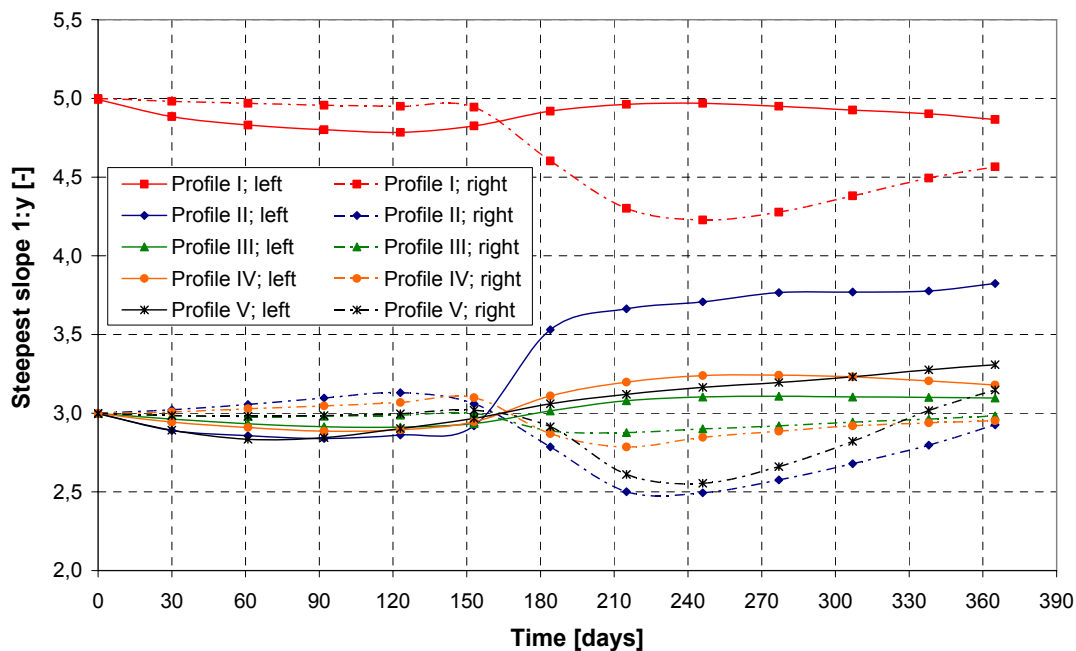


Figure 11-6: Steepest slopes of improved profiles throughout the year

The proportions of sedimentation rates between the different profiles are more or less the same, although the absolute amounts of sediment are less, see Table 11-3. In MFS the channel bottom entered the (prohibited) required profile, but this does not occur in

MMS. It will be interesting to determine the life span (time until bed level intersects required profile) of all improved profiles, but that is not the aim of this thesis.

Table 11-3: Backfilling behaviour of improved profiles in MMS

		Profile I	Profile II	Profile III	Profile IV	Profile V
initially dredged channel volume	m ³ /m	2500	2500	2400	2450	2500
volume of overdredging	m ³ /m	790	790	690	740	790
sedimentation within initial channel boundaries	m ³ /m	25	26	23	24	32
sedimentation within required profile	m ³ /m	0,0	0,0	0,0	0,0	0,0
remaining channel volume	m ³ /m	2475	2474	2377	2426	2468
remaining volume of overdredging	m ³ /m	765	764	667	716	758
sedimentation/initial channel volume	%	1,0	1,1	1,0	1,0	1,3
sedimentation/volume of overdredging	%	3,2	3,3	3,3	3,3	4,1

Again the profiles will be compared to each other on the same 6 criteria (Table 11-4). Some small deviations in the scores are noticeable. Because the contribution of bed load transport to morphological development is larger for larger sediment, the resemblance to a threshold profile (Paragraph 7.1.3) or a dynamic equilibrium slope (Paragraph 8.2.3) pays off. Profile III resembles these profiles most and scores well at the other criteria.

Table 11-4: Multi-criteria analysis of improved profiles in MMS

Criteria	Profile I	Profile II	Profile III	Profile IV	Profile V
capital dredging volume	-	-	+	0	-
sedimentation within initial channel boundaries	0	-	+	0	-
slope development	-	-	+	+	0
static liquefaction	0	+	-	0	0
cyclic liquefaction	0	-	+	+	0
risk on reduction of navigable width	+	0	-	+	-
total	-	---	++	+++	---

Finally, the morphological behaviour of profile IV can be observed in Figure 11-7. Only very minor slope development takes place.

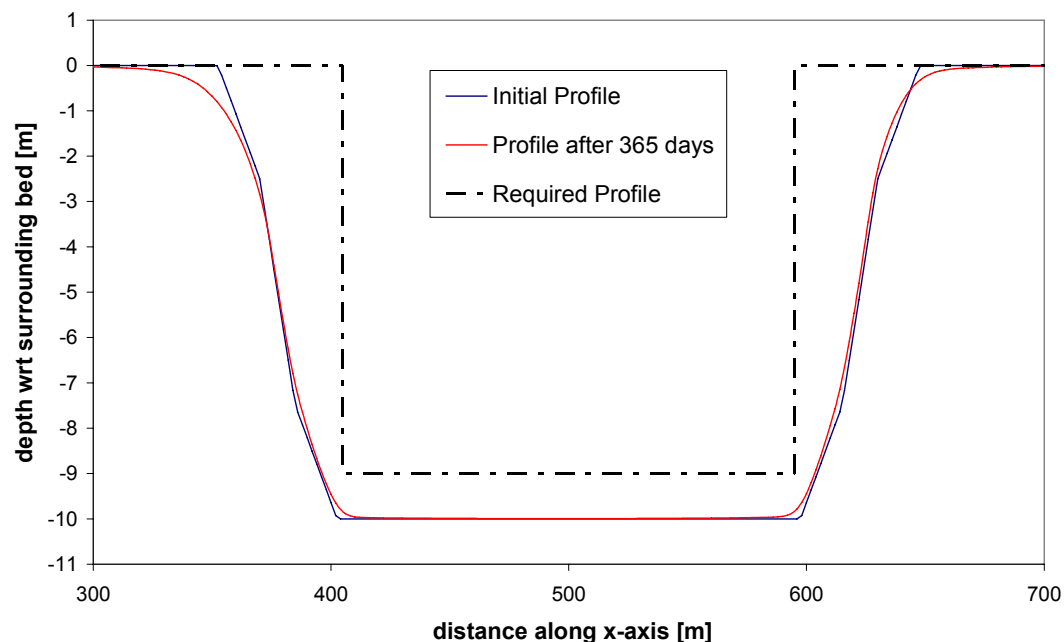


Figure 11-7: Initial profile IV and profile after 365 days

The above mentioned multi-criteria analysis does not contribute any weights. It depends on the soil properties which criterion is most important. Furthermore, one should constantly realize that these simulations are based on logarithmic velocity profiles. Steep slopes should therefore be regarded suspiciously.

12 Guidelines for slope design

Throughout this thesis research it became more and more clear that a straightforward and easily applicable set of design graphs that cover a wide range of hydrodynamic conditions, channel geometries and sediment properties probably cannot be composed. For example, tidal flow creates totally different slopes compared to unidirectional flow. Between these two opposites, all kinds of asymmetric tidal flow exist. Channel geometries strongly interfere with flow conditions. This interference becomes more significant, if side slopes become steeper, if the channel bottom becomes narrower and if the relative depth expansion increases. Only for gradually changing bed levels, the schematization with logarithmic velocity profiles in equilibrium is completely satisfactory. In the morphological simulations only the non-cohesive sediments were considered, assuming a rather narrow grading. Wider gradings ask for a 'multiple-fraction approach'. A reasonable approximation can be obtained if the grain size of the suspended sediment is assumed to be smaller than the bed-load sediment, but there is often a lack of precise data. Moreover, the composition of the suspended sediment changes with the hydrodynamic conditions.

Hopefully, one is aware of the above mentioned drawbacks, which are only a few amongst many others, when applying the presented guidelines, as some guidelines can indeed be deduced from this research as presented in the previous chapters. So in a way this chapter can be considered as a summary of the most important conclusions regarding slope design. At the same time this chapter has a generalizing nature. Simple guidelines cannot be represented, while taking all relevant parameters into account. When some guidelines seem obscure, the reader is referred to the relevant chapter.

12.1 Non-cohesive soils

12.1.1 No loads

As long as the soil shows no cohesive behaviour, the unloaded stability is governed by micro-stability (ϕ), see Table 4-1. Slip circle analysis showed that macro-stability is not likely to occur, unless weaker soil layers or a water level somewhere through the slope are present. Then the slip circle is attracted to these local weaknesses.

The governing parameter is therefore the angle of internal friction, which is usually larger for denser soils. Silts have an angle of internal friction that is about 3° smaller than sands of the same porosity.

When the instability mechanism is forced into a slip circle (macro-stability), safety factors are typical 10-40% larger (Table 4-2), depending on the slope height and the soil type, but the consequences of macro-instability are also larger. When the slope height goes to infinity macro- and micro-stability coincide; this occurs faster for weaker soils.

It was shown that when the water level drops and the soil behaves drained, safety factors are reduced about 15%, if the water level is located somewhere between the toe of the slope and $0,75D$. Besides, the normative failure mechanism changes from micro-instability to macro-instability. If the side slopes contain a relative impermeable layer, this strength reduction will be far more significant.

Flow slides can be caused by static liquefaction, when the sand is loosely packed. An easy to use criterion is based on the dry critical density which is for most sands about $n=0,42$. Soils with densities larger than the critical density ($n < 0,42$) are unconditionally stable. Soils with densities smaller than the dry critical density ($n > 0,42$) can still be stable, depending on the soil geometry (Figure 4-19). Slopes with heights smaller than 5 m are almost unconditionally stable, unless the soil is very loosely packed ($RD < 10\%$). For other combinations of slope height and soil properties the reader is referred to Paragraph 4.2.

One should keep in mind that these results can be a bit conservative due to the assumption that liquefaction is prohibited anywhere in the slope. By far the most important parameter is the relative density (or maximum, minimum and in-situ porosity). Especially the determination of the in-situ porosity used to cause difficulties. Nowadays very good correlations between the dielectric constant and porosity are available ($n = 0,0136\epsilon + 0,02$ for any soil type, see Paragraph 4.2.2.).

The slope height is very important when dealing with static liquefaction; doubling of the slope height requires a reduction of the slope steepness with a factor 2. When considering dredging of an overdepth, it can be dangerous to just increase the slope height if the soil is susceptible to liquefaction. A flatter toe of the slope can improve stability and simultaneously creates a more natural morphological profile. Variation of the slope geometry, for instance slopes with steeper upper parts and flatter lower parts, did not yield very large reductions of capital dredging volumes (less than 5%, which can increase for smaller channel widths), but adaptation of the side slopes to a more morphological channel profile is very well possible without giving concessions to slope stability or dredging costs. Small channel upper width reductions are also feasible. Due to the limited advantages on slope stability with respect to liquefaction, 'custom-dredged' slopes are only recommended if some combined advantageous effects can be obtained (sediment reservoir, width reduction, overdepth etc).

Although these guidelines are based on research on homogeneous soils, some remarks can be made on soil composites, like silty sand or sand-gravel-mixtures. Various researchers drew opposite conclusions, mainly caused by different assumptions of the relative density: some maintain the relative density of the original soil (neglecting the 'contaminant'), some of the soil composite. However, when a constant settling process is applied (which is representative for submarine conditions), all composites are less resistant against liquefaction. It should be noted that these results are built on two-composite mixtures. It is very well possible that widely graded soils will be more resistant against liquefaction.

Breaching is only likely to occur in slopes of medium to densely packed fine sand or silt ($d_{50} < 300\mu\text{m}$) that are steeper than the angle of internal friction. These soils have a very low permeability and are therefore able to maintain a situation of underpressured sand for some time. Although this instability mechanism controls the dredging process, it will not occur on the side slopes of a navigation channel, which (after dredging) are often not steeper than 1:3. Often for 10 m high slopes the following design rules are used: coarse sand slope of 1:2,5; medium sand 1:3; fine sand 1:3,5. A good idea can also be obtained from Figure 4-27.

Combinations of failure mechanisms 'liquefaction flow slide' and 'breaching' can occur, for instance a breach that starts at a steep scar produced by a small flow slide and retrogrades for many hours.

12.1.2 Wave loads

Waves appear in many guises when submarine slope development is discussed. Waves were already mentioned as a trigger mechanism for static liquefaction. They can also play a more influential role. Although wave forces are very common around sea level, waves below sea level act more as a reduction of strength. Many researchers distinguish between the direct, elastic effect and the indirect, plastic effect. The first approach predicts the fluctuation of water pressures and effective stresses within one wave cycle. This approach has no net effect on the mean stresses. The second approach takes cyclic compaction into account, which causes a gradual water overpressure. It depends on the consolidation properties of the soil whether these excess water pressure (EEP) can attenuate or will build up during a storm.

Direct effect

Every soil experiences this direct effect. It was shown (Paragraph 5.1.3) that the amplitude of the wave pressure has an upper boundary of about 25% of the hydrostatic

pressure, based on the assumption of linear wave theory and therefore decreasing water pressures with depth according to a cosh-function. In the direct approach these wave pressures fade out into the seabed according to an 'e-power-decay'. Maximum changes in effective stress occur at a depth of about 1/6 of the wavelength and are about 37% of the amplitude of the wave pressure at the bed. At this depth the overburden is already significant and the critical zone will be situated much shallower, where effective stresses are still small.

Simple calculations for realistic (depth-limited) storm waves showed for depths varying from 5 to 20 m that the cyclic stress ratio (CSR= wave-induced amplitude of effective stress divided by geostatic effective stress) will usually be in the order of 15-20% and will never exceed 40-45%, even if the unit weight is very small. Failure of the seabed can occur if the horizontal permeability is approximately 7 to 8 times larger than vertical permeability.

Although failure of a flat seabed caused by this direct effect is not very likely, safety against slope failure will decrease in presence of waves. The Mohr-Coulomb criterion should be adapted to effective stresses under wave crests and troughs. It asks for numerous calculations to define the critical wave (length, period, height and orientation to the slope). Such calculation tools do not exist yet. A safe compromise would be a reduction of the effective stresses (about 20-25%), so an increase of the required safety factor with the same percentage. Although still many questions exist on a completely sound theoretical description of wave-induced pressures and stresses inside the seabed, it seems that in engineering practice, where slope design usually not will be governed by micro-stability and slopes are often in the range of 1:3 or flatter, the risk of wave-induced slope failure implicitly is accounted for in the safety factor, as long as the indirect effect (see below) does not cause failure, which generally is the case for denser, more permeable soils.

Indirect effect

Every load cycle will induce compaction of the soil skeleton, no matter how dense the soil is, but it depends on the combination of load and strength whether the soil will be susceptible to cyclic liquefaction. The load is represented by the wave height, wave period, storm duration and water depth, whereas the strength is represented by the consolidation properties (k , m_v) and the relative density (RD).

Coarse, permeable and dense soils will have such good consolidation properties that they won't experience wave pressure build-up. Finer, impermeable, more loosely packed soils should be checked on this instability mechanism.

As a first design criterion Figure 5-14 can be used to determine the number of load cycles for a given wave condition (safe upper boundary). This graph is based on undrained response, which means that for better consolidation properties and/or pre-shearing conditions the situation will be more favourable. If local wave-induced relative shear stresses are in the order of 0,10 or larger, generally there is some risk on cyclic liquefaction. Then drainage and pre-shearing should be taken into account. In many off-shore conditions such stresses will be present as was shown in Figure 5-15.

Numerous calculations with MCycle made it clear that the dense reference soils (RD=0,8) are unconditionally stable under (depth-limited) waves ($H=5m$). The loosely packed reference soils (RD=0,2) all collapsed in a very mild wave climate ($H=1,00-1,75m$), which is an indication that such loosely packed soils are not very likely in a moderate wave climate. The common medium packed soils liquefied, when subjected to storms with waves of 3,25 to 4,75m.

Slope flattening was not very effective (see Paragraph 5.2.4). Instead one can achieve great results with improving soil properties. Relevant soil properties are:

- (hydraulic) permeability: an increase means that excess pore water pressures can easy dissipate to deeper layers, thereby increasing 'the depth of influence';
- vertical compressibility: a decrease means that compaction of the grain skeleton and the matching decrease pore volume takes more time, which is favourable for drainage of EPP's;

- relative density: an increase considerably raises the number of load cycles until cyclic liquefaction occurs or the maximum resistible relative shear stress. In other words, longer storms or storms with bigger waves can be withstood.

Of course all of the above soil properties are related to each other and manipulating one of them will certainly affect the others.

More permeable soils will spread the EPP's over a larger depth and the critical time will be at the beginning of a storm (typical after half an hour). Relative impermeable soils keep building up EPP's during the entire storm and these EPP's are more concentrated in the upper layer. So soils with a larger hydraulic permeability, like coarse sands, or a smaller compressibility, like dense sands, therefore are far less susceptible to liquefaction due to cyclic loading, see Figure 5-23.

Preceding storms appear to be primarily favourable for soils with small relative densities ($RD < 0,5$). The maximum wave height can increase with 0,5m.

Not many combinations of various soil layers have been investigated. A two-layer system (5 m thick upper layer on a 15 m thick ten times less permeable lower layer) did not yield spectacular results on maximum wave heights. Very thin, weak layers on top of an impermeable, dense layer can be more unfavourable. This has to be investigated.

Measures to reduce the risk and consequences of wave-induced cyclic liquefaction are:

-) adaptation of the channel geometry or alignment such that large sedimentation under calm hydrodynamic conditions is prevented or is forced to certain predestined places, like sediment pocket holes. This should prevent fast accretion of the side slopes, resulting in loosely packed sediment;
-) selection of a 'rough' dredging method which triggers regulated liquefaction flow slides or, if this does not occur, causes cyclic compaction of the soil, thereby increasing safety against liquefaction;
-) construction of discrete zones with stable material like gravel;
-) (partial) (vibro-)compaction of the soil;
-) soil improvement

Most effective measure certainly is compaction, because the relative density is the most influential parameter. Although slope flattening is not considered to be an effective measure to increase slope stability, slope steepness can be important, once a wave-induced flow slide has occurred. The erosive capacity of the resulting turbidity current is strongly dependent on slope steepness and slope height.

So whereas the direct, elastic wave effect was implicitly accounted for in the safety factor, the plastic, indirect effect can ask for serious measures!

Other loads

Not much attention was given to other than wave loads. Loads like earthquakes, tectonics and man-made loads (anchor forces, thrust of a ship's propeller) can act as a trigger mechanism to induce static liquefaction, but can also act in a more active, cyclic way. Seismically induced liquefaction shows many similarities to wave-induced liquefaction. Sophisticated software packages can be applied or the more straightforward approach of steady-state lines (Figure 5-33). Centre of fundamental research is located in Japan and the USA, because of the high earthquake frequencies.

12.1.3 Morphology

Often the initial channel profile will develop under the influence of hydrodynamics. In this thesis, (tidal) currents and wind waves are investigated. Once the bed-shear stresses, induced by these hydrodynamic forces, exceed a certain threshold value, sediment transport will reshape the channel profile. Especially in the presence of waves, almost all reference soils will come into motion. In special conditions slopes will develop towards less favourable shapes with respect to geotechnical instabilities; most of the time side slopes will flatten and slope development will be governed by morphology.

When currents approach a navigation channel there are two extreme possibilities. Currents that are directed perpendicular to the channel axis will slow down due to continuity, while currents parallel to the channel axis will accelerate due to equal water slopes inside and outside the channel, see Figure 6-1.

Oblique currents give a more complex situation and the relative flow velocity (V_1/V_0) can decrease as well as increase inside the channel, depending on the relative depth expansion, incoming flow angle, channel width and bed roughness outside and inside the channel. The perpendicular component will decrease almost instantaneously, while the longitudinal component needs some time to adapt. In practice this means that currents with incoming flow angles smaller than 30° will almost reach the equilibrium situation of fully refracted flow (Figure 6-4), whereas currents with incoming flow angles larger than 60° will slow down, unless the channel width is very large ($>1500\text{m}$, depending on the relative depth expansion), see Figure 6-6.

Analysis of a full 3D-Reynolds-averaged Navier-Stokes model taught that besides the current refraction, also a secondary flow will develop caused by shear in velocity profile. This means that the bed-shear stress on the upstream slope is refracted more than the depth-averaged current velocity. This difference in flow angle can grow as large as 30° for incoming flow angles of $60-80^\circ$ (Figure 6-10). At very steep side slopes (around 1:2) also flow separation can occur. There is hardly any sediment exchange between the separation bubble and the current.

When waves are directed perpendicular to the channel axis, the wave heights inside a navigation channel generally are 80-110% of the incoming wave height, depending on the wave period and the relative depth expansion. Based on continuity of wave energy flux, the opposite of wave shoaling occurs, see Figure 6-12.

Parallel incoming waves will also experience this effect, but due to curved wave crests the wave energy will be reduced at the side slopes.

Oblique waves either will be reflected or will be refracted, depending on the wave period and the relative depth expansion. The transition between refraction and reflection concerns wave angles of $20-60^\circ$, see Figure 6-13. Waves with smaller incoming angles will totally reflect, while large-angle-waves will refract. Especially incoming wave angles around this critical wave angle will result in an increase of the wave height inside the channel, see Figure 6-14.

The threshold of motion for the reference soils is about 0,3-0,5 m/s in terms of depth-averaged flow velocity. For grain sizes smaller than $500\text{ }\mu\text{m}$ this threshold value is nearly constant. Critical flow velocities decrease for downward sloping bottoms. In case of perpendicular flow this effect is noticeable for slopes steeper than 1:10, see Figure 7-6. Based on logarithmic profiles, the formula for the reduction factor (Eq. 7-10) and equally stable grains at the surrounding seabed and the top of the upstream slope, a threshold profile was deduced. This channel profile will develop under very mild hydrodynamic conditions, but also has an indicative meaning for stronger currents, because sediment transport is based on the difference between actual bed shear stresses and threshold values.

Fine to medium sediments are very easily stirred up by waves. Wave heights of a few dm are able to move some sediment. In combination with a current this can result in actual sand transport, because under the assumption of linear, non-breaking waves, waves themselves cause no net transport.

Simulations with SUTRENCH, a computer model developed by WL|Delft Hydraulics, gave insight in the morphological behaviour of channels and side slopes. The amount of sediment transport, and thus the time scale of morphological development, is most sensitive to wave height, current discharge, water depth, particle fall velocity and wave-related roughness.

The minimum sediment gradient, which is a measure for local sedimentation and therefore (upstream) slope development, is most sensitive to current discharge, water depth and rather sensitive to wave height and current- and wave-related bed roughness.

Sediment transport for the reference case 'navigation channel' can be approximated by formula 10-4. Although it can be very dangerous to apply this formula in a wide range of conditions, it shows the sensitivity to the important hydrodynamic parameters.

The current discharge and direction have a large influence on slope development. Two extreme conditions have been investigated: unidirectional and symmetric tidal flow. In the former case, two completely different side slopes will develop, hereafter named *upstream* and *downstream* slope. In the latter case both side slopes show a more or less equal morphological development. In between all kinds of asymmetric tidal currents are possible and the side slopes will show intermediate behaviour.

In unidirectional flow, downstream slopes will always flatten and stretch out over a considerable width. Upstream slopes will migrate and can show steepening as well as flattening behaviour. This behaviour is also time dependent. It was shown that initial upstream side slopes flatter than 1:4-5 first show some slope steepening and then a gradual development towards dynamic equilibrium slopes. Typical values of slope steepness are 1:5 to 1:7. Variation of initial side slopes taught that slopes converge during the first three months.

Very fine sediments did not show autonomous slope development, because backfilling proceeds very fast.

If the sediment transport capacity is increased by increasing current discharge or wave height, slopes can steepen significantly. Steepest upstream slopes seem to occur for increasing sediment sizes and hydrodynamic forcing (see Figure 10-18 and Figure 10-24), because the fast adapting bed load transport has a large share in total transport. A depth-averaged current velocity of 1 m/s resulted for sediment sizes larger than 200 μm in slopes even steeper than the angle of friction. Although such slopes are not realistic (slope failure will occur) and cannot be modelled properly with SUTRENCH, the morphological behaviour is in line with observations in nature. Large (constant) sediment supply, e.g. in river deltas, can cause oversteepening and, consequently, slope failure.

In tidal flow both slopes develop similarly. Upstream slope behaviour dominates over downstream slope behaviour. Because of the turning current, morphological development proceeds more slowly, slope migration and steepening are less pronounced and failure is not likely to occur. Slopes after one year are typical 1:4 to 1:6, unless backfilling processes start to interact with slope development; then fast slope flattening occurs.

If the hydrodynamic forcing is increased again some slope steepening can be observed (see Figure 10-38 and Figure 10-43). Currents are more effective in causing steeper slopes. Larger waves also increase sediment transport, but bigger waves start to interact with particles on the side slopes. Fine to medium sediment ($d_{50} = 200\text{-}500\text{ }\mu\text{m}$) combine the properties of being transported in large amounts and having a large sediment fall velocity. As soon as the current enters the channel, grains start settling down.

Simulations with a real tidal and wave climate yielded two important conclusions. Backfilling can well be predicted when using one representative wave height instead of a full-year wave climate. This representative wave height is rather large. When wave data are obtained as significant wave heights over small periods, a significant wave height, that will not be exceeded 80-90% of the year, has to be chosen as representative wave height. Although backfilling predictions are rather accurate, slope development is not well predicted. Slope development is governed by the largest waves. The minimum wave height that affects fast slope development is dependent on the water depth (see Eq. 10-5 for an approximation). A simulation over only the period in which this minimum wave height is exceeded yields accurate slope development.

Furthermore it appeared that the order of the waves throughout the year does not affect morphological development, if one is interested in one or more years. So, the effect of a storm, that occurs just after dredging has finished, is hardly noticeable after one year, see Figure 10-53.

Four different 'improved slopes' in fine and medium sand were compared to straight side slopes, see Figure 11-1. Slope development was evaluated on six different criteria,

varying from capital dredging volume to susceptibility to static liquefaction. Depending on the soil properties, tailor-made slope design seems advisable, unless suspended transport has a major share in total transport. For example, improved slope geometry 'dynamic equilibrium' shows least slope development and has lowest backfilling rates (see Paragraph 11.2). Geometry 'static liquefaction' scores worse on all criterions, but one: susceptibility to liquefaction. In loosely packed soil, this criterion will certainly outweigh the other criteria. It should be noted that the technique of improved soils will only be advantageous if slope heights are significant, bed load transport exceeds suspended transport and the designer has some interest in reducing dredging volumes, increasing stability or reducing slope development.

12.2 Cohesive soils

Cohesive soils received less attention than cohesionless soils in this thesis; partly because a number of stability mechanisms are not very likely to occur when some cohesion is present and partly because the underlying theories are not well established (research is often site-/soil-specific).

No loads

When no external loads are present, failure will always occur along (deep) slip circles. Micro-stability is prevented by cohesion. Because of the low permeability of cohesive soils ($k < 10^{-9}$ m/s) undrained calculations based on total stresses (undrained shear strength) have to be done. Besides, the long-term condition for fully drained soils should be checked. A completely satisfactory relation between drained soil properties (c , ϕ) and undrained soil properties (s_u) does not exist. Stress history and (over) consolidation ratio influence the undrained shear strength. In practice, determination of both properties is recommended.

Calculations based on a rather simple relation resulted in slightly lower Safety Factors for undrained calculations. Slope flattening appeared to be less effective if the soil behaves undrained (Figure 4-8). The sensitivity to slope height also becomes larger for undrained clay (Figure 4-9). The over-all conclusion however is that unloaded slope failure is not very likely in cohesive soils, because just stable slopes ($SF=1,5$) are in the order of 1:2 for slope heights of 10 m and in the order of 1:2,5-3,0 for slope heights of 20-30m. One should take into account that the reference clays under investigation are all relatively weak; in reality also much stiffer clays occur.

For small periods of time vertical walls can persist if the slope height does not exceed values of 1-2 m (Table 4-4).

Wave loads

Waves can liquefy cohesive soil just as cohesionless soil. Due to the even stronger dependency on soil properties, the large variation of these properties in cohesive soil, the lack of a computer program and the lacuna in theory of slope instability caused by wave-induced liquefaction, it was chosen not to investigate wave-induced slope failure in cohesive soils. Some researchers investigated this problem (see De Wit [1995]) and got some very nice results, although conclusions are only soil-specific (and thus site-specific).

De Wit found that waves with a wave height exceeding a certain threshold value were able to liquefy cohesive soils. This wave height increases with the consolidation period. Once a layer of fluid mud is present on the seabed, turbulence intensities in a current tend to decrease. The fluid mud layer behaves as a viscous fluid.

Morphology

Although in recent years great efforts were made at the ability to predict morphological behaviour of cohesive sediment, this subject received very little attention in this thesis. This lack of attention doesn't mean that this subject is not important, but it takes a rather different approach of sediment transport. Besides, the variability in soil properties of clays that are relevant to sediment transport makes it almost impossible to create a universal set of guidelines. With cohesive sediments, the bulk properties of the

mixture determine sediment behaviour, whereas non-cohesive sediments could well be represented by just the particle diameter (and the related particle fall velocity).

At present state-of-the-art it is still not possible to predict cohesive sediment behaviour from its physical and chemical properties alone. Most research therefore is aimed at laboratory tests in which hydrodynamic field conditions are simulated and cohesive sediment is taken from the field. A disadvantage of this method is the rather strong relation to the site. Additional fundamental research is needed.

The relevant processes and sediment states are illustrated in Figure 12-1. The four processes are erosion and transport, deposition and consolidation. Especially the last mentioned process is typical for cohesive sediment. Consolidation is the gradual expulsion of interstitial water by the self-weight of the sediment and cyclic stresses (due to waves) resulting in an increase in both the density of the bed and its strength (resistance against erosion) with time. Deposition is influenced by flocculation of particles caused by a change of the physico-chemical properties.

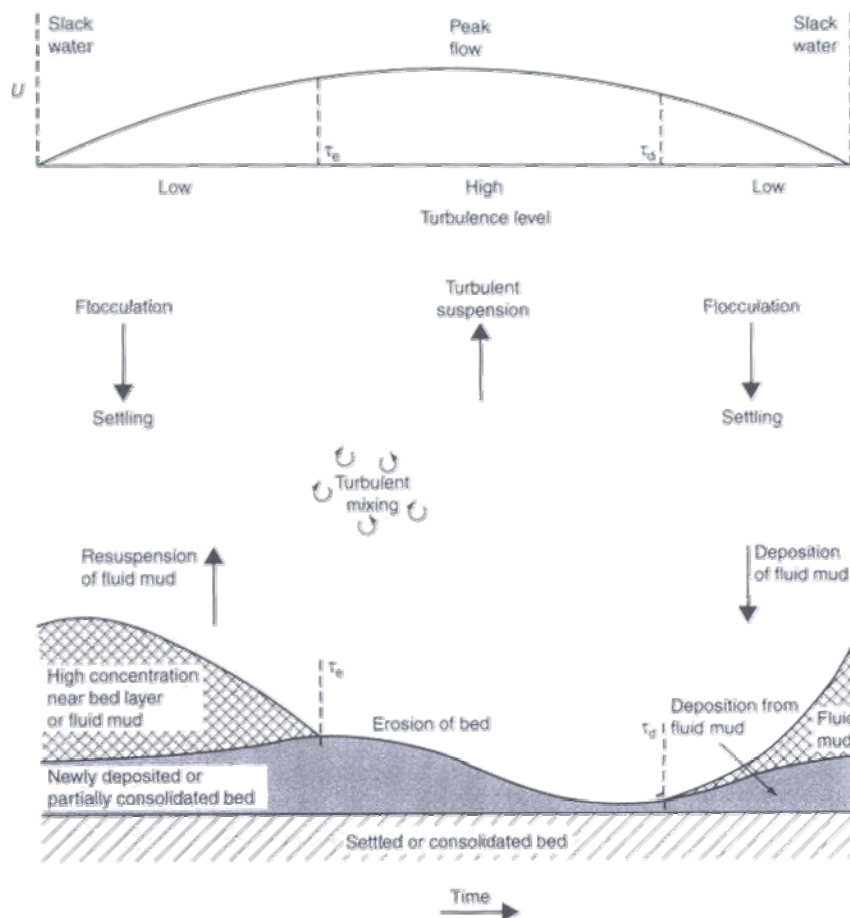


Figure 12-1: Schematization of cohesive sediment transport [source: Whitehouse et al., 2000]

The four states in which cohesive sediment can occur are mobile suspended sediment, a high concentration near bed layer (fluid mud), a freshly deposited or partially consolidated bed and a settled or consolidated bed. Thus, the separation between the seabed and the transported sediment particles is less pronounced than in non-cohesive sediment.

Due to the highly concentrated (erosive) fluid mud layers near the bed, slope development probably will be harder to predict than in non-cohesive soils. Once these fluid mud layers have come into existence, side slopes may become much flatter than non-cohesive slopes under equal hydrodynamic conditions.

In future years cohesive sediment transport and slope development in particular will slowly develop from site-specific research in theoretical research.

SUTRENCH is equipped with a module for mud transport, but this module hasn't been tested. Even sand-mud mixtures can be introduced.

13 Conclusions and recommendations

13.1 Conclusions

13.1.1 General

In the previous chapter guidelines for slope design were presented. In a way these guidelines can be interpreted as conclusions of this research. In this paragraph not the results of the various simulations are discussed, but the findings on achieved goals, present state-of-knowledge, the reliability of the results, the applicability of the present-day computer models and future developments. This will eventually lead to a set of recommendations for future research, described in Paragraph 13.2.

The main objective of this paragraph is of course to investigate to what extent this research has been able to satisfy the aims and objectives of Paragraph 1.3. Does this thesis report answer all the stated questions regarding slope design in engineering practice? This thesis should gain insight in:

1. submarine slope development in dredged trenches and channels;
2. the relevant processes and failure mechanisms;
3. the importance of soil mechanics and morphology and the (possible) interaction between them;
4. the behaviour of different types of soil (sand-silt-clay) with different particle diameters and soil properties.

1. Development of submarine slopes was considered due to morphodynamics. If this 'development' is caused by geotechnical processes, this was considered failure, because usually large masses are wasted. Simulations with SUTRENCH (to be mentioned more extensively in the next paragraph) gained insight in morphological development. The focus was aimed at the influence of initial side slopes, current discharge, wave height and wave schematization throughout the year.

2. Especially Chapter 2 gave a broad overview of all known failure mechanisms for a very wide range of slopes. It was shown that subaqueous slope failures do not only occur at steep slopes; even slopes of only a few degrees can collapse. The classification into (subaqueous) regions showed that most slope failures occur at continental slopes, while numerous release mechanisms were mentioned, which can act individually or in combination. Examples from dredging practice showed the state-of-the-art and problems that occurred in real cases.

3. In present engineering practice the initial slope is designed from a geotechnical point of view. Morphological development of side slopes usually receives little attention; computations are mainly aimed at predicting backfilling and expected life time of a navigation channel or pipeline trench with respect to maintenance costs. So the focus shifts in time from geotechnical towards morphological aspects. Interaction becomes important if due to morphological development the slope stability is reduced. Two possible causes are:

- the slope steepens, while the slope height does not decrease too much;
- soil properties, like porosity or permeability, worsen, e.g. because of large sedimentation under calm weather conditions.

Simulations proved that the first cause can occur under specific circumstances. The current has to be (almost) unidirectional and the wave height has to be limited: wave stirring on the surrounding seabed with limited wave action on the side slopes. This situation of a rather large, constant, unidirectional sediment supply of grains with large particle fall velocities primarily occurs at river deltas. In most cases this unfavourable combination will not be present.

The second cause sounds very logical, but is very hard to predict, because little is known about the soil skeleton during the submarine settling and consolidation process.

The soil strength increases in time, waves cause cyclic compaction and stirring at the same time. Soil stratification on the side slopes may occur, because larger grains will settle faster than smaller grains. If interaction should be considered, side slopes tend to be steep, which means that the current and wave pattern is rather complicated. Modelling of these processes is still very difficult, if not impossible. However, one could estimate the risks on this type of failure due to interaction. Most susceptible are slopes in fine to medium sands (no cohesion) in unidirectional flow with little wave compaction and large slope heights.

4. Not all soil types received equal attention. Cohesive soils were only discussed briefly; partly because a number of instability mechanisms are not very likely to occur, when some cohesion is present, and partly because the underlying theories are not well established (research is often site-/soil-specific). In Paragraph 13.2.1 some brief remarks on cohesive sediment transport are made. Non-cohesive sediments were studied for a wide range of soil properties. Every failure mechanism has its own important soil parameters, which are often not related. Sometimes, simple assumptions had to be made.

So, although all aims and objectives have been studied, the problem could not completely be solved.

This thesis research tended to bridge the gap between theoretical research and practical engineering tools. But not only this gap appeared to be very important throughout this study, also the gap between two fields of expertise turned out to be very evident, namely 'soil mechanics' and 'coastal morphology'. In literature specific problems were treated separately; the same symbols were used for totally different quantities in both disciplines; most researchers were either geotechnical or hydraulic/morphological engineers. Gradually, some mutual efforts on the interface between soil and water are made by representatives of both disciplines, for instance in Delft Cluster on breaching processes.

Starting from the development of both research fields, this division can be understood, but it is to be expected that the interface of soil and water will play a more and more important role. An increasingly larger area of land is reclaimed from the sea. The demand on reclamation soils increases, which was already shown in Singapore. The submarine slopes determine to a large extent the required amount of soil. As world trade and harbours grow, vessel sizes will increase and navigation channels have to be deepened. Due to a lack of space, side slopes need to be steeper. As a consequence of steeper side slopes and larger relative depth expansions, the hydrodynamics are more and more influenced by the presence of a channel and start to interact with the seabed. Soil mechanics, fluid mechanics and morphology cannot be treated separately anymore. They are constantly involved in a process of dynamic interaction.

Due to the very broad set-up of this thesis, it was impossible to study every single instability mechanism in great depth. This was not only caused by the limited time or theoretical knowledge of the author, whose educational background is morphological, but also due to the already large gap between the most sophisticated theories and the much more straightforward approach in practice. Therefore this thesis research was characterized by a constant search for balance between theoretical depth, which is a prerequisite for a good master's thesis, and practical applicability, which is of course the express wish of the customer, Royal Boskalis. As a consequence every specialist on a certain subject will perhaps not be dazzled by striking innovations or completely new theoretical ideas, but on the other hand he will learn something from the other topics that have been discussed.

Because of the complexity of many of the governing processes, it appeared to be very difficult to make sound guidelines for slope design right at the interface of both disciplines, because on the one hand the lack of theoretical background and on the other hand the fact that known theories had to be applied at their boundaries of validity. For instance, wave pressures inside the seabed are based on a flat seabed, or velocity

profiles based on logarithmic velocity profiles. Nevertheless some universal guidelines were deduced and presented in Chapter 12.

In engineering practice only a few, but certainly not all parameters needed for good sediment transport predictions are available. Nevertheless some calculations have to be executed that have some predicting value. Qualitative behaviour sometimes can reasonably be estimated, but quantitative behaviour and exact timescales are a lot to ask for. A reliable method to improve the accuracy can be obtained by dredging a trial trench or channel.

As was mentioned before almost all calculations methods are based on schematized sinusoidal waves, whereas in coastal regions these waves should be described by the cnoidal wave theory. Because of the lack of mathematical formulations on cnoidal waves it is impossible to introduce cnoidal waves in mathematical models. In numerical models this can be done, but most sediment transport formulations and wave pressure penetration formulas are based on sinusoidal waves.

When considering the indirect plastic effect, more attention should be given to the wave pattern, because the occurrence of a group of very large waves is far more effective in causing flow slides than a more regular wave pattern. Also the history of the sea bed is very important, although it is almost impossible to describe it accurately, because of the porosity which is dependent on the current and wave conditions throughout the year. A sudden storm in an otherwise calm hydrodynamic region has far more impact than a slowly increasing storm in a rough climate throughout the year.

13.1.2 Applied computer models

In this thesis various computer programs have been used, some of which are available on the market, some of which are for internal use only. All used computer programs will be discussed with respect to the validity of the computed results and the applicability in engineering practice.

M-Stab

This 2-dimensional slope stability program by GeoDelft is easy to use for both drained and undrained calculations. The possibilities to include loads are limited; only constant earthquake loads can be taken into account in a very schematized way. For groundwater flow additional calculations can be done with M-Seep, but a satisfactory method to model waves still does not exist.

SLIQ2D

This 2-dimensional program calculates the just stable slope with respect to static liquefaction. The results probably are a bit conservative due to the criterion that liquefaction is prohibited everywhere in the slope. The input asks for extensive soil tests to obtain no less than 11 soil input parameters, which is already a strong improvement compared to earlier versions of this program. In this thesis a sensitivity analysis was executed to reduce the large number of input parameters to only a few, more familiar input parameters. Indeed, it can be concluded that it is possible to obtain reasonable results when, besides of course the slope properties (height and shape), only the porosity (or relative density) and the maximum volume strain are known. Still, even these two properties are hard to predict in field conditions. Therefore a reliable correlation on the porosity and the dielectric constant, which can be measured easily with simple adjustments to the well-known CPT-equipment, was presented. One big advantage of this correlation was the independence on soil type. It is recommended to increase the efforts on electric permittivity measurements.

The development of SLIQ2D has stopped in the mid 90's and future improvements are not to be expected. SLIQ2D can only handle homogeneous soils and is based on a number of PLUTO-stress-calculations and therefore not very transparent. The more widely used program PLAXIS may be more suitable for development of a special module that calculates susceptibility to liquefaction.

MCYCLE

This is another computer program developed by GeoDelft that is based on the uncoupled approach of direct, elastic stress-strain behaviour within a wave cycle and the indirect, plastic behaviour over a large number of wave cycles. Many schematizations were needed to develop a model that is able to predict whether excess pore pressures in the seabed will build up during a certain storm period. The results probably are reasonable for a flat seabed, but will deviate from reality if the seabed is sloping or in the presence of structures, like caissons or breakwaters. Many researchers consider wave pressure built-up as one of the most difficult problems. Too little is known about the exact wave pressures below the water surface (especially larger waves deviate from linear sinusoidal waves), the state of the seabed, the interaction between wave pressure at the seabed (p_0) and the composition of the seabed itself, et cetera.

SUTRENCH

SUTRENCH developed by WL|Delft Hydraulics is a user-friendly model with a less user-friendly interface. The model yields reasonably good results and can still compete with more sophisticated models like Delft3D. In fact if some calibration data are available, it produces more realistic morphological development of trenches and channels than Delft3D, as was investigated by Klein [2003]. Because SUTRENCH is primarily suitable for gradually varying flows, DELFT3D should probably yield better results for steeper slopes and more severe hydrodynamic conditions, although this program is also bounded by hydrostatic pressure distributions. A model that solves the full Navier-Stokes equations developed by Jensen et al. was described. This model appears to produce nice results, but this very sophisticated model has the strong limitation that waves can still not be implemented.

A disadvantage of SUTRENCH concerns the very schematized input data. Tidal currents have to be discretized in relatively large 'blocks'. The time step to compute bed changes is dependent on the duration of such a tidal 'block'. A wave climate can only contain 20 periods of a constant wave height.

Another disadvantage is the constant grid size. On the one hand a certain adaptation length must be guaranteed, asking for a large grid size, while on the other hand one desires a small grid size at the location of the side slopes, where the bed level and flow are changing rather fast.

The output was not aimed at producing the steepness of the side slopes, which is understandable, because this program is not intended to simulate slope development. Besides, the definition of slope steepness is arbitrary and should be based on the instability mechanism one is interested in.

Because of the relative steep side slopes, it is expected that the results are reasonable accurate with respect to the amount of sedimentation within the channel boundaries, whereas it is questionable whether the exact sedimentation/erosion pattern on the side slopes is correct. The results seem realistic and can explain some of the phenomena observed in practice. So, there is some confidence in the qualitative behaviour, but the quantitative behaviour and the representative timescales may be less reliable.

13.2 Recommendations

Based on the idea that this thesis should be the bridge between theoretical research and dredging practice, the recommendations are subdivided into two categories. In Paragraph 13.2.1 the lacunae in current theories are discussed. These lacunae need to be filled to improve the calculation methods. In Paragraph 13.2.2 some steps to fill the gap between slope design guidelines, in fact this thesis, and the application in practice are explored.

13.2.1 Future fundamental research

Wave pressure build-up in water and in layered soil

Little is known on pressures caused by non-sinusoidal waves. Some remarks were made on differences between dense and loose sea beds. Pressure development inside slopes will also deviate from a flat seabed. Still, many questions have to be answered;

Wave pressure modelling in finite element method

Waves are still not implemented in finite element methods, like PLAXIS. Some additional research on wave pressures inside the seabed probably has to be done, but with present-day knowledge nice predictions can be done;

Morphological modelling of slopes in cohesive sediment

As was mentioned in Paragraph 12.2, progression is achieved in modelling of sediment transport in cohesive soil. On steeper side slopes the layer of fluid mud makes predictions on slope development very difficult. Once a layer of fluid mud comes into existence, slopes can flatten significantly and cause failure of the navigation channel;

3D-hydrodynamic and morphological modelling at steep side slopes

Jensen et al. [1999] made a very nice attempt, but they still not included waves, see Paragraph 6.1.2 for a possible model set-up. It was mentioned many times that the representation of the current velocity with logarithmic profiles introduces errors that increase for steeper side slopes. As a first indicative step one could compare measured bed shear velocities with the 'logarithmic-based' bed shear velocity to get an idea of the magnitude of this error. Probably, some more laboratory data with various current and wave input over steep side slopes are needed. At the same time, wave pressures on top of and inside the seabed can be measured.

13.2.2 Applicability in dredging practice

Correlations between soil properties

The porosity and the maximum dilatant volume strain appeared to be the most important soil parameters that determine the susceptibility to liquefaction. In dredging practice often no data on porosity are available. Therefore in Paragraph 4.2.2 a method based on the dielectric constant was presented, which showed reliable results. In the future this method will become invaluable in regions that are susceptible to liquefaction and an investment on such a device is recommended.

Probabilistic slope optimization for economic purposes

In the future, tender forms like '*design, construct & maintenance*' will become more common. Contractors will be responsible for the functionality of the channel or trench for a certain period. The trend towards maintenance contracts will also shift the period of interest of a contractor. The transition from optimal slope design in the period of capital dredging to optimal slope design over the entire life span of the navigation channel (capital and maintenance dredging) asks for a somewhat different approach. The focus shifts from the side slopes and their stability during construction towards the morphological behaviour of the entire channel geometry. In fact, the design of the side slopes is not longer based on stability calculations and steepening/flattening processes, but on the consequences for the backfilling of the channel as a whole. During this research it already turned out to be rather difficult to treat the side slopes as separate parts of a navigation channel. As time went by, the channel started to behave more and more as a whole of bottom and slopes.

A very nice 1D-attempt was made for the Mississippi by Moser et al. [1995]. Of course in offshore conditions, the situation is far more complicated. Risk-based planning has to be involved. Some very interesting questions will arise. Will the contractor be responsible for a '1:10.000-year storm'? What will happen if coastal measures in the neighbourhood change the sediment pattern? Which risks is the client willing to take and which risks are for the contractor?

Database of navigation channels

Because few fitting data are available, many models are calibrated with the same data. To enlarge the range of such models, a database of all navigation channels over the world would be of enormous value. During this research it appeared to be very difficult to obtain input data from old project files. If a database on navigation channels is available, new projects can easily be added to the database. One can think of hydrodynamic data, soil properties, survey data et cetera.

13.2.3 Set-up of laboratory experiment

Already many shortcomings of theoretical knowledge have been mentioned before. Some additional laboratory experiments are suggested to answer some of the remaining questions. In Figure 13-1 a possible model set-up is proposed that should gain insight in the following:

- wave pressures are often assumed to satisfy linear wave theory, although especially in coastal regions waves become non-linear. What water pressures in a vertical column of water are induced by cnoidal or second order waves? What is the difference with linear waves? Especially the orbital pressures at the seabed are important;
- some researchers already stated that the porosity of the seabed influences the wave pressure at the seabed. The (hydraulic) permeability seems to be important. Is this phenomenon always important for every wave type?
- how will wave pressures attenuate inside the seabed, considering three different relative densities ($RD = 0,3; 0,5; 0,7$), two different grain diameters ($d_{50} = 200; 300 \mu m$) and two different layer thicknesses;
- is there some phase lag noticeable? It was already mentioned that a phase lag could cause failure, but it is questionable whether such failures do occur in practice.

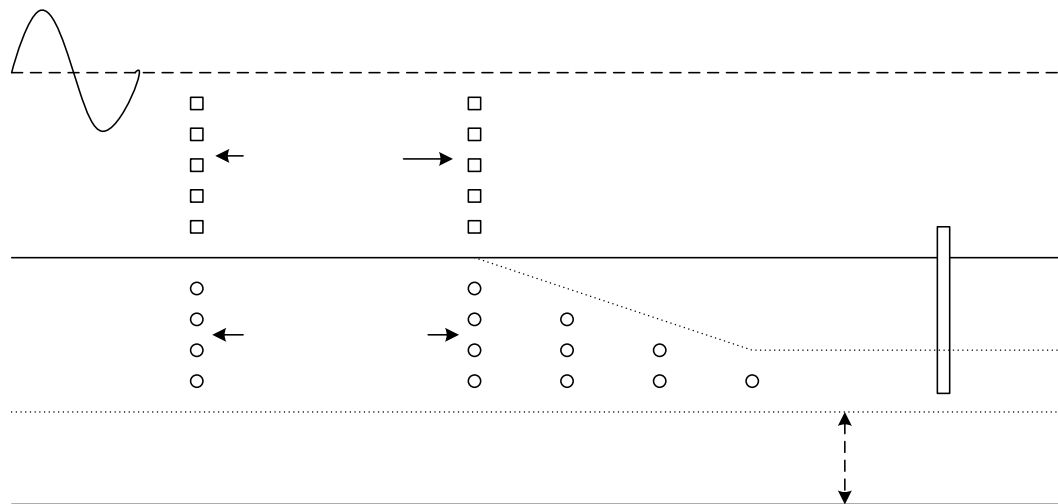


Figure 13-1: Schematization of laboratory set-up to measure wave pressures outside and inside a flat seabed for different wave types, sands, relative densities and thickness of the seabed.

After all measurements on a certain soil are carried out, some soil tests are executed to obtain the in-situ soil properties (unit weight, porosity, relative density, permeability and angle of internal friction). These soil tests will be done in the area that will be excavated to 'dredge a channel'.

Then the same waves as the previous experiment will be generated, see Figure 13-2. The left-hand piezometers and stress cells serve as a control system. These measuring instruments should be located so far from the side slope that the test results should not be affected by the presence of the sloping seabed. The right-hand instruments record the wave pressures at a steep side slope. It will be studied which waves (H , T , L) cause slope failure. These failures will be registered on film to answer the following questions:

- where do failures start;
- how deep is the failure plane located (macro/micro-instability?);
- what are the consequences of failure (sliding/slumping, liquefaction flow slide, turbidity current);
- what does the resulting geometry look like? How much material is involved.

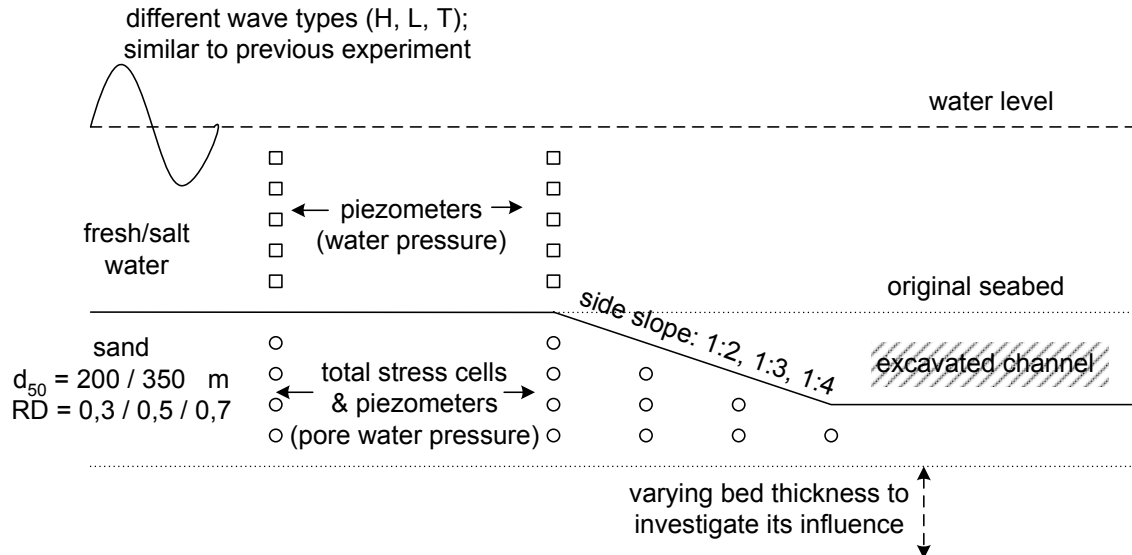


Figure 13-2: Schematization of laboratory set-up to measure wave pressures outside and inside a flat seabed for different wave types, sands, relative densities and thickness of the seabed.

The following experiments deal with sedimentation behaviour. A unidirectional current will be generated from left to right. Can the observed velocity pattern well be predicted by a numerical model? Is this also the case for very steep side slopes, when separation bubbles appear?

This current supplies large amounts of sediment. Can the sediment transport process well be predicted, using a numerical model?

Besides the no-wave situation, also small waves are considered. How will the morphological development of the side slopes be influenced by these waves? What are the differences in settling behaviour? What is the relative density of the freshly deposited sediment? Based on the observed morphological development, a certain moment in time will be chosen at which the side slope may be more susceptible to failure than the initial situation. Large waves that have proven to be able to cause slope failure will be generated. Again failure will be monitored. Will this failure mechanism be equal to the initial situation? Is failure caused by an unfavourable slope geometry (steeper side slopes) or an unfavourable relative density (more loosely packed sediment)?

μ

The above described sediment should gain insight in the soil-water-interaction at steep side slopes. Wave loads act as a sudden release mechanism and a more gradual transport and compaction mechanism. Based on the experimental data, the numerical models that predict pore water pressure, consolidation, current velocity profiles and sediment transport can be improved. Eventually, the ultimate goal is a complete numerical model that describes submarine slope development in time and that can handle morphological development as well as slope failure. A set-up of such a model is described in the next paragraph.

13.2.4 Set-up of calculation model

In order to be able to model the interaction between morphology and soil mechanics a coupled approach is advised. The set-up of a possible model is presented in Figure 13-3.

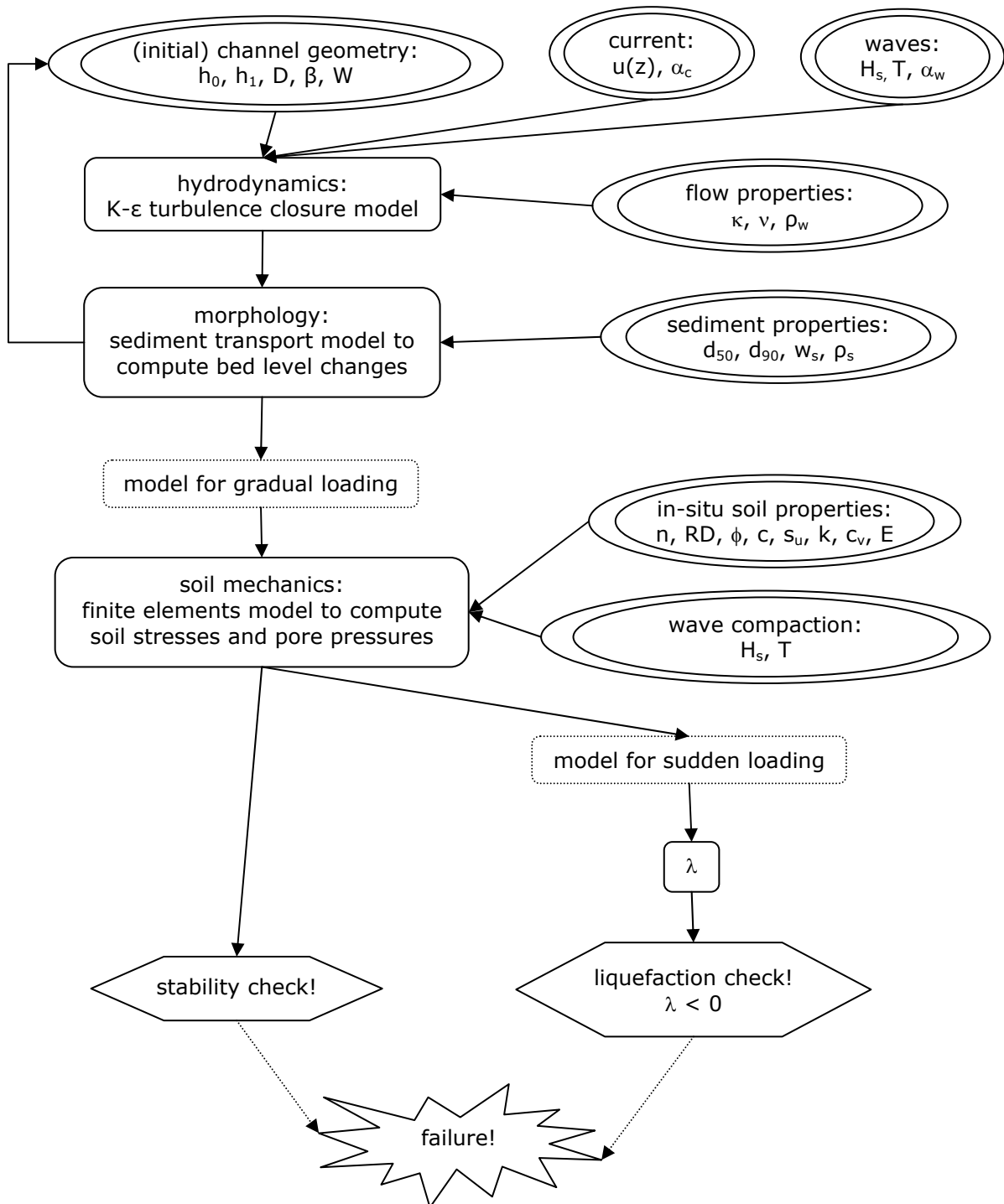


Figure 13-3: Schematization of calculation model set-up

The biggest difficulty that will be encountered when setting up such a sophisticated interactive model will be the different stage in theoretical foundation of all components. Therefore some additional research was recommended. Another difficulty is the range of different time scales one has to cope with when running such a model. One can think of:

- small timescale for turbulent flow and sediment transport;
- medium timescale for adaptation of bed level;
- medium to large timescale for consolidation of (sedimented) layers under the combination of time and wave action;
- large timescale for stability-checks (slope collapse, slumps or liquefaction flow slides). Some loads, like earthquakes or storm waves, can be exerted on the side slopes. For each specific load a safety factor F or even better a probability distribution of failure can be defined. In this way the behaviour of a navigation channel throughout time (instead of just the initial stability) can be predicted. An example of its applicability is for instance a navigation channel that is dredged during summer under mild hydrodynamic conditions and that will be subjected to large sediment supply during winter and that will experience severe storms or earthquakes. The engineer will gain insight in the most critical stage of the channel during its lifespan and may be able to adapt the current design.

Of course such a complete computer program asks for a lot of effort and cooperation of two still rather separated disciplines. Besides, many fitting data have to be available and because only very few projects have been monitored so extensively that all relevant input parameters are available, laboratory experiments are needed for supplementary data. These experiments can be done under fully regulated conditions, while observations and measurements are relatively easy, compared to real projects, especially because failure mechanisms cannot be controlled.

13.3 Final remarks

Most critical remarks of the readers are implemented in this thesis. My appreciations go out to all who pointed me at errors, indistinct formulations or newer theories. However, not all remarks are implemented due to time limitations or differences of opinion.

Relation between drained and undrained soil properties

The estimation formula of the undrained shear strength appeared to be out of date. Henkel included a correction factor for the second term to account for the effect of over- or underconsolidation. Because the relation was used for indicative purposes, it was decided not to execute all undrained MSTAB-calculations again. Besides, the conclusions are not expected to change.

Ratio between horizontal and vertical earth pressure

The ratio between horizontal and vertical earth pressure was obtained from a linear elastic stress-strain relationship (equation 4-5). Another well-known expression for the earth pressure at rest is that of Jacky (1944, as described by Keskin et al. [2004]):

$$K_0 = 1 - \sin \phi \quad \text{Eq. 13-1}$$

This equation is more commonly used for determination of K_0 in normally consolidated soils. Keskin et al. [2004] summarized a number of expressions for K_0 and conducted a lot of oedometer-tests. They stated that this factor K_0 changes depending on relative density, stress-history, overconsolidation ratio (OCR), plasticity index and similar soil properties. Because it is strongly recommended to measure the undrained shear strength and not to deduce this soil property from cohesion and friction angle, the calculations have not been executed once again.

Appendix A: List of references

This list contains all used references and some recommended literature. In the thesis text, references are indicated by '[...]' signs.

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Appendix B: List of symbols

Symbol	Description	unit
Latin		
a	wave-induced particle excursion	m
or	thickness of bed load layer, reference level	m
c	cohesion	kPa
C_{K-C}	coefficient in formula of Kozeny-Carman; 0,01	-
C_v	consolidation coefficient	m ² /s
d	distance between plates in capacitance measurement	m
d_s	thickness of the seabed	m
d_{50}	particle diameter at which 50% by weight is finer	m
d_{90}	particle diameter at which 90% by weight is finer	m
d^*	dimensionless particle diameter	-
e	void ratio	-
or	trapping efficiency	-
e_{max}	maximum void ratio	-
e_{min}	minimum void ratio	-
f_w	friction factor for waves	-
g	constant of gravity	m/s ²
h	water depth	m
h'	effective water depth of Tsui et al.	m
h_0	surrounding water depth	m
h_1	water depth in trench/channel	m
h_c	critical wall height of purely cohesive soil	m
i	water surface slope	-
k	wave number; $2\pi / L$	rad/m
or	hydraulic conductivity, permeability	m/s
k_r	effective bed roughness	m
$k_{r,c}$	current related bed roughness	m
$k_{r,w}$	wave related bed roughness	m
k_x	permeability in x-direction	m/s
k_z	permeability in z-direction	m/s
m	fit constant of dilatation curve (SLIQ2D)	-
m_v	coefficient of vertical compression	m ² /MN
n	porosity	-
n_{max}	maximum porosity	-
n_{min}	minimum porosity	-
p	(pore) water pressure	kPa
p_0	amplitude of wave pressure at seabed	kPa
r	fit constant of dilatation curve (SLIQ2D)	-
s	specific density (ρ_s/ρ_w)	-
s_{max}	asymptote of relative shear stress (SLIQ2D)	-
s_u	undrained shear strength	kPa
s_2	relative shear stress at which $\epsilon_{vol_{dm0}}$ occurs (SLIQ2D)	-
u	excess pore pressure (EPP)	kPa
or	horizontal flow velocity in x-direction, perpendicular to channel axis	m/s
or	power of decompression curve (SLIQ2D)	-
u_h	flow velocity at water surface	m/s

$u_{h,e}$	equilibrium flow velocity at water surface	m/s
u_*	bed shear velocity	m/s
v	horizontal flow velocity in x-direction, parallel to channel axis	m/s
σ	power of isotropic stress (SLIQ2D)	-
w	vertical flow velocity in z-direction	m/s
w_L	liquid limit	%
w_P	plastic limit	%
w_s	particle fall velocity	m/s
x	horizontal axis, perpendicular to trench/channel axis; positive in flow direction	m
y	horizontal axis, parallel to trench/channel axis; positive in flow direction	m
z	vertical axis; positive upward	m
z_0	zero-velocity level	m
z_a	reference level of bed form height	m
z_b	bed level	m

Latin Capitals

A	parameter which represents internal generation of water pressure due to waves	
σ	surface of plates in capacitance measurement	m ²
σ	auxiliary parameter of dilatant volume strain	-
B	auxiliary parameter of dilatant volume strain	-
Ba	Bagnold number	
C	Chézy coefficient	m ^{0.5} /s
σ	capacitance	F=V/C
CSR	Cyclic Stress Ratio ($=\Delta\sigma'/\sigma'$)	-
D	trench/channel depth $D = h_1 - h_0$	m
D_{init}	initial trench/channel depth	m
D_{90}	trench/channel depth after 90 days	m
D_{req}	required depth of trench/channel	m
E	compression modulus, bulk modulus	MPa
EEP	excess pore pressure	kPa
F	stability factor of slope stability	-
F_g	force exerted by grains	kN
F_r	Froude number	
F_w	friction force	kN
G	shear modulus	MPa
GC	gravel content of soil	%
H	wave height	m
H_e	equivalent wave height (relative shear stress changes)	m
H_s	significant wave height	m
K	Young's modulus	MPa
K_{fs}	normally distributed factor of erosion length due to a flow slide	-
K_s	decompression modulus (SLIQ2D)	kPa
K_{s0}	decompression modulus at begin of decompression (SLIQ2D)	kPa
$K_{\alpha,\beta}$	slope reduction factor for grain stability as function of flow angle and slope angle	-
L	wavelength	m
$L_{flowslide}$	erosion length due to flow slide	m
N	number of horizontal grid points in SUTRENCH	-
N_{i0}	number of load cycles to cause cyclic liquefaction in undrained conditions	-

$N_{l0;hist}$	number of load cycles to cause cyclic liquefaction after a certain history (compaction due to drainage)	-
N_t	number of time steps	-
NV	number of vertical grid points in SUTRENCH	-
Q	discharge	m^3/s
RD	relative density based on minimum and maximum void ratios	-
Re	Reynolds number	-
T	wave period	s
or	(water) temperature	°
T_e	equivalent wave period (relative shear stress changes) (Mcycle)	s
T_p	peak wave period	s
T_s	wave period belonging to significant wave height	s
V	flow velocity	m/s
V_g	volume of the grains in a soil sample	m^3
V_p	volume of the pores in a soil sample	m^3
V_t	total volume of a soil sample	m^3
W	width of the trench/channel	m
W_{req}	required width of trench/channel	m
Greek		
α_0	approach angle of current to channel axis	°
α_1	angle of current inside channel	°
α_{c-w}	angle between current and wave action	°
α_{br}	breaking coefficient representing influence of breaking waves on the sediment mixing process	-
α_{eq}	'equilibrium' angle of current inside channel	°
α_{c-w}	angle between current and wave action	°
α_s	numerical smoothing coefficient for bed level changes in SUTRENCH	-
$\alpha_{w0;crit}$	critical wave angle to channel axis for which waves are refracted parallel to the channel axis	°
β	ratio sediment mass mixing and fluid momentum mixing coefficients	-
or	compressibility of water	-
β_{init}	angle of side slope	°
$\beta_{down;90}$	angle of downstream side slope after 90 days	°
$\beta_{up;90}$	angle of upstream side slope after 90 days	°
δ	thickness of wave boundary layer near bed	m
ϵ	dielectric permittivity of medium in capacitance measurement	$F/m=C/Vm$
ϵ_0	dielectric permittivity of vacuum = $8,854 \cdot 10^{-12} F/m$	$F/m=C/Vm$
ϵ_f	fluid mixing coefficient	m^2/s
ϵ_{hist}	constant representing 'history effect' of cyclic loading	-
ϵ_s	sediment mixing coefficient	m^2/s
ϵ_{vol_d}	dilatant volume strain	-
$\epsilon_{vol_{dm0}}$	maximum dilatant volume strain	-
γ_{dry}	unit weight of dry soil	kN/m^3
γ_m	shear strain rate of soil-water mixture	1/s
γ_{sat}	unit weight of wet saturated soil	kN/m^3
γ_{unsat}	unit weight of wet unsaturated soil	kN/m^3
γ_w	unit weight of water	kN/m^3
γ_{xy}	shear strain	-
η	coordinate perpendicular to slope; positive into the soil	-

α	coefficient of sediment mixing coefficient; 2 for current alone; 1-2 for current with waves	-
κ	constant of Von Karman	-
α	intrinsic permeability	m^2
λ	eigenvalue of stiffness matrix (static liquefaction)	-
μ	dynamic viscosity	kg/ms
ν	kinematic viscosity	m^2/s
α	Poisson's ratio	-
ϕ	angle of repose	°
α	turbulence damping factor	-
ξ	coordinate along slope	-
α	auxiliary parameter representing friction caused by currents and waves	-
ρ_s	density of solid sediment material	kg/m^3
ρ_{sat}	density of completely saturated soil	kg/m^3
ρ_{unsat}	density of wet, but not completely saturated soil	kg/m^3
ρ_w	density of water	kg/m^3
σ	total stress	kPa
σ'	effective grain stress	kPa
σ_d	single amplitude cyclic axial stress in triaxial test	kPa
σ'_0	initial effective confining stress in triaxial test	kPa
σ'_{v0}	initial effective stress in simple shear test	kPa
$\sigma'_{\text{vol};0}$	isotropic stress at begin of decompression	kPa
τ_b	bed shear stress	kPa
τ_c	current-induced bed shear stress	kPa
τ_{cw}	bed shear stress caused by the combined action of current and waves	kPa
τ_h	double amplitude cyclic shear stress in simple shear test	kPa
τ_w	wave-induced bed shear stress	N/m^2
τ_{xy}	shear stress	kPa
ω	wave frequency ($=2\pi/T$)	rad/s

Greek Capitals

Δ	relative density	-
Ψ_c	stability parameter of movement	-

Appendix C: Soil properties of reference soils

Name	Description	ρ_s kg/m ³	n	e	RD	ρ_{sat} kg/m ³	γ_{sat} kN/m ³	ρ_{dry} kg/m ³	γ_{dry} kN/m ³	γ_{unsat} kN/m ³	ϕ °	c	Su	d ₅₀ µm	d ₉₀ µm	k	κ	C _v	w _s m/s	ν	E MPa
LCS	loosely packed coarse sand	2650	0,45	0,82	0,16	1908	19	1458	15	17	30	0	0	1000	1500	1,9E-04	1,0E-08	3,8E-01	0,1084	0,30	20
MCS	medium packed coarse sand	2650	0,40	0,67	0,43	1990	20	1590	16	18	35	0	0	1000	1500	3,5E-05	1,8E-09	1,2E-01	0,1084	0,35	35
DCS	densely packed coarse sand	2650	0,30	0,43	0,86	2155	22	1855	19	20	40	0	0	1000	1500	1,2E-05	5,5E-10	5,9E-02	0,1084	0,40	50
LMS	loosely packed medium sand	2650	0,45	0,82	0,16	1908	19	1458	15	17	30	0	0	500	750	1,4E-05	7,5E-10	2,9E-02	0,0663	0,30	20
MMS	medium packed medium sand	2650	0,40	0,67	0,43	1990	20	1590	16	18	35	0	0	500	750	8,8E-06	4,4E-10	3,1E-02	0,0663	0,35	35
DMS	densely packed medium sand	2650	0,30	0,43	0,86	2155	22	1855	19	20	40	0	0	500	750	3,0E-06	1,4E-10	1,5E-02	0,0663	0,40	50
LFS	loosely packed fine sand	2650	0,45	0,82	0,16	1908	19	1458	15	17	30	0	0	200	300	2,3E-06	1,2E-10	4,6E-03	0,0252	0,30	20
MFS	medium packed fine sand	2650	0,40	0,67	0,43	1990	20	1590	16	18	35	0	0	200	300	1,4E-06	7,1E-11	5,0E-03	0,0252	0,35	35
DFS	densely packed fine sand	2650	0,30	0,43	0,86	2155	22	1855	19	20	40	0	0	200	300	4,7E-07	2,2E-11	2,4E-03	0,0252	0,40	50
LSI	loosely packed silt	2650	0,45	0,82	0,16	1908	19	1458	15	17	27	0	0	50	75	1,4E-07	7,5E-12	2,9E-04	0,0029	0,30	20
MSI	medium packed silt	2650	0,40	0,67	0,43	1990	20	1590	16	18	32	0	0	50	75	8,8E-08	4,4E-12	3,1E-04	0,0029	0,35	35
DSI	densely packed silt	2650	0,30	0,43	0,86	2155	22	1855	19	20	37	0	0	50	75	3,0E-08	1,4E-12	1,5E-04	0,0029	0,40	50
SOC	soft (organic) clay, low plasticity	2650	0,75	3,00		1413	14	663	7	11	18	5	5	2	3	3,8E-09	2,7E-13	7,6E-07		0,20	2
MEC	medium clay, medium plasticity	2650	0,60	1,50		1660	17	1060	11	14	18	7	20	2	3	9,0E-10	5,4E-14	4,5E-07		0,30	5
STC	stiff clay, high plasticity	2650	0,40	0,67		1990	20	1590	16	18	20	10	50	2	3	1,4E-10	7,1E-15	1,4E-07		0,40	10

