SANDPIT

SAND TRANSPORT AND MORPHOLOGY OF OFFSHORE SAND MINING PITS

Process knowledge and guidelines for coastal management

The European Commission
Community Research
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OF
OFFSHORE SAND MINING PITS

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for coastal management

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PREFACE

The coastal zones of Europe are of strategic importance in terms of social, economic and ecological activities. Many of these coastal zones suffer from severe erosion, requiring the nourishment of large quantities of sediment to mitigate the erosion problems. Furthermore, large-scale reclamation of new land for growing populations and for industrial activities (extension of seaports and airports) requires massive extraction of sediments.

However, as the number and intensity of human uses in European coastal zones increase, both the natural resource base and the social structure of the coastal zone are being irreversibly degraded. Widespread coastal erosion, provision of adequate flood defences, and increasing pressures for navigational access, as well as habitat destruction, loss of biodiversity, contamination of soil and water resources, are problems common to many parts of Europe, where the potential of the coastal zone continues to be reduced.

In addition, the Coastal Zone is probably the area where the consequences of Global Climate Change (sea level rise, increase in storms frequency and intensity) are likely to be increasingly felt in the future.

The SANDPIT project fostered an enhanced collaboration and combination of expertise, efforts and resources of 17 institutes from 7 European countries. It provides a typical example of how the "European added value" of the RTD projects funded by the European Union can best be applied for successfully addressing complex research problems.

SANDPIT has broken new frontiers in some areas related to the understanding of marine sediment extraction and the development of methods, models and formulae for predicting morphological development in coastal and estuarine waters.

In the true spirit of the European Union's RTD Framework Programmes, the partners of the SANDPIT project through the publication of this book, are aiming to disseminate their results to the widest possible community. We hope that the potential users of these results (researchers, practitioners, port authorities, coastal/harbour engineers, governments authorities, … just to mention a few) will test the validity of the methods and the tools that are being presented hereby and contribute constructively to their possible improvements for the benefit of the entire society.

In concluding, I would like to take this opportunity to thank all SANDPIT partners for their commitment, hard work and open collaborative spirit throughout the duration of the project.

Alan Edwards
EU-Coordinator SANDPIT Project

ACKNOWLEDGEMENTS

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Delft Hydraulics is gratefully acknowledged for their financial support to publish this book.
All participants are thanked for their contributions to this book.

PART I

SUMMARY OF PROJECT RESULTS

1. INTRODUCTION

1.1 Sand mining problems

The demand and production of aggregates (sand and gravel/shingle) in Europe is continuously growing and has reached a level of nearly 10 tonnes per person per year in northwestern Europe and probably is the second largest commodity after water. The demand for these aggregates has been met traditionally from land-based pits, but in recent years offshore sources have made an increasingly important contribution.
Massive mining of sand from the middle and lower shoreface (depths of 10 to 30 m) in large-scale mining and extraction pits/areas will be required in future in many European countries. For example, around the North Sea and the Mediterranean Sea, mining of sand will be required to nourish beaches and coastal dunes in response to increased coastal erosion due to the expected sea level rise. Furthermore, the large-scale reclamation of land and the construction of large-scale artificial islands (for industrial purposes; ports and airports) in coastal seas which are presently being considered, will also require huge amounts of sand as building material.
Given the scale of these undertakings, the volume of sand required in the near future (10 to 20 years) will be of the order of 100 to 1000 million m^3 per country surrounding the North Sea.
To meet these demands, the existing areas for mining of sand need to be extended considerably and new potentially attractive areas should be explored and exploited. Massive mining of sand may take place by dredging in artificial sand pits or channels (also navigation channels) or by removal (dredging) of existing large-scale sand banks/shoals in the offshore zone (middle and lower shoreface).
Large-scale mining pits will have a significant impact on the near-field and far-field (up to the coast) flow and wave patterns; the flow velocities inside the pit will be reduced and the wave heights may also be reduced, depending on the depth of the pit. As a consequence, the sand transport capacity inside the pit will decrease and sediments will settle in the pit area, resulting in deposition. Thus, the pit will act as a sink for sediments originating from the surrounding areas and depending on the local flow and wave patterns. Hence, erosion of the sea floor will take place in the (immediate) surrounding of the pit. This may lead to a direct loss of sediment from the nearshore zone (beaches, see Figure 1Top). Indirect effects result from the modification of the waves moving and refracting over the excavation area (pit), which may lead to modification of the nearshore wave conditions (wave breaking) and hence longshore currents and sediment transport gradients and thus to shoreline variations (see Figure 1Bottom). Considering the massive scale of future mining of sand and hence the large spatial scales that will be affected by the mining activities, the mining areas need to be situated in the offshore shoreface zone to minimise the effects of nearshore coastal erosion. On the other hand the mining of sand will be progressively more expensive at greater distances from the shore. Research is required to find the optimum solution between the effect on the coast and the costs of mining.
The technical and environmental evaluation of sand mining activities requires fundamental knowledge of morphological processes, sand transport processes, sand budgets and ecology in the offshore coastal zones. The accurate determination of offshore and nearshore sand budgets is a primary problem in Coastal Zone Management for many of the European coasts. Closure of sand budget estimates for the sand mining area considered requires rather accurate information of sand transport rates at the boundaries. Often the accuracy of the computed sand transport rates is not sufficient, because of the application of relatively simple sand transport models. Furthermore, model verification and validation based on detailed field data is lacking in most cases. The main reason for this is that detailed field data sets of sand transport in offshore (deep water) conditions are scarcely available.
Guidance for the management of marine sediment extraction is given by the International Council for the Exploration of the Sea (ICES, Palægade 2-4 DK-1261 Copenhagen K Denmark). ICES established a Working Group on the effects of marine sand and gravel extraction on fisheries in 1972. Later it was renamed Working Group on the effects of extraction of marine sediments on the marine ecosystem, which produced various reports (1992, 2001), a Code of Practice and guidelines for management.

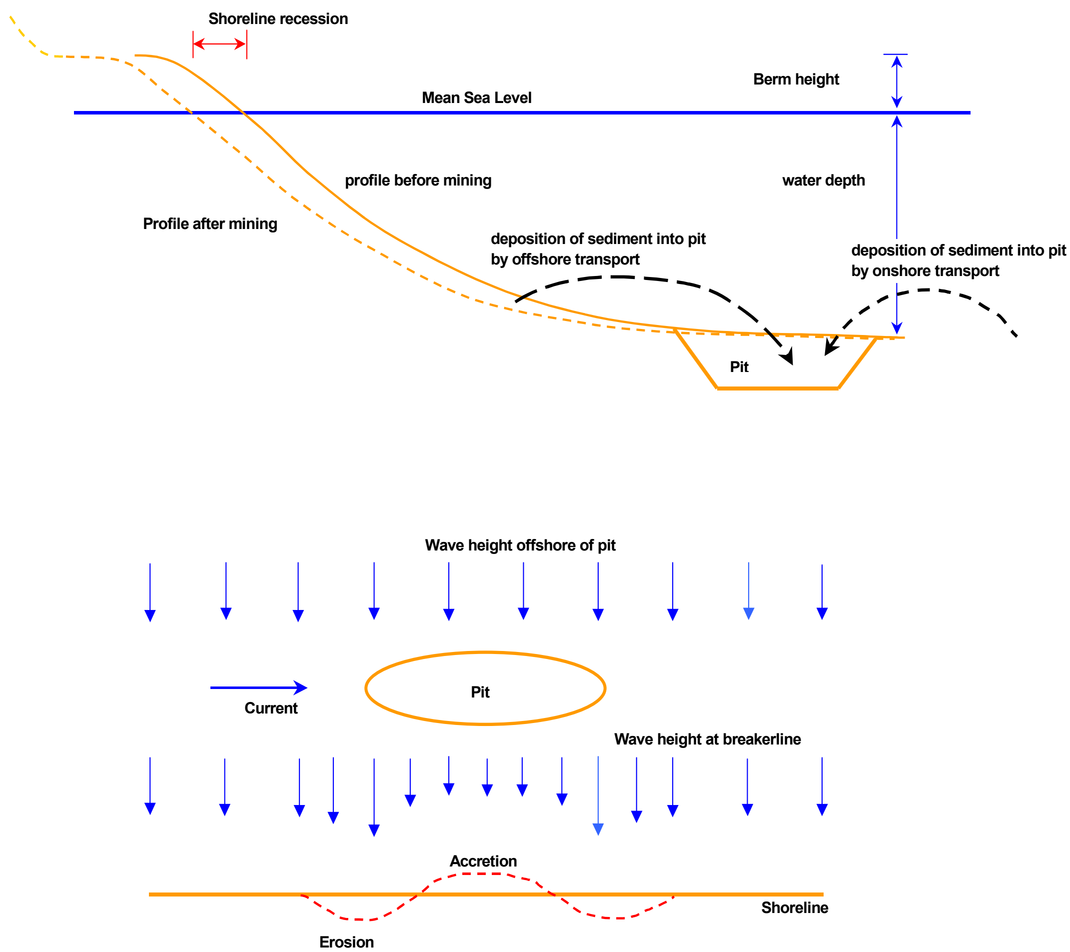

Figure 1.1.1 *Effects of sand mining pit on shoreline*
Top: Cross-shore effects
Bottom: Longshore effects

1.2 Science in support of coastal management

An important aspect in Coastal Zone Management (CZM) is the presence of relevant and reliable information. Coastal science can play an important role in providing this information. However, the question how specialist knowledge can be translated to coastal management is far from trivial. From this point of view the most challenging objective of the SANDPIT Project was 'to deliver validated modelling tools and methodologies for their use in a form suitable for coastal management'.
Coastal managers often feel that it is difficult to apply the results of research or studies to policy formulation and practical management. For a specialist in coastal behaviour, relevant knowledge might be a set of equations or a model that better describes coastal morphology under currents and waves (information about reality). For a coastal manager, relevant knowledge might be a decision recipe for the design of sand mining pits (information for reality). The former knowledge may be used to improve the latter. In practice such benefits of specialist knowledge are often not or hardly recognised. This difficulty of specialist answers not providing the information needed to solve coastal management problems, is caused by a 'gap' between coastal management and coastal science. Bridging the gap requires effective communication.

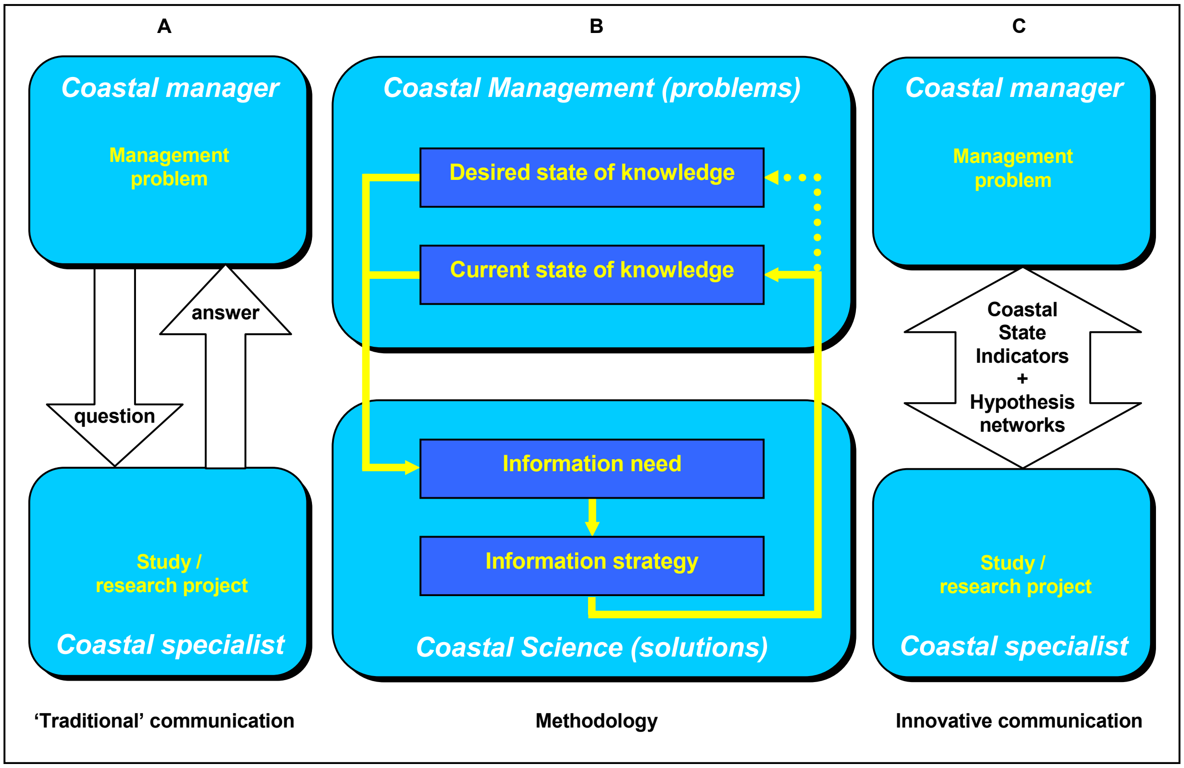

Figure 1.2.1 *Bridging the gap between coastal science and coastal zone management*

Communication

The traditional communication model (Figure 1.2.1Left) shows four distinct elements in the communication process:

- **problem** : the problem of CZM, usually defined at a higher level of aggregation,
- **question** : downward communication of a request for information,
- **study** : the (sub)problem to be studied by the specialist, and
- **answer** : the upward communication of the answer to the generalist.

In practice, whenever there is a situation where the specialist answer is not, or is only marginally, useful in dealing with the coastal management problem, there is a tendency toward blaming the other party. Specialists are often accused of a lack of consideration for the potential customers of their research and an assessment of their needs at an early stage. Coastal managers are often accused of not having a clear image of the information need and not being able to formulate clear questions. These opposing views indicate the need for a constructive methodological approach to match coastal science with coastal management needs. On the one hand such an approach should support the enduser in formulating the information need as clear as possible. On the other hand it should support the specialists in designing practical studies and translating their findings into useful coastal management information. This means that both coastal managers and specialists need to make a serious effort to communicate more effectively. The methodology, summarised in Figure 1.2.1, is explained hereafter in more detail.

Methodology

Given a CZM problem, coastal managers, as consumers of specialist advice, usually require information on aspects of coastal behaviour that they do *not* yet fully understand and on ways to deal with this behaviour. Although coastal managers may have a clear view of reasons *why* they need specific information, a vision of the required content and form often is much more vague. This is more true when the CZM problem at hand is more complex. Because the information required for the solution of these problems is not known accurately beforehand, specialists contracted to deliver the results consequently find room for interpretation. On the one hand this flexibility supports the innovative process. On the other hand it involves the risk that the results mismatch the needs of endusers in solving the problem.

Based on this, a new methodology for communication is proposed, as follows:

Problem setting

Desired state of knowledge

To overcome the above-described difficulty we need to shift the focus from problem solving to problem setting. Therefore a first step in the methodology is to inventory the knowledge that is required for dealing with the problem at hand. To avoid omitting crucial aspects, it is best to start the analysis from a broad and strategic perspective.

Current state of knowledge

However vague the vision of a coastal manager may seem, it is based on a sort of tacit knowledge of the system. Externalisation of these (tacit) mental models in a kind of mobilisation process is a key factor in bridging the gap.

Hypotheses and state indicators

To stimulate the communication process it is very important to make sure that explicitation of tacit knowledge is not limited to abstract statements. The problem owner should be challenged to formulate representative hypotheses on how the system works and to define quantitative management objectives expressed in terms of representative coastal state indicators (CSIs) including reference values for these indicators. A proposed procedure is to interview coastal managers. The resulting hypotheses on how the system works and how it should be managed may then be improved in interaction with specialists. Sub-hypotheses requiring fundamental research may be formulated at the most detailed level.

Study

Information need

When there is agreement on the relevant indicators, reference values and hypotheses, comparing the desired state of knowledge with the current state of knowledge yields the information that is needed in dealing with the initial problem.

Information strategy

When the information need is clear, constraints like available time, money, tools and experts determine a feasible information strategy. The information resulting from study and/or research should then be translated back to the level of indicators and hypotheses. This translation process may require close interaction with end users for an optimal transfer of relevant knowledge. Increased insight then alters the current state of knowledge, and possibly the desired state of knowledge of the enduser. Renewed confrontation of both may or may not result in additional information requirements.

Conclusion

Explicitation in terms of hypotheses and CSIs helps endusers in defining their information need and helps specialists in matching their knowledge with these needs. As such it is a promising methodology for bridging the gap between coastal science and coastal management. This approach implies an 'innovative' communication model (Fig. 1.2.1Right), showing three elements (in stead of the 'traditional' four):

Problem setting : analysis of CZM objectives within their context;
Hypotheses : define and exchange hypotheses on relevant Coastal State Indicators (CSIs);
Study : study and research to describe CSI's and falsify relevant hypotheses.

1.3 Sandpit Project

Objectives and methodology

The **overall objective** of the SAND PIT project is to develop reliable prediction techniques and guidelines to better understand, simulate and predict the morhological behaviour of large-scale sand mining pits/areas and the associated sand transport processes at the middle and lower shoreface and the surrounding coastal zone.
For the achievement of the overall objectives the following **route** has been followed:
I Definition of Problem (WP2)
II Formulation of Enduser questions, Research questions and Hypotheses (WP2)
III Definition of Information need and Strategy (WP2)
(What is known; what is unknown and what is needed?)

- determination of State of the Art (processes and modelling);
- selection of appropriate models;
- selection of appropriate output parameters relevant to endusers (coastal state indicators);
- determination of accuracy of models;
- definition of benchmarking test cases;
- evaluation of results;
- improvement of models;

IV Formulation and execution of research plans based on research questions (WP2)

- execution of new laboratory and field experiments (WP3);
- determination and implementation of new formulations (improved models, WP4);

V Evaluation of new experimental data and improved models (WP4)

- based on existing benchmarking test cases;
- based on new experimental results (laboratory and field);

VI Application of improved models to answer enduser questions (WP2)

- application of models to specific cases defined by endusers (WP2);
- execution and analysis of sensitivity runs;
- quantification of results in terms of state indicators;

VII Conclusions (WP2)
An imporant achievement of the Project is that special purpose-designed field measurements at a site in the North Sea have been performed for better understanding of shoreface conditions and proper validation of numerical models, with adequate boundary conditions. The focus was on hydrodynamics and sediment dynamics. Innovative techniques have been used in the field experiments by the researchers of various institutes. Numerical modellers have worked interactively with the field researchers.
Participants from various National Authorities (endusers) ensured that the project was focussed on practical tools for coastal zone management.
The basic **research topics** of the project are:
Research on sand transport

- execution of field and laboratory experiments to fill missing gaps of existing databases (WP3),
- improvement of detailed research sand transport models (WP4),
- improvement of practical sand transport models using the results of field/laboratory experiments and of detailed process models (WP4),
- implementation of practical sand transport models in a community sand transport model (WP4),
- implementation of community sand transport model in morphodynamic models,

Research on morphology

- application of improved morphodynamic models to evaluate the morphological behaviour of sand bank at middle and lower shoreface (before and after removal of bank), (WP4),
- application of improved morphodynamic models to evaluate the behaviour of sand mining pits at middle and lower shoreface (WP4),

Practical guidelines

- integration of all results into practical guidelines and tools for use by various types of end-user, such as coastal managers, consultants, contractors and modellers.

Participants

The project team in SANDPIT comprised endusers, field experimenters and researchers from various leading research institutes and universities in Europe. The team had an established strength and a balanced composition, linking expertise in basic physical processes (experimental and modelling research) to that in coastal engineering applications.
The endusers, field experimenters and researchers involved in the project are given in the following table.

List of organisations

Code	Country	Organisation
DH	NL	WL | Delft Hydraulics
UT	NL	University of Twente
RWS	NL	Nat. Inst. Coastal and Marine Management , Rijkswaterstaat
UU	NL	University of Utrecht
HR	UK	Hydraulic Research Wallingford
UEA	UK	University of East Anglia
UAB	UK	University of Aberdeen
UWB	UK	University of Wales, Bangor
DHI	DK	DHI-Institute of Water and Environment
SINTEF	NO	Norwegian Applied Mathematics
NTNU	NO	Norwegian Univ. of Science and Technology
SGH	FR	SOGREAH
UCAE	FR	University of Caen
CET	IT	CETMEF
UGE	IT	University of Genova
UCAT	IT	University of Catania
IMAR	PT	University of Coimbra

List of participants

Workpackage 2 **Guidelines and Tools for Management** Leader: R. Soulsby (HR)	**Workpackage 3** **Data Sets and Experiments** Leader: P. Hoekstra (UU)	**Workpackage 4** **Development, improvement and application of predictive tools (Mathematical Modelling)** Leader: A.G. Davies (UWB)
L. van Rijn (DH)	L. van Rijn (DH)	D. Walstra (DH)
D. Walstra (DH)	J. van der Werf (UT)	R. Uittenbogaard (DH)
M. de Vries (DH)	M. Boers (RWS)	J. Ribberink (UT)
J. Mulder (RWS)	P. Hoekstra (UU)	J. van der Werf (UT)
D. Willems (RWS)	B. Grasmeijer (UU)	B. Grasmeijer (UU)
P. Hoekstra (UU)	M. Kleinhans (UU)	R. Soulsby (HR)
R. Soulsby (HR)	A. Brampton (HR)	T. Chesher (HR)
A. Brampton (HR)	C. Vincent (UEA)	A. Davies (UWB)
J. Birklund (DHI)	T. O'Donoghue (UAB)	J. Malarkey (UWB)
P. Sauvaget (SGH)	M. Monteil (SGH)	H. Jensen (DHI)
M. Monteil (SGH)	O. Montfort (UCAE)	K. Eidsvik (SINTEF)
M. Piqueret (CET)	P. Sergent (CET)	D. Myrhaug (NTNU)
C. Santoro (UCAT)	E. Foti (UCAT)	L. Holmedal (NTNU)
F. Levoy (UCa)		C. Guilbaud (SGH)
O. Montfort (UCa)		R. Walther (SGH)
		G. Vittori (UGE)
		F. Sancho (IMAR)
		A. Temperville (IMAR)

Organisation of project

The inter-relationships between the work packages are shown in Figure below.

WORKPACKAGE 1
PROJECT MANAGEMENT AND DISSEMINATION OF RESULTS

PARTNER DH and all others

⇑⇓ ⇑⇓ ⇑⇓

WORKPACKAGE 2
GUIDELINES AND TOOLS FOR MANAGEMENT OF SHOREFACE MORPHOLOGY

PARTNERS: All

⇑⇓ WP 3.1, 3.2, 3.3, 3.4

⇑⇓ WP 2.1, 2.2, 2.3, 4.5, 4.6

WORKPACKAGE 3

DATA SETS ON SAND TRANSPORT AND MORPHOLOGY

PARTNERS
DH
UT
RWS
UU
HR
UEA
UAB
UCAE
CET
UGE
UCAT

⇔
WP 4.1, 4.2, 4,4, 4.5

WORKPACKAGE 4

DEVELOPMENT, IMPROVEMENT AND APPLICATION OF PREDICTIVE TOOLS

PARTNERS
DH
UT
HR
UWB
DHI
SINTEF
NTNU
SGH
CET
UGE
UCAT
IMAR

Web site
http://sandpit.wldelft.nl

2. MANAGEMENT QUESTIONS, RESEARCH QUESTIONS AND RELATED PROJECT METHODOLOGY

2.1 Project methodology

For the achievement of the overall objectives of the SANDPIT Project the route as explained in section 1.3 has been followed:

I Definition of Problem (WP2)
II Formulation of Enduser questions, Research questions and Hypotheses (WP2)
III Definition of Information need and Strategy (WP2)
IV Formulation and execution of research plans (WP2, WP3 and WP4)
V Evaluation of new experimental data and improved models (WP3 and WP4)
VI Application of improved models to answer enduser questions (WP2)
VII Conclusions (WP2)

2.2 Basic Coastal Zone Management (CZM) problem and context

2.2.1 Optimum design of large scale sand mining

The coastal zone is subject to increasing pressure due to increased sea level rise combined with increasing human activities, illustrating the pressure from both seaward and landward sides. Finding optimal solutions is a major challenge to Coastal Zone Management (CZM).
An illustrative example of the general problem is the increasing demand for marine sands for purposes such as: coastal maintenance and protection, recreational beaches, onshore infrastructure and building industries, as well as offshore land reclamation and infrastructure.
The 'traditional' method of sand mining over relatively shallow depths of a few meters (e.g. for the Netherlands the presently maximum allowable depths of sandpits is 2 m) is considered to be insufficient to cover future sand needs. Increasing costs of sand dredging operations related to transport distances, induce questions on the possibilities of mining from 'local' sources of sand.
Leading to the general CZM problem:

- What is the optimal design of large scale sand mining operations?

Design options to be considered are location, dimension, shape and orientation of the pit as well as excavation method. The scale of sand mining concerns 10's to 100's of million cubic meters per location. The design **optimum** is depending on the specific context of sand mining which determines acceptable limits to an impact.

2.2.2 Context of sand mining

From a Coastal Zone Management perspective, the context of sand mining is determined by:
- the physical context (i.e. the physical- and ecological environment of the sand mining area);
- the socio-economic context (i.e. socio-economic functional uses of the coastal zone like flood protection, navigation and harbours, fisheries, aquaculture, recreation, ecological values etc.); and
- the administrative context (i.e. the institutional arrangements, regulations, legislations and directives).

2.3 Coastal Zone Management questions and Indicators

A rational Coastal Zone Management (CZM) will be based on an integrated analysis of this context. Thus, related to sand mining the overarching CZM question is:

- What are the effects of large scale sand mining on the physical and ecological environment, on the functional uses and on institutional arrangements; and vice versa ?

A rational CZM will aim for a rational decision making process that is transparent and reproducible. The (Vague) strategic CZM objectives must be translated into (concrete) operational objectives. Important aid in this process is the definition of **indicators**: *well defined physical or ecological variables as components of one or more institutional arrangements or/and quantifying a socio-economic functional use.*
Some examples of CZM-**indicators** for different functions, so called Coastal State Indicators (CSI) as developed in the Netherlands, are given in Table 2.1. In a similar way, the UK recently has started to develop so called Hydrodynamic Indices.

FUNCTION	COASTAL STATE INDICATORS - (NL)
Coastal maintenance	• Coastline position (MCL) • Minimum erosion profile • Design wave height • Sand budget per coastal cell (NAP-20m to Dune foot)
Recreation	• Beach width • Beach quality (sediment characteristics)
Offshore infrastructure	• Minimum coverage of cables • Minimum width of cable trench
Navigation	• Minimum depth and width of shipping channel • Maximum variations in current along shipping channel
Ecosystem	• Areas of habitats (subject to Habitat and Bird Directives)

Table 2.1 *Some examples of* ***CZM-Indicators***

SANDPIT: from CZM problems to CZM questions

Using the *indicator*-concept, the overarching CZM question related to sand mining can be (re)phrased as:

- What are the probable effects of large scale sand mining on the specified CZM-Indicator(s) ?

However, for various focus areas within the CZM context, the characteristic CZM-indicators are different. Thus, an analysis of the CZM question has led SANDPIT to the definition of a generic set of sub-questions (or middle level questions) both concerning Near Field effects and Far Field effects.

The Near-Field and Far Field problems for CZM are defined as:

- **Near Field**: ***Navigation, Fisheries, Offshore Structures and a Sustainable Ecosystem;***
- **Far Field**: ***Flood Protection, Coastal Maintenance, Fisheries and a Sustainable Ecosystem.***

Concentrating on the **Near Field** the following set of middle level questions are distinguished:

1 Will an offshore mining pit modify the local flow and wave fields in such a way that the transport regime and the large-scale bed forms (sand banks) in the direct vicinity are influenced? Can the magnitude of an effect be quantified ?
2 Will the mining pit act as a sediment sink and thereby have a particularly marked impact on the sea bed immediately adjacent to the pit ? Can the magnitude and extent of this impact be quantified? How can this impact be minimised?
3 Will sand banks that have been mined away, return within a period of 50 years, where does the sand come from and will it affect other nearby sand banks?
4 Will sand mining destroy the local benthic organisms which may be environmentally and commercially important? Can the effect on and recovery of these organisms be quantified? Are any mitigation measures possible?

Concentrating on the **Far Field** the following set of middle level questions are distinguished:

5 Will a large-scale mining pit affect the overall tide- and wind-induced flow regime in a coastal sea including nearby tidal inlets? Can the effect be quantified ?

6 Will the mining pit allow more wave energy to reach the coastline through mechanisms such as reducing nearshore wave limiting conditions (e.g. by removing nearshore sand banks) or by acting as a lens to focus wave energy on the coastline? Can the magnitude and extent of this impact be quantified and to what degree of accuracy?

7 Will an offshore mining pit (removal of sand below bed or removal of sand banks) act as a sediments sink and what impact will it have on nearshore sediment transport regimes and will it lead to increased coastal erosion? Can the magnitude and extent of this impact be quantified and to what degree of accuracy?

8 Will the fines (mud) released during large-scale mining process affect the overall budget of fines in the surrounding seas and the import of fines into adjacent tidal inlets and back-barrier basins? Will it result in an increase of turbidity affecting oyster farming or spawning areas ?

9 What measures can be taken to minimise the negative impacts that sand mining may have on the coastline? Can parameters such as minimum dredge depth and distance from the shore be defined ?

10 Measures to minimise the negative impacts of sand mining on the coastline will have cost implications. How does the cost per m^3 of sand depend on the location (depth and distance from shore) and dimensions of the pits?

Each of these middle level questions may be extended with the question on what **time scales** (short term / long term) a potential effect may be of importance.

SANDPIT focus

Considering the main expertise of its partners, the SANDPIT project will mainly focus on the physical aspects of sand transport. Efforts on aspects of fine sediments will be limited. Ecological aspects will be studied on the basis of a literature review only. An economical assessment of cost has not been scheduled. The development of acceptability criteria for any assessed effect, is considered to be outside the scope of SANDPIT. Considering the availability of data, SANDPIT will concentrate on the design options related to the location.
Thus, SANDPIT will focus on the central problem:

- **How does the location of sand mining (in terms of water depth and distance from the shore) influence the various effects of large scale sand mining ? Give a quantitative estimate of probable effects.**

This focus implies a concentration on the following CZM questions:

1 Will an offshore mining pit modify the local flow and wave fields in such a way that the transport regime and the large-scale bed forms (sand banks) in the direct vicinity are influenced? Can the magnitude of an effect be quantified ?

2 Will the mining pit act as a sediment sink and thereby have a particularly marked impact on the sea bed immediately adjacent to the pit ? Can the magnitude and extent of this impact be quantified? How can this impact be minimised?

5 Will a large-scale mining pit affect the overall tide- and wind-induced flow regime in a coastal sea including nearby tidal inlets? Can the effect be quantified ?

6 Will the mining pit allow more wave energy to reach the coastline through mechanisms such as reducing nearshore wave limiting conditions (e.g. by removing nearshore sand banks) or by acting as a lens to focus wave energy on the coastline? Can the magnitude and extent of this impact be quantified and to what degree of accuracy?

7 Will an offshore mining pit (removal of sand below bed or removal of sand banks) act as a sediments sink and what impact will it have on nearshore sediment transport regimes and will it lead to increased coastal erosion? Can the magnitude and extent of this impact be quantified and to what degree of accuracy?

SANDPIT approach

The selected (middle level) CZM questions represent the basis for defining a set of detailed research questions referring to underlying process parameters. The process parameters will be the subject of quantitative assessment by measurements (WP3) and modelling (WP4) within SANDPIT. The final results will be integrated as much as possible in terms of CZM-**Indicators**, to answer the overarching CZM question (WP2).
Quantitative estimates from SANDPIT may provide a starting point for future discussions on acceptability criteria by CZM authorities.

2.4 Formulation of research questions

The CZM questions are translated into various research questions to be addressed in the SANDPIT project.

CZM questions Near Field	Research questions	Indicators
1. Will an offshore mining pit modify the local flow and wave fields in such a way that the transport regime and the large-scale bed forms (sand banks) in the direct vicinity are influenced?	1) what is the change in maximum tidal current velocity due to the presence of a dredged pit (or dredged sandbank) of given size? 2) What is the change in wave height during a storm due to the presence of a pit of given size? 3) what are the effects of modified flow and wave conditions on the local sand transport capacity? 4) what is the influence area?	1) maximum current velocity 2) maximum wave height 3) net annual sand transport rates
2. Will the mining pit act as a sediment sink and thereby have a particularly marked impact on the sea bed immediately adjacent to the pit. Can the magnitude and extent of this impact be quantified? How can this impact be minimised?	1) what is the sand transport regime in relation to the current and wave regime outside the dredged pit (or dredged sandbank)? 2) what is the effect of (modified) bed forms and (modified) particle size on the sand transport regime outside the pit? 3) what is the gross and net annual sand transport outside the pit? 4) what is the amount of sand trapped in a pit of given size per year and over 50 years? 5) what is the erosion on the flanks of the pit per year and over 50 years? 6) what are the net migration rates in longshore and in cross-shore direction? 7) what should be the location and dimensions of the pit to minimise these effects?	1) annual infill volume 2) erosion length scales 3) net migration rates in longshore and in cross-shore direction 4) optimum pit dimensions; distance to shore

CZM questions Far Field	Research questions	Indicators
5. Will a large-scale mining pit affect the overall tide- and wind-induced flow regime in a coastal sea including nearby tidal inlets?	1) what is the effect of a large-scale mining pit of given size (or dredged sand bank) on the nearshore tide-induced and wind-induced currents? 2) what is the influence area of a large-scale dredged pit (or dredged bank) on the flow regime?	1) tidal ranges 2) peak tidal currents 3) maxium wind-induced currents
6. Will a large-scale mining pit allow more wave energy to reach the coastline through mechanisms such as reducing nearshore wave limiting conditions (e.g. by removing nearshore sand banks) or by acting as a lens to focus wave energy on the coastline? Can the magnitude and extent of this impact be quantified and to what degree of accuracy?	1) what is the effect of a large-scale dredged pit (or bank) of given size on the nearshore wave field? 2) what is the influence area of a large-scale mining pit (or bank) on the wave climate?	1) nearshore wave spectrum (edge of surf zone; -8 m and –5 m depth contours)
7. Will an offshore mining pit (removal of sand below bed or removal of sand banks) act as a sediments sink and what impact will it have on nearshore sediment transport regimes and will it lead to increased coastal erosion? Can the magnitude and extent of this impact be quantified and to what degree of accuracy?	1) what is the effect of a dredged pit or sand bank on the gross and net annual longshore sand transport in the surf zone? 2) what is the effect of a dredged pit or sand bank on the gross and net annual crosshore sand transport at edge of the surf zone ? 3) what are the changes in net annual cross-shore and longshore transport rates in relation to the sand volume in the nearshore zone on a coastal length scale comparable to the scale of the mining area? 4) what is the optimal location and what are the optimal dimensions of the mining area (pit or sand bank) to minimise its effect on the coastline? 5) How far offshore must a mining area be situated before its effect on the coast can be ignored?	1) net annual longshore transport in surf zone 2) net annual cross-shore transport at edge of surf zone (–8 m andd –5 m depth contours) 3) sand volume in predefined zone 4) shoreface slope between –8 m depth contour and coastline 5) beach width

The CZM-questions can be mapped onto the project subworkpackages, as shown in Table 2.2.

	Activity No.—>	2.1	2.2	2.3	2.4	2.5	3.1	3.2	3.3	3.4	3.5	4.1	4.2	4.3	4.4	4.5	4.6
Q. No.	User question																
1	Modification to local flow and wave fields?		X	X		X	X	X		X	2.5		4.4		4.5	X	X
2	Effect on sea bed adjacent to pit? How to minimise?	X	X			X	X	X	X	X	2.5	4.3, 4.4	4.1, 4.4	4.4	4.6		X
5	Modification of far-field flow and wave fields?		X	X		X	X				2.5		4.4		4.5	X	X
6	More wave energy at coastline? Magnitude & extent?		X	X		X	X				2.5		4.4		4.5	X	X
7	Impact on nearshore transport & coastal erosion?	X	X	X		X	X	4.1	4.1		2.5	4.3, 4.4	4.1, 4.4	4.4	4.6		X
		Mean sand transport rates/budgets	Optimum location/design of mining areas	Use of sandbanks	Ecological effects	Synthesis/Database / Guidelines	Inventory of existing field data	North Sea test pit	North Sea sand transport measurements	Laboratory measurements	Data banking	Improve sand transport models	Improve bedform & roughness models	Community sand transport model	Improved morphodynamic models	Model scenarios for sand-bank mining	Model scenarios for sand-pit mining

KEY **X** = this activity directly addresses this user question

x.y = this activity indirectly addresses this question by providing input to activity x.y

Table 2.2 *CZM questions in relation to project activities*

2.5 Information strategy

To identify the proper research needs and desired results, the following topics should be addressed:

- what are the present-day national practices and regulations with respect to offshore mining of sand?
- what is the present 'state of the art' of morphology and associated sand transport at the shoreface and what are the missing research topics that need to be addressed?
- what is the present 'state of the art' of mathematical modelling and accuracies involved of hydrodynamic and morphodynamic processes at the middle and lower shoreface?

The assessment of the model inaccuracy will be performed, as follows:

- define a series of laboratory and field bench marking tests (with data of processes outside and inside the pit) including the inaccuracy of the data sets involved;
- apply the selected models to the selected benchmarking tests;
- evaluate the inaccuracy of the models in relation to the inaccuracy of the benchmarking data;
  - inaccuracy of the input data of the models (magnitude and direction of waves, currents, sediment transport at boundaries; sediment and bed form characteristics at boundaries);
  - inaccuracy of model formulations (wind-current interaction, wave-current interaction, bed load versus suspended load; effect of small-scale bed forms, etc.;
  - inaccuracy due to modelling approach (accumulation of errors; process-based models versus behaviour-oriented models);
- improve the model formulations if the models are not sufficiently accurate;
- define new laboratory and field experiments if the existing data sets are not sufficiently accurate;
- apply the improved models to the new experimental results.

The present-day national practices and regulations are addressed in Chapter 3.
The definition of the 'State of the Art' and the definition of benchmarking data sets based on existing data will be addressed in Chapter 4.
The collection and analysis of new datasets is addressed in Chapter 5.
The model inaccuracies and model results are addressed in Chapter 6.
Based on this, guidelines are derived and discussed in Chapter 7.

3. NATIONAL PRACTICES AND REGULATIONS IN THE EXTRACTION OF MARINE SAND AND GRAVEL

3.1 Introduction

To answer the end-users' questions formulated in Chapter 2, first of all it is essential to have detailed information about the diverse existing national practices and regulations. The countries of Europe have all evolved different methods of regulating the extraction of sand and gravel in their coastal waters, to meet their individual needs. At an early stage of the SANDPIT research project it was decided to collate such information from the countries represented in the project. To this end, the partners in the project have drawn together information from their own countries regarding quantities, regulations, practices and experiences of offshore extraction of sand and gravel. These are purely for comparative purposes, and should not be taken to represent national or EU policies. However, when taken together with the results of the project, they could lead to recommendations for a more consistent approach to the regulation of such activities applicable throughout the EU.

3.2 Overview of regulations

The individual project scientists nominated for the UK, Netherlands, Denmark, Italy, Norway and France collected information in a standardised format, which is presented in individual national tables in Tables 3.2.1 to 3.2.6. The headings include:

- Present and future demands for marine sand and gravel
- The purposes to which it is put
- Present and potential extraction locations
- Monitoring and studies undertaken
- Authorities involved in regulation
- The consultation procedure
- Hydraulic and morphodynamic evaluation methods used
- Ecological evaluation methods used
- Regulations and criteria applied to licence applications
- Existing experience from extraction sites and monitoring.

Examination of the tables shows both similarities and differences. Those types of information that can be readily compared have been drawn together into a set of summary tables (Tables 3.2.7 to 3.2.10), listing for all the countries:

- Volumes extracted (demand, type, use);
- Regulations (authorities involved, consultation procedure);
- Evaluations (hydraulic, morphodynamic, ecological); and
- Criteria and experiences.

The current volumes extracted annually by each country range between 3 and 32 Mm^3 per annum, with estimates of future extraction per country ranging between 200 and 3000 Mm^3 over the next 50 years. The proportions of sand and gravel vary considerably, from 100% sand to 60% gravel, depending on both the availability and the uses in individual countries. The main uses are for beach nourishment, construction, and land reclamation. The exception to the above figures is Norway, where the material is entirely carbonate sands for agricultural use as lime, dredged in small quantities. France and the UK also extract some for this purpose.

In different countries, the authorities involved in consultation and regulation are a widely varying combination of government ministries and agencies, regional and local authorities, conservation bodies, and other bodies. Most countries require a formal consultation procedure involving impact studies concerning (some or all of) the coast, the seabed, the environment and ecology. Norway adopts an approach based primarily on local knowledge.

The UK, Netherlands, Denmark and France require formal evaluations of the impacts on (most of): beaches, coastal sediments, sand banks and bars, waves and currents. They also require ecological evaluations of (most of): the existing flora and fauna, the potential impacts upon them, the effects of turbid plumes and smothering, and whether alternative sites or methods of extraction might give lesser impacts. Italy and Norway require less extensive evaluations, but avoid environmentally sensitive areas.

The criteria considered in evaluating licence applications generally concern water depth, distance from the coast, and depth of extraction pit, and show considerable differences between countries. However, these criteria are only indicative, to help guide the overall decision. The experiences gleaned from the project countries, supplemented by worldwide experience collated in a literature search, show little evidence of adverse impacts. No coastal problems have been reported, at least in the last 100 years or so. The rate of infill of pits varies strongly with water depth, from fast (a matter of months) in shallow water, to very slow (decades or centuries) in deep water. In some cases the pits migrate slowly in the direction of the dominant current. The lack of coastal problems experienced shows that the present regulation procedures work; indeed, some might claim that it is indicative that they are over-cautious. However, any relaxation of the procedures would have to be based on hard scientific evidence. It would always be necessary to maintain a wide margin of safety to protect the coastline and the environment.

3.3 Typical studies to be done

Offshore sand and gravel resources must be managed properly and effectively to ensure that environmental damage to marine and coastal environments will be minimised.
Prior to mining activities at offshore extraction sites, the potential for adverse changes in local waves and current patterns due to changes in local bathymetry resulting from dredging operations must be assessed. Changes in wave heights and directions after dredging may result in localized changes in erosional and depositional patterns (near-field effects) and possibly even in shoreline changes (far-field effects) due to changes of longshore transport rates close to the coast. The 'State of the Art' approach is to initiate numerical modelling studies of waves, tide-induced and wind-induced currents, sediment transport patterns and related morphological changes in the local area near the extraction site as well as the impact on the shoreline.

During the last decade the primary concern has been, however, to address the ecological impacts on marine life as a consequence of mining/dredging of sand and or gravel.
Generally, the ecological studies have focussed on the compilation and synthesis of existing marine literature and available data sets related to offshore extraction areas, as well as biological field sampling surveys before, during and after the mining activities. Ecological effects using sampling surveys are nearly always evaluated on the basis of control sites in the neighbourhood of the extraction sites (success of recolonization following cessation of dredging). Biological sampling surveys have included collecting traditional benthic grab and core samples, sediment profile camera and video sled images.

Recognizing that environmental effects of dredging operations are similar for most areas, generic-type studies such as the **EU-SANDPIT Project** have also been initiated to develop or recommend appropriate mitigation measures, computer modelling or monitoring methods to alleviate or prevent adverse environmental impacts of offshore mining of sand and gravel. The results of all types of studies may be used for evaluation of specific proposed mining/dredging operations, as required under current environmental laws and legislation.

Guidance for the management of marine sediment extraction is given in 2002 by the International Council for the Exploration of the Sea (ICES, Palægade 2-4 DK-1261 Copenhagen K Denmark)., see ANNEX 3.

UNITED KINGDOM

Items	Description
Future demands of sand and gravel coming 50 years (volumes in m^3)	500 to 1000 million m^3 sand and gravel of which about 50% gravel; annual demands over the last 5 years are about 24 millions of tonnes.
Purpose of mined sediment	General construction (aggregate for concrete), export, land reclamation, beach nourishment.
Overview of existing and future mining locations	Five main areas: Bristol Channel (sandbanks), South Coast either side of Isle of Wight, outer Thames estuary, and east coast off Great Yarmouth and off the mouth of the Humber (see www.crownestate.co.uk). A potential new area (eastern English Channel) is now being investigated.
Overview of monitored mining pits and studies done	Many "Coastal Impact" and "Environmental Impact" studies for offshore dredging areas, with some subsequent monitoring. Studies and monitoring of nearshore shingle bank dredged for beach recharge (off Hurst Spit). Some data on infill of "accidental" pits off Bournemouth (1974/75).
Authorities and legal aspects involved	Office of Deputy Prime Minister (ODPM) (Waste and Minerals Division) – and equivalent bodies in Wales (Welsh Assembly Government) and Scotland (Scottish Executive) act as planning authorities and consult with Dept of Environment, Food and Rural Affairs (DAFS in Scotland); Environment Agency; Local Councils; National Conservation bodies (English Nature, Countryside Council for Wales) etc. Seabed "owners", mainly the Crown Estate, arrange civil law contracts with dredging contractors, and monitor activities (e.g. amounts taken from where) but only after permission from Government.
Consultation procedure	Licensing system for offshore dredging started in 1963. Dredging companies now submit an application for a licence to the ODPM (or equivalent) to dredge a defined area at a given rate, having first agreed their proposals with the landowner (normally the Crown Estate), to avoid overlapping extraction areas etc. ODPM advise the applicants on their requirements for consultations and assessment of impacts of proposed dredging. This includes both a Coastal Impact Study and a wider-ranging Environmental Impact Assessment, and the Applicant has free choice on the selection of an appropriate consultant to carry out such studies. Once these reports have been written, with consultations normally undertaken before (to establish scope) and during the reporting, ODPM undertake further consultations on the basis of the Applicant's reports. They then decide whether to refuse permission for extraction, or grant it (usually subject to conditions) or to require further studies. Permissions now given for 15 years but can be withdrawn without notice if adverse effects occur. Once permission has been given by appropriate "competent authority", the Applicants enter a civil law contract with the seabed owners, including an agreed rate for each tonne extracted. Because the present system is presently non-statutory, there is no right of appeal against a decision.
Hydro and morpho-evaluation methods	Within the Coastal Impact Study, the following phenomena are studied and evaluated: 1) the beach should not be affected from draw-down into the dredged area (no permanent trapping of sediments of beach into dredged pit); 2) the supply of sediments to the coastline should not be affected; 3) any bars and banks providing protection to the coast from wave attack should not be damaged/affected; 4) any significant changes in wave refraction patterns altering nearshore waves and hence the alongshore transport of sediment 5) any changes to tidal currents close to the coastline. Item 2 requires an estimation of modified flow and wave patterns, of changes to sediment transport over seabed and hence to (coastal) morphology based on regional and local modelling and existing field data. (e.g. bedforms, sediment distribution/ mobility calculations).
Ecological evaluation method	An environmental assessment report is required, often concentrating on the production of turbid plumes and deposition of sand or finer-grained sediment on the seabed outside the extraction area. Includes a description of existing environment and of the impacts of proposed dredging compared with alternatives. Consideration of "cumulative impacts" of multiple dredging (or other) activities in same general region.
Type of regulations and criteria	No fixed limits, but rare in water depths less than 15m (lowest tide). Each application subject to specific studies of effects on coast and of other environmental impacts. Beach draw-down: The approximate limit for onshore/offshore movement off the South Coast of England is considered to be about 10 metres below low water as being the minimum depth to ensure that beach draw-down will not take place; an additional limit is a minimum distance of 600 m from the shore. Almost all extraction areas are in much deeper water. Seabed sediment transport: Shingle (gravel) is unlikely to be mobile below 18m (CD) based on field tracer studies, but more detailed and specific studies are required for sand transport (even if extraction is for shingle). Sand bar and banks: Minimum depth based on special studies depending on location; dredging of banks adjacent to coastline is not allowed; except in conditions with high accretion rates. Effects on wave refraction: An old rule-of-thumb was a minimum water depth of 14 m based on wave refraction studies along the south coast of England. Now it is sometimes simpler to carry out wave refraction modelling for areas even in much deeper water, than to risk criticism that the effect has been ignored. Effects on currents: Not a real issue except very close to the extraction area (near-field), but may affect sediment transport locally as well (and hence affect the biology of adjacent areas).
Existing experience (lessons learnt)	No discernible effects on coastline over last 50 years; Dredged areas in deep water do not subsequently infill with fine sediment. Dredged "pits" in shallow water, for example on sandbanks, are not discernible after a few months, as natural processes fill in depressions quickly, leading to small changes (not detectable) over a much larger area of seabed.

Table 3.2.1 *National practices and regulations of the United Kingdom*

NETHERLANDS

Items	Description
Future demands of sand and gravel coming 50 years (volumes in m^3)	1600 to 3000 Mm^3 sand; the amount of gravel is negligible. The present demand is 32 Mm^3 sand per year.
Purpose of mined sediment	Beach and foreshore nourishment; land reclamation, general construction. At the moment 12 Mm^3 sand per year is used for beach and foreshore nourishment. Another 20 Mm^3 sand per year is used for general construction and strengthening/heightening of building areas. For the planned extension of the Maasvlakte about 400 Mm^3 sand will be needed. A future airport-island in the North Sea will need a further 2000 Mm^3 sand.
Overview of existing and future mining locations	Sand mining is done along the coast of the Netherlands. For beach and foreshore nourishment the sand is mined as close as possible. Most of the mining is done along the coast between Hoek van Holland and Den Helder. (see http://www.sandandgravel.com/)
Overview of monitored mining pits and studies done	1) One of the deeper mining pits (Verdiepte Loswal) has been monitored quite extensively. The monitoring programme is called PUTMOR. This PUTMOR data set contains mainly bathymetry and hydrodynamics. The pit is located near Hoek van Holland at a water depth of about 20 meter and had a depth of about 30m (about 10m below the surrounding area). The pit has been filled after half a year with dredging material of the Rotterdam harbour. Due to depth and the short period (half a year) the morphological changes are very small. 2) Dredging records from the approach channels to Ports of Rotterdam and Amsterdam 3) Temporary sand pits in the nearshore zone, e.g., a test trench near Scheveningen
Authorities and legal aspects involved	Licensing is done by the Ministry of Transport and Public Works, Rijkswaterstaat, North Sea Directorate. For mining of an area greater than 500 hectares an Environmental Impact Assessment will be necessary in the near future.
Type of regulations and criteria	In general sand mining is only allowed beyond the 20 m depth contour; while only deepening up to 2 meter is allowed. An area that has been mined once cannot be mined again. In case of harbour entrances, mining within the 20 m depth contour is allowed. For the "Verdiepte Loswal" (see above) a special Environmental Impact Assessment has been made. Because the pit was only temporary permission was possible. For the coming 10 years it is expected that deeper mining up to 10 to 30 meter below the surrounding surface will be allowed. With a smaller (and deeper) mining pit the damage to the seafloor will diminish. Also the sand needed as aggregate of concrete is only available at greater depth. At smaller depth only fine sand can be found which cannot be used as aggregate for concrete. An Environmental Impact Assessment will be necessary.
Hydro and morpho evaluation methods	Estimation of modified flow and wave patterns (modelling); estimation of morphological changes based on modelling and existing experiences.
Ecological evaluation method	Description of existing situation; environmental and ecological evaluation for all alternatives and special attention for an alternative with minimum environmental impact. Special attention is given to turbidity plumes and their consequences for primary production, benthos, fish and birds. In the mining area recolonisation of the fauna is an issue
Existing experience (lessons learnt)	No effects on present coastline; very slow migration of existing pits. Due to the fact that sand mining is not allowed within the 20 m depth contour there are no expected effects on the erosion of the present coastline.

Table 3.2.2 *National practices and regulations of the Netherlands*

DENMARK

Items	Description
Future demands of sand and gravel coming 50 years (volumes in m^3)	300 to 500 million m^3 sand, shingle gravel and fill sand. About 70% fill sand for beach nourishment, reclamation and fill. About 12% sand and 14% gravel/shingle as raw material, primarily for concrete (figures obtained by extrapolation of annual demands over the last 5 years)
Purpose of mined sediment	General construction (aggregate for concrete), land reclamation, beach nourishment
Overview of existing and future mining locations	From 113 so-called transition areas located all around Denmark (about 70 of these are being exploited), see http://www.sns.dk/internat/. After 1 January 2007, mining shall take place from geographically delimited areas that have been the object of an impact assessment. The transition areas are primarily the source for raw materials for the construction industry. In addition permissions have been granted for areas to be used for mining material, 'fill sand', for beach nourishment or for specific large construction works.
Overview of monitored mining pits and studies done	Several "Environmental Impact Assessment" studies for offshore dredging and dumping areas, in few cases with some subsequent monitoring.
Authorities and legal aspects involved	Danish Forest and Nature Agency, Ministry of the Environment.
Consultation procedure	The Danish Law on Dredging of Raw Materials was changed in 1996. In order to facilitate the adaptation to the new legislation and to avoid placing the aggregate industry under unfavourable conditions, a transition period of 10 years was defined, during which sand/gravel mining from a large number of transition areas (113) could continue under present conditions. The 113 transition areas were chosen because they had already been under exploitation and, based on surveys, it was expected that they still contained enough materials to supply the country during a number of years. When these transition areas were defined, all of them were subject to an environmental impact assessment. Consultations with authorities and organisations were also carried out in order to ensure that mining should not

	have significant negative impacts on the environment. The transition period extends from 1 January 1997 to 31 December 2006. After this period, the areas that are still wanted to be exploited by the industry will have to go through a conversion procedure and be redefined as mining areas in agreement with the requirements in the new law.
Hydro and morpho-evaluation methods	The following phenomena would be typically studied and evaluated as part of a Coastal Impact Study: 1) Any bars or banks providing protection to the coast from wave attack should not be damaged/affected; 2) Dredging should not significantly interfere with nearshore transport processes; 3) Any significant changes in nearshore wave climate and hence the alongshore transport of sediment Any changes to tidal currents, levels and tidal prism (in connection with dredging of tidal inlets).
Ecological evaluation method	Typical aspects investigated as part of an environmental impact assessment would include: 1) Spill and spreading of fine sediments during dredging operations and deposition on the seabed 2) Extension and thickness of deposition area 3) Impact on marine flora and fauna Description of existing environmental conditions and of the impacts of proposed dredging. Suggestions for alternatives and changed extraction procedures
Type of regulations and criteria	No fixed limits. Mining from well-defined transition areas and other areas with a licence granted. After 1 January 2007, each application subject to specific studies of effects on coast and of other environmental impacts. A permission will specify conditions for the extraction such as: conditions on vessels to be used, methods to be used (e.g. trailing suction), records to be kept and reporting, limits for spill, relations with fishing activities, conditions of the sea bed, max. pit depth (typically 3m if allowed), no reduction of navigation depth.
Existing experience (lessons learnt)	No reports on negative effects on coastline or significant changes in tidal levels/prism.

Table 3.2.3 *National practices and regulations of Denmark*

ITALY

Items	Description
Future demands of sand and gravel coming 50 years (volumes in m^3)	Not available
Purpose of mined sediment	Beach nourishment
Overview of existing and future mining locations	North Adriatic sea, south-central Tyrrhenian sea. Preliminary investigations to locate further sand deposits in northern Tyrrhenian sea and central Adriatic sea.
Overview of monitored mining pits and studies done	Studies have been done prior to mining, in connection to beach nourishment of Veneto coast and Lazio coast, particularly to detect sand deposits and to evaluate the opportunity and effects of sand extraction from the selected sites.
Authorities and legal aspects involved	Ministry of Environment, local authorities, Regions, civil engineers- maritime works (Genio civile opere marittime) Ministry of public works
Consultation procedure	Application to Ministry of Environment according to the law on dumping and handling of materials in sea waters. (Decreto Legge 24 Gennaio 1996 n. 24). The authorization is given by the Ministry of Environment also on the basis of the opinion of "Genio Civile - Opere Marittime" and of local administrations.
Hydro and morpho-evaluation methods	purpose of the work kind of environment (port, estuary, beach,..) dredging system dredge localization dredge extension
Ecological evaluation method	phytozoobenthos community existing in the quarry area physical, chemical and microbiological characteristics of sediments
Type of regulations and criteria	Mining areas should not be in protected areas (archaeological sites, biological protected sites, natural parks, …..) or sensitive areas (3 miles from coastline or from protected areas, grasslands of phanerogamae. …) and should not influence protected areas and fragile eco-systems and usage of marine resources. The distance from the coast should not be less than 3 nautical miles Water depth not less than 50 m. except for high and medium Adriatic sea.
Existing experience (lessons learnt)	Sand mining practice in Italy is rather new and related only to beach nourishment.

Table 3.2.4 *National practices and regulations of Italy*

NORWAY

Items	Description
Future demands of sand and gravel coming 50 years (volumes in m^3)	Mostly shell (carbonated) sand; approximately 10^8 kg per year
Purpose of mined sediment	Mainly used for agricultural purposes
Overview of existing and future mining locations	The main resources along the South-West coast of Norway
Overview of monitored mining pits and studies done	
Authorities and legal aspects involved	The Norwegian Government is the owner; the authority is given to regional authorities
Type of regulations and criteria	Very few strict rules exist; the assessment for giving permission is based on the local knowledge of the actual areas, based on e.g.: protected area; a regulated area for other purposes; hearings among others who are considered to be affected by the activity; environmental issues; breeding conditions for fish/spawn; fishing grounds
Hydro and morpho evaluation methods	
Ecological evaluation method	
Existing experience (lessons learnt)	

Table 3.2.5 *National practices and regulations of Norway*

FRANCE

Items	Description
Future demands of sand and gravel coming 50 years (volumes in m^3)	Available ressources (in less than 30 m water depth and within economically reasonable distance from a harbor) : 600 Mm^3. Annual demands over the last 5 years are about 3 Mm^3 per year.
Purpose of mined sediment	General construction (aggregate for concrete), soil conditioner (carbonate sands);
Overview of existing and future mining locations	Brittany and the Loire region (Atlantic coast and English Channel) provide 2/3 of the national production (which amounts to 2.5 $Mm^3$of sand and gravel and 0.5 Mm^3 of carbonate sands and lithothamnium). The largest site is the Loire estuary, where 1Mm^3 of sand and gravel are extracted every year.
Overview of monitored mining pits and studies done	Every dredging site has been studied more or less thoroughly within Environmental Impact Assessments. Definition of an initial state is required before dredging, subsequent monitoring is recommended every 5 years thereafter. Experimental CNEXO pit offshore the Seine estuary has been several times monitored since 1981.
Authorities and legal aspects involved	Dept. of Industry Regional Dept. of Research, Industry and Environment (DRIRE) Port admiral (Préfet maritime) Port authority (Port autonome) Marine division of the Local Direction of Public Works (DDE) Local authorities
Consultation procedure	Dredging companies need to comply with 3 administrative procedures : obtaining a mining permit, an authorisation of using national land (all underwater territories are national), and a opening permit for dredging. Local investigation for the mining permit: According to a 1982 decree, the dredging company submits an application for a licence to the Department of Industry which forwards it to the Local government authority (Préfet) and the Port authorities when applicable. The application includes an Environmental Impact Assessment (coastal impact, impact on fisheries and benthic habitats), carried out by any consultant chosen by the dredging company. After consultation of the Port admiral, the investigation is undertaken by DRIRE, which consults with local departments (Public and Water Works, Telecommunication, Defence, Environment, Culture), local authorities and governmental scientific experts (including Ifremer). The investigation also includes public consultation and consultation of the Marine division of DDE. DRIRE and DDE reports are returned to the Local government authority which issues the license with or without conditions or requires further studies. National investigation for the mining permit : The Department in charge of Mining consults with the Departments of Public Works, Fisheries, Environment, which may refuse the permission. In all cases, the Local government authority (Préfet) advises the applicant of the official answer. Permit for use of national land : The Marine division of the Local Direction of Public Works investigates whether or not the Applicant may use national land for dredging. The contract defines the rate to be paid by the Applicant per unit of volume extracted.

	Work opening : An other investigation needs to be carried out locally before obtaining the working permit. It also includes consultation of DRIRE, local authorities, as well as a public consultation. The whole licensing procedure takes about three year. Permissions were given for 15 years. They are now given for 20 years.
Hydro and morpho-evaluation methods	There is no particular regulation defining the content of the Coastal Impact Study. Reports issued by consulting companies usually rest on bibliographic syntheses describing local current and wave conditions as well as general trends for sediment transport (based on field data or existing modelling results if they are available). They assess changes in wave refraction patterns that may induce alteration of cross-shore or longshore transport of sediment.
Ecological evaluation method	A protocol defines how the inventory of the benthic habitat before and every 5 years during exploitation has to be carried out : method for bathymetry, acoustic imagery and sampling ; from the samples, granulometry, biological sorting, definition of species and genera, density of population. See www.ifremer.fr/drogm/Realisation/Miner/Sable/protocole.htm
Type of regulations and criteria	No fixed limits. Extractions take place in 8 to 23 m of water depth, 1 to 6 km from the coast line. They are 1 to 8 meters deep and on a surface of 1 to 8 km2. Filling of the pit : from wave tank experiments, Migniot and Viguier related the water depth (below low tide level) around the pit to the minimum waves that causes pit filling : df = 4.08 * Hs. If no wave exceeds this value, no filling of the pit due to waves is expected. Usually, the empirical formula of Nicholls et al., 1998, is used to evaluate a depth d_c (below low tide level) below which wave-induced sediment transport is very small : $d_c = 2{,}28 * H_{s(12h)} - 68{,}5 * \frac{H_{s(12h)}^2}{g * T_s^2}$. $H_{s(12h)}$ is the wave height of the heighest waves occuring 12h per year. A pit dredging offshore such depth may be accepted, apart from some cases (e.g. no dredging accepted if an adjacent beach already suffers erosion, or no dredging accepted on a narrow sandbank protecting the shore from wave attack…). No fixed criteria exist concerning the effect of currents. Effects on wave refraction : from wave tank experiments, Migniot et Viguier established that this effect is negligible for a pit of volume around 1 Mm3 situated at least 500m offshore from the breaking line of the highest waves. For the Atlantic coast of southern France, it means a minimum water depth of 20m below low tide level.
Existing experience (lessons learnt)	The CNEXO pit (in 17m water depth, with 3 to 13m dredging) has been shifting northwards by 1 meter per year in the past 20 years and has been filled by 1 to 6 meters.

Table 3.2.6 *National practices and regulations of France*

VOLUMES EXTRACTED	UK	NL	DK	IT	NO	FR
Demand						
Annual demand (Mm^3/a)	14	32	8	N/A	0.06	3
Demand in next 50 years (Mm^3)	500 - 1000	1600 - 3000	300 - 500	Not available	3	up to 600
Type						
Sand (%)	40	100	10	100		35
Gravel (%)	60	0	15			50
Fill sand(*) (%)			75			
Carbonate sand (%)					100	15
Use						
Beach nourishment	X	X	X	X		
Construction	X	X	X			X
Land reclamation	X	X	X			
Industrial		X	X			
Agriculture	X				X	X
Export	X		X			

(*) for use in beach nourishments and reclamations, and as fill in construction works

Table 3.2.7 *Summary table of extracted volumes*

REGULATIONS	UK	NL	DK	IT	NO	FR
Authorities involved						
Govt. ministries & agencies	X	X	X	X	X	X
Regional/local authorities	X			X	X	X
Conservation bodies	X		X			
Others	X					X
Consultation procedure						
Coastal/seabed Impact	X	X	X			X
Environmental Impact	X	X	X	X		X
Local knowledge					X	X

Table 3.2.8 *Summary table of regulations*

EVALUATIONS	UK	NL	DK	IT	NO	FR
Hydro/morpho evaluation						
Beach	X	X	X			X
Coastal seds	X	X	X	X		X
Banks/bars	X	X	X			
Waves	X	X	X			X
Currents	X	X	X			X
Ecological evaluation						
Turbid plumes/smothering	X	X	X			
Describe existing situation	X	X	X			X
Impact on flora & fauna	X	X	X	X		X
Consider alternatives	X	X	X			
Avoid env. sensitive areas				X	X	

Table 3.2.9 *Summary table of hydrodynamic, morphodynamic and ecological evaluations*

CRITERIA & EXPERIENCES	UK	NL	DK	IT	NO	FR
Criteria	(approx)					
Min. water-depth	LAT-15m	MSL-20m	none	50m(*)	none	$4.08H_{smax}$
Min. distance from coast	600m	none	none	5.6km	none	none
Max pit-depth	none	2m	~3m	none	none	none
Experience						
Coastline problems	none	none	none			none
Pit infill (deep water)	v. slow					v. slow
Pit infill (shallow water)	fast					
Pit migration		v. slow				1m/yr
Changed water levels/prism			none			

(*) except for part of Adriatic sea

Table 3.2.10 *Summary table of criteria and experiences*

4. STATE OF THE ART: PROCESSES AND PREDICTION MODELS

4.1 Introduction

In this Chapter 4 the conclusions of the "State of the Art" review (see ANNEX 1) at the beginning of the SANDPIT project in April 2002 are given, focussing on the following subjects:

Bed forms and sediment transport processes near pits (Sections 4.2 and 4.3),
Dynamics (flow, waves, morphology) of pits (Section 4.4),
Prediction models of bed forms, bed roughness, sediment transport and morphology of pits (Section 4.5),
Overview and results of existing pit studies (Section 4.6),
Definition benchmarking data sets of pit morphology (Section 4.7).

The morphological behaviour of mining pits (sediment extraction) is controlled by the sediments supplied by the approaching flow with or without waves superimposed. Therefore, the characteristics of the approaching flow and wave system and the local sediment transport processes including bed forms and associated bed roughess have to be known. A detailed review of these processes is given in ANNEX 1.
Sand mining/dredging and dumping also have various direct and indirect short and long term effects on marine and coastal benthic communities of plants and animals (ecological processes). The ecological impacts depend on complex and dynamic interactions of abiotic and biotic factors including: (i)composition and dynamics of the sediment, (ii) methods of dredging and dumping and the sediment spill and (iii) the occurrence and sensitivity of seagrass and macro-zoobenthic communities and the rate of recovery of the communities affected. An important problem of sand mining may be the release of fine sediments (silt and mud) in the environmental system during the dredging process. One of the consequences of massive sand mining will be the production of an enormous amount of fine sediment, which can be carried over large distances to the coasts and shores of the countries surrounding the mining area, threatening the environmental system at those places. Many of the processes involved are unknown and should therefore be studied more intensively. Within the SANDPIT Project the ecological aspects were studied in great detail. As a start to gain knowledge on the problem, an extensive literature study of the ecological consequences of the extraction of sediment aggregates (mining of sand and gravel/shingle) has been carried out. This research activity within the SANDPIT Project is summarized in Chapter 7.

4.2 Bed forms and sediment transport in laboratory conditions

A large number of laboratory studies have been carried out on transport processes over wave-generated sand ripples. The review (see ANNEX 1) has shown however that experiments involving flow periods and flow amplitudes typical of field-scale conditions are relatively few in number and are limited in their range of flow and ripple conditions. This is particularly true for studies of time-varying suspended sediment, ripple migration and net transport and new large-scale experiments are needed to provide more insight and data for these processes. Given the complexity of the processes and their sensitivity to the variables involved, new experiments should cover a wide range of sediment sizes and flow conditions, including irregular flows. Ripple geometry is clearly of fundamental importance and, although there have been many more studies of ripple geometry than of other processes, serious uncertainties remain: ripple geometry in irregular flow, ripple evolution in non-equilibrium flow and the occurrence of 2-d or 3-d ripples. More large-scale experiments are needed to provide better quantitative understanding of these aspects of wave-generated sand ripples. This was addressed in the SANDPIT Project by performing detailed experiments of rippled bed processes in the large-scale wave tunnels of Delft Hydraulics and the University of Aberdeen.

4.3 Bed forms and sediment transport in field conditions

Sand transport processes and sediment and bedform dynamics in field conditions have been reviewed (see ANNEX 1) with emphasis on the measured processes on the shoreface between the seaward edge of the surfzone and the upper continental shelf on time scales from seconds to a year. The studies reviewed here were done at Nova Scotia, at Duck, New Jersey, southeastern Australia and New Zealand, and in the North sea off the UK, Belgium and the Netherlands. Each environment has its own specific forcings and processes, which emphasises the need for long-term synchronous field measurements of various parameters at the site of interest.

In general, bedload transport becomes more important than suspended load transport for increasing water depth (except during severe storms or swell). Various types of ripples prevail, but in the heavy storms the (transition to) upper plane bed states do occur at water depths far beyond the depth of morphological closure of the surfzone.
A reasonable number of data sets is available for the shallow surf zone. Some data sets are available for the depth zone from 3 to 10 m. Hardly any data sets are available for deep water with depths larger than 10 m. Sand transport under storm wave conditions has only been measured at the Duck site (USA) in the period 1994-1998, on the Flemish sand banks of the southern North Sea in the period 1992-1995 and in the surf zone of Egmond site (North Sea) in 1998. Most of the data refer to the current-related suspended transport component; the wave-related suspended transport has only been measured in the Deltaflume of Delft Hydraulics and at the Egmond site (The Netherlands) in 1998.
There are few reported measurements of bed forms and associated bed load transport, particularly for the deeper parts of the shoreface. Our present sand transport formulations have been calibrated on the basis of the available surf zone data (in the shallow zone from 1 to 3 m). It has yet to be proved that these formulations yield good predictions for the deep water zone. The predicted sand transport rates for deep water (middle and lower shoreface) may suffer from serious errors due to scale effects (water depth effects).
A number of shortcomings of present knowledge are identified:

1. bed-shear stress and hydraulic roughness models for shoreface conditions, give widely varying results and have not been tested and calibrated in this range of conditions; this leads to high uncertainties concerning the bed shear stress components for sediment transport;
2. there are many environments with mixed waves and currents, but interactions between waves and currents are not well understood;
3. there is no concensus on definitions of bedforms and states, especially in conditions with both waves and currents; in addition the genesis of a number of bed states is not well understood;
4. coastal, near-bed density-driven currents derived from riverine fresh-water outflow can cause a net shoreward current with a potentially first-order effect on annual sediment transport, but this effect has not been quantified empirically;
5. the exchange of sediment between surf zone, shoreface and shelf may be important for coastal sediment budgets on longer time scales (decades), but virtually nothing is known about the magnitude and the direction of the net exchange (for different grain sizes);
6. there are very few datasets with measurements of both bedload and suspended load transport and hydrodynamics at high near-bed resolutions, and none that allow the probabilistic integration to annual transport on the shoreface.

These shortcomings lead to the recommendation for extensive field measurements within the SANDPIT Project at a site in the Dutch sector of the North Sea with mixed waves and currents. The North Sea has only small swell waves and is storm-wave dominated. Furthermore tide- and wind-driven currents occur throughout the year. Density-driven currents from river Rhine outflow play a secondary role off the Dutch coast. Consequently wave-current interactions and wave groupiness will play an important role throughout the year. Near the 10 m waterdepth, intermediate and storm waves dominate the sediment dynamics, whereas near the 20 m waterdepth, tidal and wind-driven currents dominate the sediment dynamics.

4.4 Dynamics (flow, waves and morphology) of pits

The influence of the deepened area (pit) on the local current pattern (tide and wind driven) depends on: the pit dimensions (length, width, depth), the angle between the main pit axis and direction of approaching current, the strength of local current and the bathymetry of local area (shoals around pit). Generally, the dimensions of the deepened area are so small that there is no significant influence of the deepened area on the macro-scale current pattern. In most cases the current pattern is only changed in the direct vicinity of the area concerned.
The morphological behaviour of a deepened mining area (pit, channel, trench) in coastal flow (with or without waves) shows some basic features, which depend on the orientation of the pit to the flow direction. When a current passes a pit or channel (perpendicular or oblique), the current velocities decrease due to the increase of the water depths in the pit or channel and hence the sediment transport capacity decreases. As a result the bed-load particles and a certain amount of the suspended sediment particles will be deposited in the pit. The settling of sediment particles is the dominant process in the downsloping (deceleration) and in the middle section and of the pit. The most relevant processes in the deposition and erosion regions are: convection of sediment particles by the horizontal and vertical fluid velocities, mixing of sediment particles by turbulent and orbital motions, settling of the particles due to gravity and pick-up of the particles from the bed by current and wave-

induced bed-shear stresses. The effect of the waves is that of an intensified stirring action in the near-bed region resulting in larger sediment concentrations, while the current is responsible for the transportation of the sediment. In case of flow parallel or almost parallel with the channel axis, the side slopes of the channel are flattened and smoothed due to gravitational effects. When a sediment particle resting on a side slope is set into motion by waves or currents, the resulting movement of the particle will, due to gravity, have a component in downwards direction. By this mechanism sediment material will always be transported to the deeper part of the channel yielding reduced depths and smoothed side slopes. Slope instability may occur in case of relatively steep slopes immediately after (capital) dredging. This may especially occur in deep mining pits and more research into the breaching process in deep pits is recommended.
The sedimentation in mining pits basically consists of two elements:

sediment transport (mud, silt and sand) carried by the approaching flow to the channel, depending on flow, wave and sediment properties;

trapping efficiency of the pit, depending on pit dimensions, channel orientation and sediment characteristics.

These aspects have been studied through the implementation of an extensive series of scenario testing runs using the improved morphodynamic models of the various institutes of the project.

The impact of a extraction pit on the coast can at the present stage of knowledge only be estimated in rough way from the available data of existing extraction pits made in the coastal waters of the USA, Japan, UK and The Netherlands.
Four zones are distuinguished and the impact of a pit in each zone is briefly described.

- **pit at foot of beachface (-2 to -5 m depth contour);**
  - cheap and attractive method for sheltered coasts (mild wave regimes; small littoral drift);
  - infill from beachside and from seaside (annual infill rate is not more than about 3% of initial pit volume; infill rates are between 5 and 15 m^3/m/yr, depending on wave climate; filling time scale is 20 to 30 years);
  - local recirculation of sand; no new extraction sand is added to beach system;
- **pit in upper shoreface zone (-5 to -15 m depth contour);**
  - relatively strong impact on inshore wave climate due to modified refraction and diffraction effects;
  - relatively strong modification of gradients of littoral drift in lee of pit resulting in significant shoreline changes (growth of beach salients);
  - relatively rapid infill of extraction pit with sediments from landside (beach zone); annual infill rates up to 20% of initial pit volume in shallow water (filling time scale is 5 to 10 years);
  - local recirculation of sediment; no new extraction sand is added to nearshore system;
- **pit in middle shoreface zone (-15 to -25 m depth contour);**
  - negligible impact on nearshore wave climate;
  - negligible effect on nearshore littoral drift;
  - no measurable shoreline changes;
  - new extraction sand is added to nearshore morphological system (nourishment);
  - infill of extraction pit mainly from landside with sediments eroded from upper shoreface by near-bed offshore-directed currents during storm events; annual infill rate is about 1% of initial pit volume (filling time scale is 100 years);
  - trapping of mud in pits (negative ecological effect);
  - particle tracer studies show small but measurable transport rates, mainly due to storm waves;
  - long-term deficit of sand for upper shoreface;
- **pit in lower shoreface zone (beyond -25 m depth contour);**
  - no impact on nearshore wave climate;
  - no effect on nearshore littoral drift;
  - no measurable shoreline changes;
  - new extraction sand is added to nearshore morphological system (nourishment);
  - minor infill of sand in extraction pit; only during super storms;
  - trapping of mud in pits (negative ecological effect);
  - particle tracer studies show minor bed level variations (of order of 0.03 m over winter period) during storms.

4.5 Prediction models of bed forms, sand transport and morphology

Bed forms and bed roughness

Coastal regular bedforms can be roughly classified on the basis of their geometric characteristics into large scale (wavelength of *O*~1 km), medium scale (wavelength of *O*~10 m) and small scale bedforms (wavelength of *O*~10 cm). Small scale bedforms, frequently referred to as ripples, appear when the bottom is subjected to oscillatory driving forces that, in presence of a rough bottom, induce the formation of recirculating cells producing ripple growth. The importance of studying ripples arises as a consequence of the considerable modification of bottom roughness, which in presence of such sedimentary structures is not controlled by the grain size anymore but by the ripple height. For example, from field observations, it was noticed that as nonbreaking waves approached shallow waters, in some cases up to 60-70% of their energy was dissipated. This dissipation cannot be justified according to the usual estimate criteria of bottom friction factor, unless an appropriate and enhanced friction factor, which accounts for the presence of bedforms, is considered. The consequences of small scale bedform appearance are therefore extremely important whenever the bedload sediment transport, the damping of wave motion associated to the offshore-onshore propagation, the dispersion of pollutants, etc. are considered. As a consequence of such relevance in coastal problems, ripples have been widely studied both theoretically and experimentally in order to provide reliable predictions about their appearance, their equilibrium features, as well as their migration. Review of the existing literature (see ANNEX 1) shows the presence of a wide range of ripple prediction models and associated bed roughness.

The available prediction models have been used to compute the ripple height and ripple length for a specific case with d_{50}= 0.2 mm and a range of wave conditions (wave period T= 6 s, peak orbital velocities U_o=0.25, 0.50, 0.75, 1.0 and 1.25 m/s). The results (see ANNEX 1) show that there is a wide range in computed ripple heights and lengths. The ripple heights vary between 0.07 and 0 m; the ripple lengths vary between 0.37 and 0 m. Most methods yield a ripple height decreasing with increasing peak orbital velocity, as the ripples will be washed out for large peak orbital velocities (> 1 m/s). A generally accepted method is not yet available. Various institutes within the SANDPIT Project have focussed on the improvement of existing ripple and roughness prediction models.

Sand transport

There exists a large variety of sediment transport models and different combinations between model components (see ANNEX 1). Most, however, have only been tested on laboratory and surf zone datasets, which is outside the scope of the SANDPIT study and, further, most of the transport predictors have been derived for rivers. A number of shortcomings and problems have been identified in deeper waters outside the surf zone:

1. lack of representation of wave asymmetry and irregularity (spectral waves),
2. high sensitivity to ripple height and bed state (e.g. sheet flow threshold) predictors,
3. large uncertainties in concentration predictors, even for uniform steady (fluvial and tidal) flow,
4. high sensitivity of transport predictions to wave-induced, near-seabed, residual currents, and
5. no consensus at present concerning the effect of suspended sediment stratification in high energy events.

Sand transport predictors depend critically upon the computation of the bed shear stress. Here significant uncertainty arises in connection with wave-current interactions, due in part to lack of knowledge of both the bed form (ripple) roughness, and also the mobile bed roughness, particularly in combined wave and current conditions. Due to the small spatial scales involved, these components of the roughness cannot be derived directly from measurements. Instead, based on the measured (outer) flow parameters, shear stress and hydraulic roughness formulae/models must presently be used to yield the separate components of the shear stress (i.e. the grain shear stress (skin friction) component of the total stress, based on the estimated mobile (bed load) related roughness, and the estimated bed form related roughness). This means that a mismatch between modelled and measured sediment transport might be caused by the shear stress model, or the bed form model, or the sediment transport model, or all of them. This potential uncertainty is compounded by systematic measurement errors. In principle, comparisons between various bed shear stress models and various datasets might indicate the present shortcomings, though data with enough resolution and detail for this purpose is scarce.

There seems to be four major shortcomings in the present shear stress and roughness models:

1. the prediction of bed form dimensions, bed form three-dimensionality and, hence, bed form roughness, particularly beneath irregular waves,
2. the prediction of mobile bed (bed load transport) roughness in transitional and sheet flow conditions,

3. the prediction of wave-current interactions, beneath irregular waves superimposed on currents at general angles of attack, and
4. the prediction of the flow dynamics within the wave boundary layer, including the prediction of wave-generated residual currents which may strongly influence sand transport rates.

In addition, models have often been developed for particular environments, and so these models may predict the various roughness components incorrectly when applied in other conditions.

Within the SANDPIT project, most work on the development of local sand transport models has been carried out using practical formulations applied to the rippled bed regime. Work has also been carried out on the development of simplified one-dimensional vertical (1DV) research models for the rippled regime, underpinned by 2DHV modelling studies (e.g. particle tracking models for suspended sediment above ripples). By use of 2DHV (or 3D) models, it is hoped that the essential physical difference between flow and sediment transport above rippled and plane beds will be better understood, leading to convincing parameterisations of, for example, the eddy viscosity and sediment diffusivity above rippled beds, for use in simpler (e.g. 1DV) models.

In the rippled bed regime, the flow and sediment dynamics in the near-bed layer are dominated by convective processes (vortex shedding from ripples); in the plane bed case, the dynamics are dominated by diffusive processes. Our understanding of the respective processes in the case of both steeply rippled and also plane beds has improved significantly in recent years. However, as noted above, operational models are hampered by uncertainty in the prediction of the bed roughness, which can greatly affect the prediction of sand transport rates. Further, very little modelling has been carried out in the important intermediate range involving low, 2D-3D, transitional ripples. Such ripples occur very commonly on the sea bed beneath (spectral) waves combined with currents in water of moderate depth (shallow shelf-sea areas outside the surf zone). Here it may be possible in models to treat the bed as being 'dynamically plane', in the sense that no vortex shedding is occurring from the (low) ripple crests, while still including in the model a significant degree of ripple-related roughness. Our present understanding of these important intermediate conditions owes more to experimentation than to modelling. Therefore in SANDPIT one of the key aims is to inter-compare models with field data obtained offshore in this intermediate range. In addition, the models will be compared with data obtained in controlled laboratory conditions (e.g. in wave tunnels). One outcome of such comparisons should be improved insight into the differences that exist between tunnels (where vertical wave velocities are not present) and the field (where vertical velocities are present). Most research models are based on the assumption that the free-stream flow comprises a purely horizontal motion; in practice, when such models are compared with field data, this assumption may give rise to an under-prediction of the vertical diffusion rates of momentum and mass, affecting transport predictions.

The broader aim in SANDPIT is to improve the local sand transport formulations used in (2DH or 3D) morphological models. Here a further, but separate, complicating factor is the manner in which the horizontal advection-diffusion of suspended sediment is represented over undulating topography. Failure to represent this effect adequately can lead to an incorrect prediction of the rate of seabed morphological evolution. Here the strategy in SANDPIT will be to separate, as far as possible, the prediction of local sand transport rates from the prediction of the morphological response; this can be done, for example, by tuning local transport models to a known or measured rate at the edge of the morphological model domain, and inter-comparing the consequent morphological evolution. From an operational point of view, it is of course necessary to develop modelling systems that do not involve such a tuning. The two-part challenge involved in the SANDPIT project is to combine *local* sand transport models that, while being simple, still include sufficient physics, with *morphological* models that utilise this local information (e.g. through adequate advection-diffusion schemes) to predict correctly the morphological response of coastal area, possibly perturbed by some anthropogenic change (e.g. a sand pit).

Morphology of pits

The models available for mathematical simulation of the sand transport processes and associated morphological changes of mining pits have been summarized (see ANNEX 1). Various models are discussed: simple engineering rules, analytical models and detailed 2DH, 2DV and 3D mathematical (numerical) models based on advection-diffusion schemes. For each model an overview is given of the modelled processes, the basic simplifications and the types of boundary conditions which have to be specified. Furthermore, the suitability of the models for the various characteristic morphological areas is indicated. These types of models have been improved, validated and applied to practical situations within the SANDPIT Project by various modelling participants

4.6 Overview and results of existing pit studies

The hydrodynamic and morphodynamic effects of extraction pits in the USA, UK, Canada and The Netherlands (see ANNEX 1) at various depths in the nearshore coastal zone have been studied by using wave refraction, flow, sand transport and shoreline change models.

As regards hydrodynamics, the wave climate at and inshore of the extraction area is affected (reduced wave heights). The flow patterns outside the extraction area are modified over a distance of maximum twice the width and length of the extraction area. The wave transformation and flow patterns can be simulated quite well provided that the boundary conditions at the model inlet are accurately known. In the absence of such data the uncertainty margins are relatively large (up to factor 2).

As regards morphodynamics, the cross-shore morphological changes are relatively small for pits seaward of the 15 m depth contour; the migration rates are mainly affected by the local water depth and not by the pit dimensions (depth, width, length). The migration velocity of the pit in longshore direction was found to be 10 to 15 m/year. The morphological changes remain within the local surrounding of the pits. On the time scale of 100 years the overall longshore migration of the pit is of the order of 1 to 2 km. The sedimentation of the pit (infilling rate) increases strongly with decreasing water depth outside the pit. The modelling of morphodynamics is not very accurate due to the absence of accurate field data of sand transport processes. In the absence of such data the uncertainty margins are relatively large (up to factor 5).

The presence of a sand pit results in the formation of circulation cells which may trigger the development of a sandbank pattern (based on stability analysis studies). As time evolves, the sand bank pattern spreads out and migrates, alternatingly generating trough and crest zones. The pit itself deepens and the pattern spreads at a rate of 10 to 100 m/year. The migration rate of the centre of the pit is of the order of 1 to 10 m/year.

Beach erosion in the lee area of the pit was found to increase with increasing pit depth and with decreasing original water depth. It can be concluded that extraction pits seaward of the 15 m depth contour donot lead to any significant shoreline erosion.

Similar studies have been done within the SANDPIT project by implementing an extensive series of scenario testing runs using the improved morphodynamic models of the participating institutes.

4.7 Definition of existing benchmarking data sets of pit morphology

An overview of available datasets of the morphology of laboratory and field pits is given in Table 4.1.
Based on this overview, a series of benchmarking testcases of mining pits have been defined to evaluate the selected hydrodynamic and morphodynamic models on their ability to represent longshore and cross-shore processes. High quality benchmarking data is only available for the longshore direction. No reliable field data sets are available for the cross-shore direction, which is the most important direction regarding the impact of a pit on the coast. The uncertainty of the transport poceses is largest in the cross-shore direction due to the delicate balance of onshore and offshore transport rates.

Cases	Longshore			Cross-shore
	Hydrodynamic	Sediment transport	Morphology	Morphology
Pit in laboratory flume (waves and currents parallel to coast)	**Van Rijn (1986)** - velocity profiles inside and outside pit	**Van Rijn (1986)** - concentration profiles inside and outside pit	**Van Rijn (1986)** - bed level profiles after 10 hours	**Migniot and Viguier (1980)** - 2D flume tests with bakelite
Pit in laboratory basin (waves normal shore; current parallel to shore)	**Havinga (1992)** - velocity profile upstream of pit	**Havinga (1992)** - concentration profile upstream of pit	**Havinga (1992)** - bed level profiles after 12 hours	**Migniot and Viguier (1980)** - 3D flume tests with bakelite
Pit in field conditions	1) Pit near Euromaas geul in North Sea, 1999 (Putmor data 1999); velocity profiles in and outside pit; no morphological data **(Svasek, 2001)**	none	1) Scheveningen trench in North Sea, 1965 ; - trench normal to shore - tidal currents parallel to shore - waves from southwest y morphodynamic data during summer period (about 3 months) in depths of 7 to 10 m **(Svasek, 1964)** 2) CNEXO pit in English Channel near mouth of Seine river **(Gomi and Sergent, 2004)**	
Ridges/shoals in field conditions.	none	none	Artificial sand ridge in North Sea near Hoek van Holland, Netherlands **(Rijkswaterstaat, 1996; Delft Hydraulics, 1998)**	none

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Van Rijn, L.C., 1986. Sedimentation of dredged channels by currents and waves. Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 112, No.5, p.541-559

Table 4.1 *Available benchmarking data setsof pit morphology*

5. NEW LABORATORY AND FIELD DATA RESULTS

5.1 Introduction

Based on this overview, a series of benchmarking testcases of mining pits have been defined to evaluate the selected

5.2 Laboratory and field data

5.2.1 New data sets

5.2.2 Analysis of

6. RESEARCH MODELLING RESULTS

6.1 Introduction

Based on this overview, a series of benchmarking testcases of mining pits have been defined to evaluate the selected

6.2 Laboratory and field data

6.2.1 New data sets

6.2.2 Analysis of

Based on this overview, a series of benchmarking testcases of mining pits have been defined to evaluate the selected

7. MANAGEMENT QUESTIONS AND ANSWERS

7.1 Introduction

Based on this overview, a series of benchmarking testcases of mining pits have been defined to evaluate the selected

7.2 Laboratory and field data

7.2.1 New data sets

7.2.2 Analysis of

Based on this overview, a series of benchmarking testcases of mining pits have been defined to evaluate the selected

7. PRACTICAL RESULTS AND SUMMARY OF GUIDELINES FOR ENDUSERS
(including assessment of practical questions for endusers)

7.1 Introduction

....

7.7 Ecological effects

One of the greatest challenges when determining the impact benthic communities face from dredging is the lack of baseline data and overall context in which benthic communities can be compared. Compilation (databases) and analysis of historical benthic data sets for various regions would provide baseline information for use in future biological studies. More knowledge should become available about the ecological relationship between the unique habitats of sand ridge, shoal and bank features and resident benthic communities that live in those habitats. Sand sources such as ridges, shoals and banks that are used repeatedly may require ongoing biological and physical monitoring to alleviate adverse impacts. These features tend to be focal points for various fisheries, both recreational and commercial. Altering the physical characteristics of these areas (grain size, waves, currents and bathymetry) could be detrimental for various fish species. Areas that are often selected as potential sand resources sites are in many cases used by fish as migration corridors, habitat for juvenile devlopment, and spawning grounds. Activities that adversely influence these features, through disturbances in migration patterns and changes in substrate, water quality, or acoustic parameters, can directly result in a decrease in benthic communities and in fisheries.

ANNEX 1

STATE OF THE ART: PROCESSES AND PREDICTION MODELS

ANNEX 1 STATE OF THE ART: PROCESSES AND PREDICTION MODELS

1.1 Introduction

In this ANNEX 1 the "State of the Art" review at the beginning of the SANDPIT project in April 2002 is given, focussing on the following subjects:

Bed forms and sediment transport processes near pits (Sections 1.2 and 1.3),
Dynamics (flow, waves, morphology) of pits (Section 1.4),
Prediction models of bed forms, sand transport and morphology (Section 1.5),
Overview and results of existing pit studies (1.6).

1.2 Bed forms in laboratory conditions

1.2.1 Introduction

The development and validation of models for predicting wave-generated sand transport in the rippled bed regime require measurements of the relevant processes under controlled laboratory conditions. In particular, data are required relating ripple geometry, sand suspension, ripple migration and net sand transport to given oscillatory flow and sand conditions. A very large number of laboratory studies have been carried out on wave-generated ripples since Bagnold's early experiments with oscillating trays of sediment in still water (**Bagnold, 1946**). Many studies have been conducted in small wave flumes with flow periods typically less than 3s. These have provided valuable insights into the basic processes but severe scale effects mean that results are not readily applicable to field-scale conditions. Other studies (see **Table 1.2.1**), conducted in large wave flumes and oscillatory flow tunnels, have eliminated the scale effects by operating with flow periods and orbital amplitudes typical of field conditions. This review concentrates on these large-scale experiments. The review is structured according to the main processes - ripple geometry, sand suspension, ripple migration, net transport. The review concludes with recommendations for further laboratory work on ripple regime sand transport processes.

1.2.1 Ripple geometry experiments

Sand transport models ranging in complexity from relatively simple empirical models to complex, process-based numerical models all require a priori estimates of the ripple geometry for the given flow and sand bed conditions. A number of empirical formulae have been developed for ripple geometry (e.g. **Vongvisessomjai, 1984; Nielsen, 1992; Wiberg and Harris, 1994; Mogridge et al., 1994**). Many are heavily based on empirical data from laboratory wave flume experiments and are in general agreement with each other and predict ripple geometry well for the short period, low amplitude flows that are typical of wave flume conditions. However, the formulae diverge extremely when applied to field-scale flows. A recent comparison of predicted ripple dimensions using a number of formulae with ripple measurements from large-scale flow tunnel experiments involving a wide range of sands and field-scale flows (**O'Donoghue and Clubb, 2001**) has shown that the **Mogridge et al. (1994)** formula for ripple geometry gives best agreement with measured large-scale data.

Ripple geometry predictive formulae apply to two-dimensional (2-d) ripples, i.e. long-crested, parallel ripples with height and length constant over a large bed area. However, three-dimensional (3-d) ripples have been frequently observed in laboratory settings (Figure 1.2.1), especially in flow conditions with periods and orbital amplitudes typical of field conditions. **Ribberink and Al-Salem (1994)** measured predominantly 3-d ripples in their experiments with 0.21mm sand, while **Sato and Horikawa (1986)** recorded both 2-d and 3-d ripples in experiments with 0.18mm sand and flow periods between 1 and 7 seconds. **Lofquist (1978)** and **O'Donoghue & Clubb (2001)** observed under large-scale oscillatory flows that, as 2-d ripples develop from an initial plane bed, they first go through a 3-d transitory stage before the formation of the final, larger 2-d equilibrium ripples. They both found that 2-d ripples eventually occurred with larger sands but that 3-d ripples persisted in conditions of relatively fine sand and large flow orbital diameters. **O'Donoghue and Clubb (2001)** showed that previously-suggested simple criteria for the occurrence of 2-d/3-d ripples fail when applied to their results covering a wide range of sand sizes and flow conditions. More work is needed to determine criteria for the occurrence of 2-d/3-d ripples and to study predictive formulae for the dimensions of 3-d ripples where they occur.

Reference	Facility	T (s)	d (m)	d_{50} (mm)	Comments
Mogridge & Kamphuis (1972)	OFT	4 8	0.05-1.8	0.36	measured ripple geometry only; regular sinusoidal flows; other expts carried out with bakelite and polystyrene; study included expts in wave flume.
Lofquist (1978)	OFT	2 -16	0.18-0.92	0.18 0.21 0.55	relatively small tunnel; measured ripple geometry only; regular sinusoidal flow; low mobility for long period flows; 3-d ripples for finer sand
Sakakiyama *et al.* (1986)	LWF	3-12	H=0.25 - 1.76m; variable depth	0.27 0.47	measured ripple geometry and beach profiles; sand beach had slope 1:20 or 1:33; ripples change along the beach; some very large bedforms.
Sato & Horikawa (1986)	OFT	1-7	0.05-0.65	0.18	relatively small tunnel; regular and irregular Stokes/cnoidal flows; measured geometry and transport; discusses occurrence of 2d/3d ripples; net offshore transport; proposes net transport formula based on max Shields.
Ribberink & Al-Salem (1994)	OFT	2-10	0.3-3.8	0.21	large tunnel (LOWT) – Series A expts; sinusoidal flows; study included sheet-flow expts; measured geometry and averaged concentration profiles; mainly 3d ripples;
Ribberink & Al-Salem (1994)	OFT	5 6.5 9.1	0.56-1.35 (est. for the 6 irreg flows)	0.21	large tunnel (LOWT) – Series B expts; 6 ripple regime expts with irreg flows; study included sheet-flow expts; measured geometry, migration, averaged conc profles, net transport; small 3d ripples; migration 6-30mm/min; net onshore transport in all 6 cases.
Williams et al. (2000)	LWF	5	0.3-1.1	0.162 0.329	DELTAflume expts; regular and irreg jonswap waves; measured geometry but main emphasis is on averaged c-profiles; ripples reported to be long-crested, nearly 2d.
Villard et al. (2000)	LWF	≈4	≈0.9 (max)	0.25	wave time series consists of repeats of a group of 8 waves; measure bed geometry and spatial and temporal variations in suspended concs (ssc); emphasis is on influence of wave groups on ssc; gets 3d ripples.
O'Donoghue & Clubb (2001)	OFT	2-15	0.16-2.92	0.18 0.26 0.34 0.44	large tunnel (AOFT); regular sinusoidal and Stokes 2nd type flows; measure geometry only; 2-d ripples except for fine sand but time to eqm. can be many hours; good agreement with Mogridge et al. ripple formula.
Clubb (2001)	OFT	5 10	1.1-2.2	0.34	large tunnel (AOFT); regular sinusoidal and Stokes 2nd type flows; measure geometry, t-varying conc, migration, transport, particle sizes; 4 concentration expts and 4 transport experiments only.
Vincent et al. (2001)	LWF	6.5	≈0.8-1.5	0.23	Hannover wave flume; 3 wave conditions: 2 regular wave and 1 "repeating wave groups"; bedform measurements and ABS suspended sand concs (ssc); compares ssc for regular and ireg wave with similar Hs.

OFT=oscillatory flow tunnel; LWF=large wave flume.

Table 1.2.1 *Large-scale" laboratory studies of wave-generated ripple regime sand transport processes*

Irregular flows have been used in a few laboratory studies in order to better model natural environments. There are conflicting results regarding the degree of difference in ripple geometry produced under regular and irregular flows. **Marsh et al. (1999)** found in wave flume studies that irregular waves simulated from field measurements did not produce ripples that fit the ripple geometry prediction models they tested while the models were found to perform better under monochromatic waves. In large-scale oscillatory flow tunnel experiments, **Ribberink and Al-Salem (1994)** observed a decrease in ripple size under asymmetric and random flows compared with sinusoidal flows, while **Williams et al. (2000)** found that ripple dimensions under regular and irregular waves were in good agreement with those predicted by **Nielsen's (1992)** formula for regular flows. Using a sand size of 0.23mm in a large wave flume, **Vincent et al. (2001)** report ripples of similar magnitude (η=5-20mm, λ=200-400mm) under regular and irregular "groupy" waves of similar significant wave height but the ripples were superimposed on much larger bedforms (η=0.08m, λ=2m) in the case of the irregular waves. Altogether the range of irregular, large-scale flow experiments is very limited so that much uncertainty remains regarding the determination of ripple geometry for irregular wave conditions.

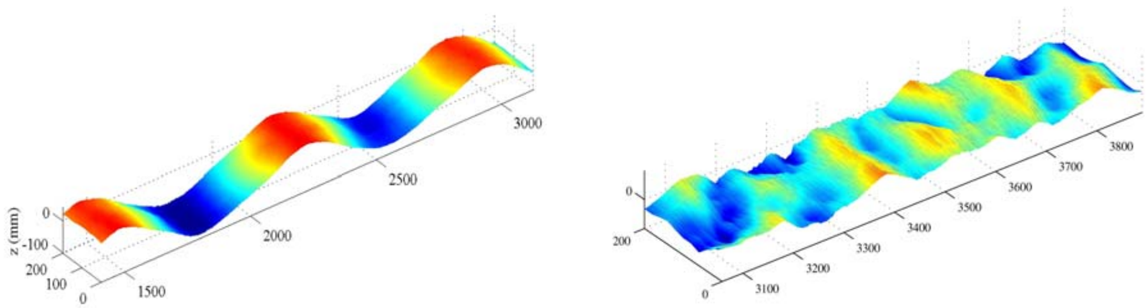

Figure 1.2.1 *Example 2-d and 3-d rippled beds measured by O'Donoghue and Clubb (2001) in large-scale regular flows.*

Marsh et al. (1999) suggest from their wave flume experiments that it is rather difficult to change the wavelength of ripples once they have formed implying that bed history is sometimes as important as hydrodynamic conditions in determining ripple geometry. Similarly, **O'Donoghue and Clubb (2001)** observed that, depending on the mobility, it can take many hours for equilibrium ripple geometry to be reached in a given constant flow. They conclude that, because it takes time for ripples to grow towards equilibrium and for established ripples to respond to a change in the flow, ripple geometry in the field at any given instant may not be the equilibrium geometry corresponding to the flow condition at that instant. (There is much evidence from the field to support this view; e.g. **Traykovski et al., 1999; Hanes et al., 2001**). However, no laboratory studies have yet been carried out to systematically study the time scales at which ripples respond to a change in the prevailing flow conditions. Such work is needed to help better understand field measurements of ripple geometry and to determine the extent to which "non-equilibrium" conditions may prevail in real sea conditions.

1.2.3 Sand suspension experiments

The process of sand suspension above wave-generated ripples is dominated by vortex growth in the lee of each ripple and ejection of the sediment-laden vortex into the main flow at around the time of flow reversal. Suspended sand concentration depends on time (wave phase), height above the bed and horizontal location relative to the ripple crest. The suspended sediment concentration profile (concentration c as function of height z above the bed) is therefore both time- and horizontally-varying. Laboratory studies of suspended sediment above rippled beds have largely been concerned with establishing the time- and horizontally-averaged concentration profile for given flow (wave) and bed conditions. Much of this research has been based in small wave flumes or small flow tunnels (e.g. **Nakato et al., 1977; McFetridge and Nielsen, 1985; Bosman and Steetzel, 1986; Ono et al., 1994**). Suspended sediment concentrations have also been measured in a few large-scale laboratory studies, involving longer period, larger amplitude flows typical of field-scale conditions, in oscillatory flow tunnels (**Hayakawa et al., 1983; Ribberink and Al-Salem, 1994; Clubb, 2001**) and in large wave flumes and basins (**Williams et al., 2000; Vincent et al., 2001; Villard et al., 2000; Villard and Osborne, 2002**).

Ribberink and Al-Salem (1994) used suction sampling to measure time- and horizontally-averaged concentration profiles for a number of sinusoidal flows and a few irregular flows over a sand bed with d_{50}=0.21mm. They concluded that the time- and bed-averaged concentration profile is reasonably well described by a negative exponential distribution with a decay length approximately equal to the ripple height. **Williams et al. (2000)** also used suction sampling to measure time- and horizontally-averaged concentration profiles for two sand sizes (d_{50}=0.33mm and 0.16mm) under regular and irregular waves in a large wave flume. They conclude that a diffusion-based formula best describes the averaged concentration profiles for both regular and irregular wave conditions.

Measurements of time- and horizontally-varying concentrations above ripples in large-scale flows are rare. **Clubb (2001)** used a carousel-based suction sampling system to obtain time- and horizontally-varying concentration measurements at many locations above large 2-d ripples in two sinusoidal and two asymmetric flows over a 0.34mm sand bed. He found that sinusoidal and asymmetric flows with the same period and rms velocity resulted in very similar time- and horizontally-averaged concentration profiles despite very large differences in the time- and horizontally-varying concentrations in the two flow types. **Villard and Osborne (2002)** and **Vincent et al. (2001)** have used acoustic backscatter systems (ABS) to measure time-varying concentration profiles in large wave flumes and wave basins involving regular and irregular waves, but the range of experimental conditions was relatively small in both cases. **Villard and Osborne (2002)** found that the suspended sediment concentrations in irregular flows are influenced by the recent history of the flow, with large suspension events associated with the pairing of antecedent and developing vortices due to the passage of wave groups. **Vincent et al. (2001)** compared suspended sediment concentrations measured in regular and irregular waves and found that regular waves suspend an order of magnitude more sediment than "groupy" irregular waves with similar significant wave height.

Overall, the range of large-scale experiments for which detailed suspended sediment concentration data are available is very limited indeed. More time-varying concentration data are needed for regular and irregular flows and a wide range of ripple conditions in order to provide more insight and quantitative data for the development and validation of process-based numerical models.

1.2.4 Ripple migration experiments

Asymmetry in the wave-generated near-bed flow generates ripple migration. For flows with higher maximum onshore than maximum offshore velocities, ripple migration tends to be in the onshore direction, contributing towards ***onshore*** transport. On the other hand, the flow asymmetry produces asymmetry in vortex size and in the volume of sediment entrained at flow reversal, with a larger vortex and greater volume of sand being entrained at the on-offshore reversal than at the off-onshore flow reversal. This results in a net ***offshore*** suspended sediment transport. The net total transport depends on the relative magnitudes of the offshore directed suspended transport and the onshore directed ripple migration transport. Which of the two dominates depends on the ripple geometry, the flow and the sediment characteristics.

Despite its importance in determining net transport rate, the total number of controlled, large-scale experiments for which ripple migration has been studied is very limited. **Faraci and Foti (2002)** observed ripple migration in the laboratory for irregular waves but the wave periods were relatively short (<5s). Only **Ribberink and Al-Salem (1994)** and **Clubb (2001)** present laboratory measurements of ripple migration from field-scale laboratory experiments. **Ribberink and Al-Salem** measured ripple migration for 0.21mm sand in irregular flows and reported 2-d and 3-d small ripples (η<15mm, λ<100mm) with high onshore migration rates in the range 6-30mm/min. Net total transport in the Ribberink and Al-Salem experiments was onshore, which means that the onshore transport related to ripple migration was greater than the offshore transport related to suspended sediment. This behaviour is different to that described by **Clubb (2001)** for large 2-d ripples (η=70-140mm, λ=510-770mm) comprising 0.34mm sand in regular, asymmetric flows. In Clubb's case the large ripples migrated onshore with speeds in the range 2.5-8.5mm/min but high sand suspensions occurred over the large ripples so that the net total transport was offshore directed. Clubb showed that the onshore transport was mainly determined by ripple migration and might be reasonably estimated from the product of ripple height and migration speed.

The conclusion here is similar to that for suspended sand concentrations, i.e. the number of controlled, large-scale experiments for which ripple migration has been studied is very limited so that quantitative understanding of ripple migration under waves is poor. Further controlled, large-scale experiments are required to study ripple migration under regular and irregular oscillatory flows.

1.2.5 Transport experiments

Net transport over rippled beds has been measured in early laboratory experiments using oscillating trays in still water (**Manohar, 1955; Kalkanis, 1964; Abou-Seida, 1965**) and more recently in wave flumes with short period (< 3s), low orbital amplitude flows (**Ishida et al., 1983; Vongvisessomjai et al., 1986; Sistermans, 2001**). Laboratory experiments with flow periods and amplitudes closer to those of typical field conditions have been carried out in oscillatory flow tunnels by **Sato and Horikawa (1986), Ribberink and Al-Salem (1994) and Clubb (2001)**.

Clubb (2001) and Sato and Horikawa (1986) measured transport for cases of equilibrium ripples and found the net transport to be ***offshore*** directed in all cases. Sato and Horikawa also measured transport as the ripples developed from an initially flat bed and showed that net transport was initially ***onshore*** before becoming increasingly offshore as the equilibrium ripples developed. **Vongvisessomjai et al. (1986)** also measured maximum onshore transport when ripples were beginning to form, before decreasing as the equilibrium ripple was reached. These results point again to the importance of flow duration, bed history and time-scales of bed evolution in determining the sediment transport. In contrast to **Clubb (2001)** and **Sato and Horikawa (1986), Ribberink and Al-Salem (1994)** measured net ***onshore*** transport in each of their irregular asymmetric flow ripple regime experiments. The **Ribberink and Al-Salem** ripples are reported as being small (η<15mm, λ<100mm) with definite onshore migration. The net onshore transport will have been a result of low sand suspensions at flow reversals caused by the ripples being small, resulting in low offshore suspended transport compared with the onshore ripple migration.

The **Sato and Horikawa (1986)** transport experiments were carried out with fine 0.18mm sand in a relatively small flow tunnel with flow periods not greater than 5s. Transport experiments in larger facilities with longer flow periods are limited to the 4 regular asymmetric experiments of **Clubb (2001)** with 0.34mm sand and the 6 irregular flow experiments of **Ribberink and Al-Salem (1994)** with 0.21mm sand. Given the complexity of transport over ripples, many more large-scale experiments are needed in order to develop better predictive models for wave-generated ripple regime transport. New experiments should cover regular and irregular flow conditions and, ideally, should be linked with corresponding measurements of ripple migration and suspended sediment transport.

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1.3 Bed forms in field conditions

1.3.1 Objectives

The objective is to review the sand transport processes and related small-scale bed states in inner, shallow shelf regions and on the shoreface seaward of the surfzone. This work is based on a more extensive review (**Kleinhans 2002b**). The dynamics of this region, and the exchange of sediment with the surfzone, are largely unknown. It is only in the past decade that a number of datasets were published that include wave and flow dynamics, suspended sediment concentrations as well as bedform dynamics. The datasets have been collected in various environments with different dominant forcings. Herein, the sediment transport and bedform knowledge is reviewed.

The shoreface is defined here as the realm in which waves are shoaling but not breaking in rather high-energy conditions (see Figure 1.3.1; **Vincent 1986**), while this is only the case for the inner shelf in extreme high-energy conditions. In many cases the transition from inner shelf to shoreface is not gradual but shows a distinct break in the bathymetry.

In terms of depth contours, the following subdivision will be used:

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

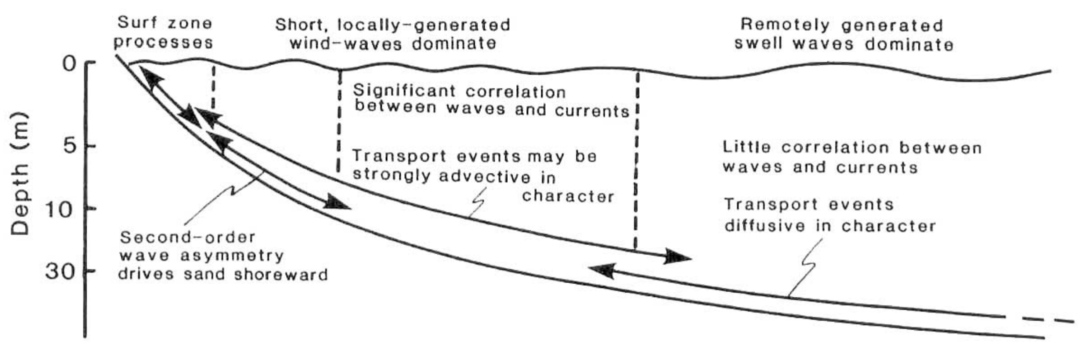

Figure 1.3.1 *Definition sketch of the shoreface and inner shelf regions (Vincent 1986).*

In the surfzone, morphological changes usually are large throughout the year, but taper off seaward of the breaking waves. This activity can be represented as an enveloping band around the mean bed level, in which bar migration and large-scale erosion or sedimentation cause fluctuations in the order of meters. Seaward, this band of activity tapers off to the so-called depth of closure, where morphological change is no longer measurable. The depth of closure is related to the time-scale of the measurements and to the measurement accuracy.

Hallermeier (1981) introduced the concept of offshore closure depth defining a limit beyond which no measurable bed level variations due to wave and current motion are assumed to occur (approximately 20 to 25 m based on outer limit of Hallermeier).

This limit may also be identified on the basis of field observations related to:

- **transition in sediment size;** along many micro-tidal coasts the transition from relatively coarse sand near the coast to relatively fine sand offshore occurs in depths of about 20 m; along meso/macro-tidal coasts there may be another transition from finer to coarser sand in depths seaward of -15 m line due to the presence of longshore tidal currents winnowing the fines from the sea bed;
- **transition in slope;** bed consisting of nearshore coarser sand has a markedly steeper slope;
- **transition in bed forms;** pronounced periodic bed forms are generally absent seaward of 25 to 30 m depth line;

- **transition in observed bed level variation:** maximum observed bed level variations seaward of the 20 m depth line generally are less than 0.1 to 0.2 m.

The absence of morphological change, however, does not mean that there is no sediment activity and no net sediment transport. Any activity may lead to exchanges of sediment between the shelf, the shoreface and the surfzone. Knowledge of the processes at the interface between these zones may therefore be important for long-term coastal sediment budget studies. This need not be limited to cross-shore sediment movement, but also extends to longshore sediment movement. Gradients in longshore sediment transport near the seaward boundary of the surfzone (where there is significant exchange between the surfzone and the upper shoreface) may also be important for the coastal sediment budget.
Forcings on geological temporal and spatial scales have been determined in the geological history. These forcings are the boundary conditions for the present-day spatial and time scales of interest here. Boundary conditions are the form of the basin or ocean (determining tidal amplifications and currents), the exposure to wind (fetch length for waves), the exposure to swell waves, the bathymetry of the inner shelf and shoreface (determining tidal currents and wave shoaling) and the composition of the sea bed.

1.3.2 Synthesis of bed forms and other bed states

Bedforms are a primary cause of hydraulic roughness of flows over sediment beds and may severely modify the flow field in the near-bed region. In waves or currents, wave ripples or current ripples and dunes play significant roles in the suspension of bed sediments. The bedform height determines the active layer thickness at the bed surface, which is important for transport and morphological computations over sediment mixtures. Moreover, the presence of ripples may lead to large phase differences between sediment suspension in vortex shedding from the ripples and orbital wave motion, which may invert the sediment transport direction. Certain bedform types occur only in a limited range of flow conditions and sediment sizes. Inversely, the presence of certain (relict) bedform types or their deposits may indicate the flow conditions during their creation.

Definitions and prediction of occurence
Despite a number of concerted attempts to find concensus in bedform classification, there is still a wide range of terminology for bedforms in use. In addition, terminology for bars, ridges and banks is often mixed with those for bedforms, while there are many indications that the formation mechanisms of these larger-scale features are fundamentally different from those of bedforms. Large-scale features such as sand waves, bars, ridges, shoals and banks are not considered in the present review. The nomenclature for bedforms includes both descriptive and genetic terms (e.g. upper plane bed versus sheet flow regime). Bedform occurrence and development depends on two factors: the nature and magnitude of the near-bed shear stress (e.g. waves, current or a combination) and on the composition of the sediment. Therefore bedform stability diagrams classify (or map) bedforms in genetic terms of sediment mobility (based on wave orbital velocity or current velocity) and grain size. However, such diagrams are not generally available for waves and combined flows due to lack of data and various shortcomings in knowledge (here only for megaripples and miniripples), as follows:

- ***superposition of bedforms*** **(e.g. Hanes et al. 2001, Van Lancker and Jacobs 2000, Boyd et al. 1988, Li and Amos 1999a)**;
- ***irregularity of the waves:*** it is not known how the ripple dimensions change in regular waves to slighty irregular to irregular to extremely irregular (various wave fields mixed, e.g. swell and sea waves from the same and different directions);
- ***hysteresis and history effects:*** reaction to changing conditions tardy for larger bedforms **(Allen and Collinson 1974, Kleinhans 2002a, Boyd et al. 1988, Li and Amos 1999a, Traykovski et al. 1999)**;
- ***sediment composition:*** wave ripples become larger with increasing grain size while current dunes and megaripples may become smaller with increasing grain size. In addition, the presence of silt seems to inhibit ripple formation. In badly sorted sediments, the bedforms do not necessarily behave conform their median or average grain size;
- ***intrinsic three-dimensionality*****:** there are various 3D forms, described as regular, bifurcated, chaotic, sinuous, serpentine, terminated, etc**. (Allen 1984, Boyd et al. 1988, Southard et al. 1990, Van Rijn 1993, Traykovski et al. 1999)**. It is not clear which forms are stable in certain energy levels, and which forms are transitional between others.

Bedform stability diagrams for waves and combined flow based on a small number of parameters must be improved to clarify both the terminology and bedform occurrence prediction. Unfortunately, the construction of such a diagram is hampered by the lack of knowledge on the fraction of shear stress that actually moulds the bedforms from the bed. On the other hand, a diagram that is successful in separating various bed states (if it can be found at all) may indicate which parameters are best for bedform dimension predictors as well.

Overview of data sets

A number of field datasets were identified in the literature (see Figure 1.3.2 for the site locations). The studies for each site are extensively discussed in **Kleinhans (2002b)**. Table 1.3.1 presents an overview of some of the datasets for the temporal and spatial scales of interest.

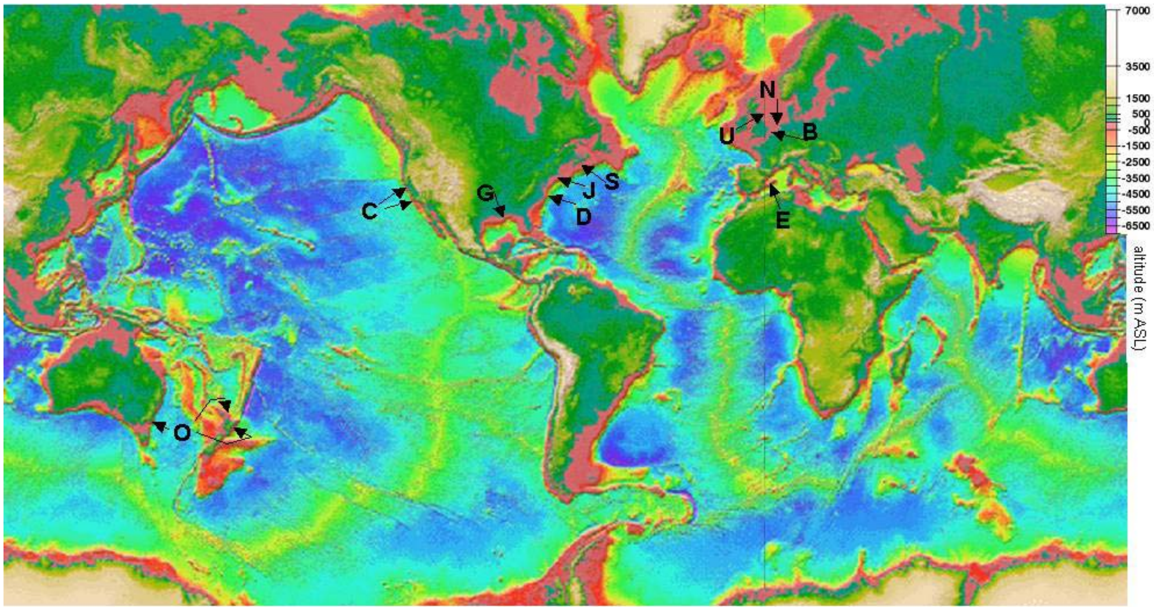

Figure 1.3.2 *Bathymetric and topographic map of the world from satellite altimetry and ship depth soundings (Smith and Sandwell 1997).*
The color scheme is such that shelf regions stand out in red colours.
The arrows and lettering refer to field sites of datasets (O=Oceania (Australia and New Zealand), C=California, G=Gulf of Mexico, D=Duck, J=New Jersey, S=Nova Scotia, U=United Kingdom, N=The Netherlands, B=Belgium, E=Ebro delta, Spain).

Set*	water depth (m)	grain size (mm)	wave height (m)	wave period (s)	tidal range (m)	max tidal velocity (m/s)
D: Duck	1.4 – 20	0.09 – 0.21	0.2 – 2.7	4 – 16	<1.2	0.1 – 0.5
S: Nova Scotia	2.4 – 59	0.14 – 0.34	0.6 – 7	4 – 16	<2	0.2 – 1.9
J: New Jersey	11	0.4	<2	5 – 18		0.1 – 0.2
B: Belgium	3 – 21	0.1 – 0.5	1.9 – 4.3	6.2 – 10.8	2.7 – 5.4	0.3 – 1.32
N: The Netherlands	3 – 25	0.14 – 0.30	0.5 – 5	4 – 13	1.2 – 2.8	0.3 – 1.1

* For references see the extended table in Kleinhans (2002b).

Table 1.3.1 *Summary of field datasets.*

Preliminary analysis

Table 1.3.2 shows the field datasets of small-scale bedforms that are available for certain ranges of waterdepth, grain size, wave height and current velocity outside the surfzone and disregarding datasets from rivers. For most ranges there is some data available from different environments to compare with possible results from the SANDPIT measurements, which will be necessary to determine the right parameters for bedform classification.

As a preliminary approach, the existing diagrams are sketched in Figure 1.3.3. Ripple dimensions and the transition to upper plane bed and sheet flow are often computed with the wave mobility numbers ψ and θ, where ψ is based on the peak orbital velocity and θ (Shields parameter) on the orbital shear velocity. The difference is that the former is computed directly from the near-bed velocity, whereas the latter is computed from the shear velocity (shear stress) and hydraulic roughness model. The Shields parameter can therefore also be based on the grain roughness only, (theoretically) excluding the form roughness by ripples. This has advantages as was pointed out by **Van den Berg and Van Gelder (1993)** for currents. The disadvantage, however is that it is partly based on a model.

grain size (mm)	**water depth (m)**			
	4 – 10	**10 – 20**	**20 – 35**	**35 – 60**
0.1–0.3	D1: Hanes et al. 2001 *(wave ripples; 0.5-5 cm; 2-120 cm; sheet flow)*	S3: Boyd et al. 1988 *(wave ripples; ?-14 cm)* SANDPIT?		S7: Li and Amos 1999b *(no motion to sheet flow; ?-10 cm)*
0.3–0.5		J1: Traykovski et al. 1999 *(wave ripples; ?-70 cm)* B1: Van Lancker et al. 2000 *(current dunes; 2-100 cm)*	B4: Williams and Rose 2001 *(wave ripples; 0.6-60 cm)*	S5,6: Li and Amos 1998, 1999a *(wave-current ripples; 1.4-12 cm)*
> 0.5				

wave height (m)	**current velocity (m/s)**			
	< 0.1	**0.1 – 0.5**	**0.5 – 1.0**	**> 1.0**
< 1	D1: Hanes et al. 2001	J1: Traykovski et al. 1999 S7: Li and Amos 1999b SANDPIT?	B1: Van Lancker et al. 2000 SANDPIT?	S3: Boyd et al. 1988 B1: Van Lancker et al. 2000
1–3	D1: Hanes et al. 2001	S7: Li and Amos 1999b J1: Traykovski et al. 1999 SANDPIT?	B4: Williams and Rose 2001 SANDPIT?	S3: Boyd et al. 1988 S5,6: Li and Amos 1998, 1999a
> 4		S7: Li and Amos 1999b SANDPIT?	SANDPIT?	

Table 1.3.2 *Bed state data outside the surfzone reported in literature classified according to conditions. Bedform type and indicative dimensions (height-length in cm) are given in italic print. Potential contribution of Sandpit measurements is indicated.*

The following parameters are probably necessary for the construction of a bedform stability diagram for waters deeper than the surfzone (most important parameter first):

1. wave-related mobility parameter (based on velocity, shear velocity or grain shear velocity of 1/3 or 1/10 largest waves),
2. current-related mobility parameter (Shields parameter θ' based on grain shear stress),
3. grain size (possibly made dimensionless as the Bonnefille grain parameter D*),
4. angle between current and waves.

Having four parameters means that a two-dimensional plot will only suffice for a small range of the other two parameters. Since the waves and currents are the most variable for a single field site, it seems logical to construct a diagram with the wave- and current related Shields mobility parameter on the two axes. This can then be done for following, perpendicular and opposing waves and currents and for fine, medium and coarse sand. Currents may modify wave ripples if they are following or opposing, whereas current ripples may be superimposed on wave ripples for perpendicular flows. The effect of grain size is also large; above 0.7 mm no current ripples but dunes only can exist, and for increasing grain size the gap between upper plane bed and sheet flow in waves increases.

Based on the existing diagrams for currents or waves, a minimum number of bedform fields must be recognised, even though they may overlap: no motion, lower plane bed, current ripples, wave ripples, dunes, wave megaripples, upper plane bed and sheet flow (Figure 1.3.3). All diagrams are based on flume data except **Li and Amos**. The current flat bed in **Van Rijn** is probably supercritical flat bed because of low water depth. In Arnott and Southard it is unclear where the distinction between small and large ripples is based on. The conjectured diagram should be constructed for a number of different grain sizes and angles between waves and currents, as the current ripples will disappear for grain sizes larger than 0.7 mm and the gap between upper stage flat bed and sheet flow will increase with increasing grain size.

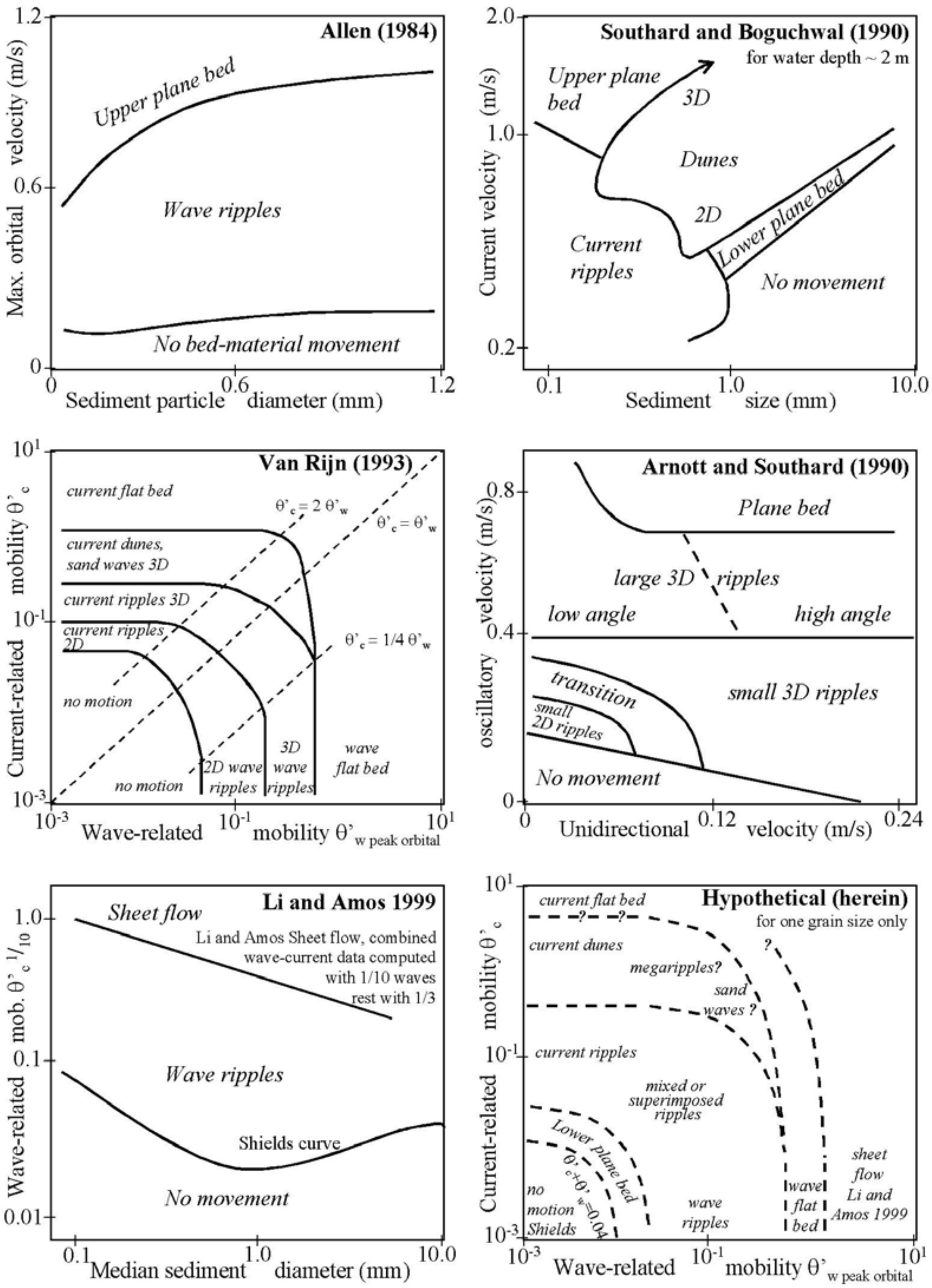

Figure 1.3.3 *Bedform stability diagrams from literature and (lower right) as conjectured herein for waves, currents and superimposed combined flow at various angles.*

Reconstruction of flow conditions from sedimentary structures of bedforms

Sedimentary structures near the sea-bed surface may provide complementary process information. The structures may be interpreted as relicts of certain bedforms or bed states, which indicates the prevalent conditions during their formation, and certain sequences of deposits may indicate a sequence of processes. When some additional parameters are known, such as water depth, tides and wave climate, the interpretations can be constrained with ripple predictors and transport threshold predictors **(e.g. Wiberg and Harris 1994)**.
Characteristic findings on sandy shorefaces are an upward fining trend in the top 10-50 cm, with structures indicating a decrease of energy as well from high-angle cross-stratification or hummocky cross-stratification or planar bedding to wave ripples. The upper part of the bed often is bioturbated **(e.g. Van de Meene et al. 1996 off the Dutch coast)**. Hummocky cross-stratification (HCS) is considered to be characteristic for combined current-wave conditions near the transition to upper plane bed (sheet flow) **(e.g. Arnott and Southard 1990, Southard et al. 1990, Van de Meene 1994)**.

1.3.3 Synthesis of sediment transport processes

Overview of datasets

The field datasets are presented in Section 1.3.2 (see Figure 1.3.2 and Table 1.3.1). Because of the practical difficulties in measuring bedload sediment transport, most workers concentrated on measuring suspended sediment concentrations with in-situ tripod measurements using a range of instruments. The sophistication of the instruments has obviously consequences for the validity of the conclusions and is often limited. For instance, **Osborne and Vincent (1996)** warn that the position of high-resolution concentration and velocity sensors relative to the underlying bedforms is important: the phase relationship is such that it could give completely opposite transport values depending on the relative position.

Main transport components outside the surfzone and in deep water

The most relevant and detailed datasets concern the following sites (Figure 1.3.2):
D: Duck (USA), **Wright et al. (1991), Li et al. (1996), Hanes et al. (2001), Lee et al. (2002)**.
S: Nova Scotia (Canada), **Boyd et al. (1988), Li and Amos (1998, 1999a,b)**.
J: New Jersey (USA), **Traykovski et al. (1999)**.
B: Middelkerke (Belgium), **Vincent et al. (1998), Van Lancker et al. (2000), Williams and Rose (2001)**.
N: Terschelling and Holland coast (The Netherlands), **Van der Giessen et al. (1990), Guillén and Hoekstra (1996, 1997), Ruessink et al. (1998, 1999), Houwman (2000), Van de Meene and Van Rijn (2000)**.

At the Dutch Terschelling site (and probably along the Holland coast and at Duck, New Jersey and Nova Scotia as well), there is a delicate balance with no significant cross-shore sediment transport at the seaward boundary of the surfzone. However, there is a strong tide-, wind- and wave-driven longshore sediment transport.
At the seaward boundary of the surf zone, the net cross-shore suspended sediment transport during storms is usually seaward due to undertow, gravity transport or (decoupling) long waves at the New Zealand, Australia, Dutch and Duck sites. The undertow and the gravity effect always give seaward suspended transport, but the long waves may also give a landward component depending on phase lags in suspension and long wave orbitals. The presence of ripples may cause important phase lag effects between gravity waves and suspension, which can also lead to reverse net suspended transport directions (see chapter on bedforms).
The cross-shore bedload transport on the other hand is often in the landward direction, as inferred mostly from ripple migration directions, and dominates in fair weather and larger water depths (Nova Scotia, Australia, New Jersey, Duck) due to wave asymmetry and possibly (that is, theoretically but not observably) Longuet-Higgins streaming and gravity-driven transport. In fair weather and swell waves off Duck, the cross-shore component of suspended sediment was also directed landward.
On the one hand, the surfzone of sandy coasts seems to be largely decoupled from the shoreface and shelf on the annual time scale. Sediment transport rates on the shelf and shoreface (deeper waters) are orders of magnitudes smaller than in the surfzone. On the other hand, the balance between offshore and onshore transport components at the seaward surfzone boundary is delicate and may depend on small cross-shore fluxes and gradients in longshore transport. The cross-shore and longshore sediment transport in deeper waters may be small but is certainly not insignificant, especially not during storms. Sedimentary structures indicate depths of activity in the order of bedform heights, and the presence of sand waves and large current megaripples also indicate significant transport. From this activity it can be inferred that there may be significant gradients in

cross-shore and longshore sediment transport for different grain sizes. Concluding, the exchange of sediment between surf zone, shoreface and shelf may be important for coastal sediment budgets on longer time scales (decades), but virtually nothing is known on the order of magnitude and the direction of the net exchange (for different grain sizes).

Only a small number of datasets are available to estimate the order of magnitude of the sediment transport. A limited dataset of bedload transport in large waterdepth in the Dutch sector of the North Sea is given by **Van de Meene (1994, also Van de Meene and Van Rijn 2000)**. The Van de Meene data set is only available parallel to the current, which is the shore-parallel net transport for half a tidal cycle.These studies indicate a transport rate of 25-75 m^3/m/yr depending on the location along the coastline and the depth. **Van Rijn (1997)** combined a model with observations of migrating dredging pits and ridges. The net annual cross-shore transport (also including pores) computed by Van Rijn (1997) was 10 ± 10 m^2/year at 20 m depth, and 0 ± 10 m^2/year at 8 m depth near the Dutch coast.
Migniot and Viguier (1980) present information of tracer studies using radioactive sand tracers in the Gulf of Casgogne north of Biaritz (France) facing the Atlantic Ocean with a severe wave climate. The experiments were carried out at depths between 6 and 22 m in the period between 15 September and 15 December 1975 (autumn and winter) in conditions with almost normal incident waves. The results show significant particle movement (fine to medium coarse sand of 0.1 to 0.8 mm) with transport rates of about 0.5 m^3/m over 3 months at a depth of 22 m upto transport rates of about 80 m^3/m over 3 months at depths of 6 to 8 m.

Wave groupiness
Infragravity waves may determine the direction of wave-driven suspended sediment transport, whether they are coupled to the gravity wave field (outside the surfzone) or decoupled (inside the surfzone), **(Ruessink 1998)**. In addition, recent measurements of intrawave flow and suspension **(Williams et al. 2002, Vincent and Hanes 2002)** demonstrate that groupiness of waves at large water depths has a significant increasing effect on suspended concentrations. Due to the time lag of suspended settling, the subsequent large waves in the group are able to increasingly suspend sediment, called 'pumping up' mechanism. In addition, the net settling velocity of the sediment is decreased by the near-bed flow.

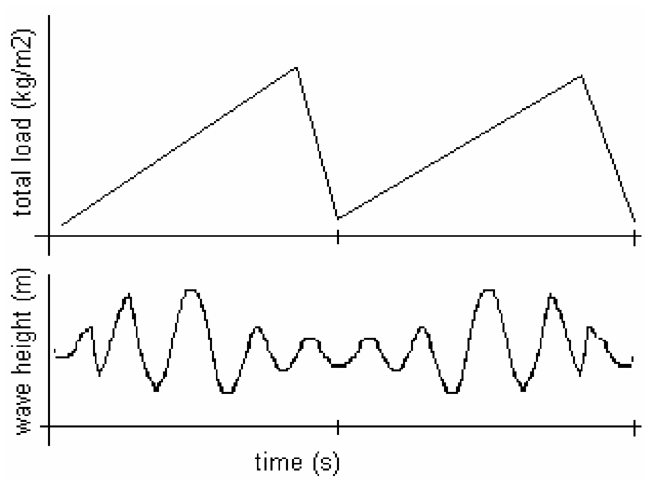

Figure 1.3.4 *Effect of wave groups on the suspended sediment load (schematically after Vincent and Hanes 2002); increasing concentration towards end of wave groups is due to the pumping up effect.*

Wave-current interactions
There is a complex interaction between ripples, the wave boundary layer and the overlying currents **(Nielsen 1992, Van Rijn 1993, Fredsøe et al. 1999, Houwman 2000, Lee et al. 2002, Smyth et al. 2002)**. Because of additional current roughness caused by waves, the near-bed velocities and (modelled) current-related sediment transport decrease with a factor two compared to the net tidal current only.
At Nova Scotia, the wave-current interaction was found to increase the grain shear stress within the wave boundary layer **(Li et al. 1997)**. When either waves or currents are weak, the enhancement was limited to only 5%, while it was 20% with equal wave and current shear stresses in the same direction (within 30°). For waves and currents perpendicular, the shear stress enhancement again was only 5%.

Thus the reaction of shear stress above and within the wave boundary layer are opposite: the velocity above the boundary layer is smaller in wave-current interaction due to increased apparent roughness, whereas the shear stress is larger within the boundary layer. This is important for bedload transport and reference concentrations: the net suspended flux is also smaller in wave-current interaction due to the relatively lower current velocity, whereas the bedload transport and reference concentration are larger within the boundary layer due to the higher shear stress in the wave boundary layer. The effects are not well understood in orthogonal waves and currents.
Moreover, in swell the vortices shed from small ripples enhanced the exchange above and below the wave boundary layer, leading to higher sediment concentrations above the boundary layer in swell than in storm. In storm conditions on the other hand, strong currents prevented the vortices from extending beyond the boundary layer **(Lee et al. 2002)**.

Influence of rivers
Over time, rivers have delivered enough silt and clay sediment for the formation of a mid-shelf mud belt off California, possibly in the Gulf of Mexico and also off the Ebro delta **(Puig et al. 2001)**, where the conditions (deep waters and sheltered conditions, respectively) favour deposition of this fine material. For the present study, the mud and sand delivery at that time scale can be neglected and the presences of the mudbelt can be taken as it is. In what follows, the location of the rivers is assumed to be in temperate climate zones, and only indirect effects on the flow will be considered.
The presence of fresh water, on the other hand, may have an effect on sediment transport on the shoreface. In the North Sea basin the river Rhine delivers enough fresh water to generate density-driven shoreward currents. These currents may cause a significant shoreward sediment transport (on the annual scale) outside the surfzone in water depths at least up to 20 m, although this has not yet been demonstrated with measurements. **Van Rijn (1997)** computed for the Dutch coast that the contribution of density-driven flow to the cross-shore sediment transport is of the same order and at least of secondary importance compared to tidal and wave-driven net cross-shore sediment transport. It is conceivable that the density-driven current is more important in wet years with higher river discharge or with higher discharge peaks.
The density stratification by the Rhine river plume is far from uniform. Apart from variations in river discharge, wind and wave conditions, there are two regular variations in the stratification. The first is a semidiurnal oscillation between a highly stable stratification and nearly full vertical mixing due to tidal straining **(Simpson and Souza 1995)**, which takes place in the Rhine ROFI and elsewhere. The second is a tidal modulation of the river discharge leading to a pulsed discharge of fresh water and consequently a train of fresh water lenses.

Influence of sediment sorting
The grading of seabed sediment has a first order effect on the sediment transport directions, mostly because of grain size-selective suspended sediment transport (coarser sediment in bedload mode, finer sediment in suspended load which may be in a different direction). In some cases graded sediment is segregated due to size-selective advection of suspended sediment **(Vincent et al. 1998, Van Lancker et al. 2000)**. In and just outside the surfzone this led to a sea-ward fining trend on the Dutch and Duck shorefaces. In others cases it is dominated by vertical sorting in bedforms (Anthony and Leth 2002). Also on the tops of local topographic highs (e.g. shoreface-connected ridges off the Dutch coast) the sediment is coarser and better sorted due to increased winnowing by wave action **(e.g. Van de Meene 1994)**. As a consequence, the sea-bed surface may become armoured. A positive feedback for winnowing by suspension is that ripples become even larger for coarser sands.
The depth of depletion of fines is related to the thickness of the so-called active layer. Interestingly, model computations are extremely sensitive to assumptions of using a single grain size, many grain size fractions and surface armouring **(Lee et al. 2002, Reed et al. 1999)**. If bed armouring must be represented in models, then detailed vertical tracking of the grain size profiles is necessary as well (history effects and graded storm beds). These effects of grading in the bed and armouring of the bed surface are well known to be important in rivers **(e.g. Ribberink 1987, Kleinhans 2002a)**. In the presence of ripples or dunes in the river, the bedform height and variation in height indeed determines the active layer thickness, while the vertical sediment sorting within the bedform (and in waning discharge or ‘storm’ sequences) create vertical grading in the bed.

Influence of mud
The natural flux of fine sediments (size<0.05 mm) is generally concentrated in a relatively narrow coastal zone of about 10 to 20 km due to geostrophical effects. Field observations show that there is a pronounced cross-shore gradient in suspended sediment concentration, with the larger values occurring near the coast. The total

(gross) flux of fine sediments along the Dutch coast is estimated to be about 20 10^6 ton/year or 50,000 ton/day. Assuming a uniform cross-shore distribution of the longshore mud transport, this is equivalent to about 2.5 ton/day/m or 0.03 kg/s/m. Taking a mean discharge of about 3 m^3/s/m (depth of 10 m and current of 0.3 m/s), the mean sediment concentration is about 0.01 kg/m^3 or 10 mg/l.

The sand mining activities affect the transport and fate of fine grained sediments in two ways:

- large amounts of fines can be mobilised and released in the environment during the sand mining activities,
- fine grained sediments may accumulate temporarily or permanently in the resulting deep sand pits.

The sediments of the North Sea bed generally contain a few percentage of fines (between 1% and 3 %). Suppose that the sand mining is carried out with a modern, large suction hopper dredger with a capacity of 20,000 m^3/hr in a sandpit with a diameter of 300 m. This would imply a sand production of 60,000 to 120,000 m^3/day, i.e. 0.1 to 0.2 10^6 ton/day, yielding 1,000 to 6,000 ton of fines per day. This amount of fines is mobilised by:

- breaching of the sea bed during the suction activities,
- overflow of fines during the filling of the hopper.

The amount of fines released in the environment can be controlled by strict working procedures. However, if no regulations are set, it is estimated that 10% to 50 % of the fines present in the seabed sediments can be mobilised, as a result of the breaching processes and/or the overflow. Hence, it is estimated that for a sand mining production of 60,000 to 120,000 m^3/day, an amount between 100 and 3,000 ton per day of fines is released in the environment.
This amount should be compared to the natural flux of fine sediments, which is currently estimated at about 20 10^6 ton/yr, or 50,000 ton/day (Dutch coast). Considering the width of the sandpit of 300 m and assuming a uniform cross-shore distribution, the natural fine sediment flux per 300 m width is estimated at about 900 ton/day. Hence it is concluded that the amount of fines mobilised (100 to 3,000 ton) by the sand mining can be of the same order of magnitude as the local natural sediment flux, depending on the location of the mining pit from the shoreline and the dredging technique applied.
Another source of mud is the overflow plume from the dredging vessels (during mining activities). These overflows may be released in clouds of sediment, or either be mixed with the environmental water, or behave as a density current over the seabed. The behaviour of the sediment in the vicinity of the mining activities, released through one of these three modes is considerably different. Clouds of sediment can behave as individual entities; within these clouds, segregation of coarse and fine sediment material may take place, known as convective settling.
Whether the overflow behaves as a plume that is rapidly mixed with the environment, or as a density current over the sea floor, depends on the overflow conditions and the types of sediments involved. The impact in the direct vicinity of the dredging vessel is the largest. The fine sediments mobilised during breaching are for the major part directly mixed over the water column, as the breaching process generates additional turbulence.
At present, it is not known whether all or part of the sediments, released during the sand mining process, will be reworked into the seabed, either temporarily, or permanently.

1.3.4 Recommendations

Sand transport processes and sediment and bedform dynamics have been reviewed with emphasis on the measured processes on the shoreface between the seaward edge of the surfzone and the upper continental shelf on time scales from seconds to a year. The studies reviewed here were done at Nova Scotia, at Duck, New Jersey, southeastern Australia and New Zealand, and in the North sea off the UK, Belgium and the Netherlands. Each environment has its own specific forcings and processes, which emphasises the need for long-term synchronous field measurements of various parameters at the site of interest. In general, bedload transport becomes more important than suspended load transport for increasing water depth (except during severe storms or swell). Various types of ripples prevail, but in the heavy storms the (transition to) upper plane bed states do occur at water depths far beyond the depth of morphological closure of the surfzone.

An overview of (quantitative) field data sets of sand transport in the coastal zone is presented in Table 1.3.3. A reasonable number of data sets is available for the shallow surf zone. Some data sets are available for the depth zone from 3 to 10 m. Hardly any data sets are available for deep water with depths larger than 10 m. Sand transport under storm wave conditions has only been measured at the Duck site (USA) in the period 1994-1998, on the Flemish sand banks of the southern North Sea in the period 1992-1995 and in the surf zone of Egmond site (North Sea) in 1998. Most of the data refer to the current-related suspended transport component; the wave-related suspended transport has only been measured in the Deltaflume of Delft Hydraulics and at the Egmond site (The Netherlands) in 1998.

There are few reported measurements of bed forms and associated bed load transport, particularly for the deeper parts of the shoreface. Our present sand transport formulations have been calibrated on the basis of the available surf zone data (in the shallow zone from 1 to 3 m). It has yet to be proved that these formulations yield good predictions for the deep water zone. The predicted sand transport rates for deep water (middle and lower shoreface) may suffer from serious errors due to scale effects (water depth effects).

A number of shortcomings of present knowledge are identified:

for the shoreface conditions, shear stress and hydraulic roughness models give widely varying results and have not been tested and calibrated a range of datasets; this leads to high uncertainties concerning the bed shear stress components for sediment transport;

there are many environments with mixed waves and currents, but interactions between waves and currents are not well understood;

there is no concensus on definitions of bedforms and states, especially in conditions with both waves and currents; in addition the genesis of a number of bed states is not well understood;

coastal, near-bed density-driven currents derived from riverine fresh-water outflow can cause a net shoreward current with a potentially first-order effect on annual sediment transport, but this effect has not been quantified empirically;

the exchange of sediment between surf zone, shoreface and shelf may be important for coastal sediment budgets on longer time scales (decades), but virtually nothing is known about the magnitude and the direction of the net exchange (for different grain sizes);

there are very few datasets with measurements of both bedload and suspended load transport and hydrodynamics at high near-bed resolutions, and none that allow the probabilistic integration to annual transport on the shoreface.

These shortcomings lead to the recommendation for extensive field measurements within the SANDPIT Project at a site in the Dutch sector of the North Sea with mixed waves and currents. Based on the review results, the characteristics at this site regarding hydrodynamics and sediment dynamics are given hereafter.
The North Sea has only small swell waves and is storm-wave dominated. Furthermore tide- and wind-driven currents occur throughout the year. Density-driven currents from river Rhine outflow play a secondary role off the Dutch coast. Consequently wave-current interactions and wave groupiness will play an important role throughout the year. Near the 10 m waterdepth, intermediate and storm waves dominate the sediment dynamics, whereas near the 20 m waterdepth, tidal and wind-driven currents dominate the sediment dynamics.
Consequently, various wave- and current-generated bed states can be expected. The vertical sediment sorting and the active layer thickness of the bed are related to these bed states. However, the bedforms on the seabed will be obliterated frequently by fishing activities, and the sediment will be vertically mixed to a depth of 0.1-0.2 m.

The approximate morphological depth of closure at a time scale of 50 years is approximately at 10 m depth; the approximate sediment transport depth of closure (above which significant exchange take place between the upper shelf and surfzone) at the time scale of 50 years is unknown but probably between 10-20 m depth. The net annual suspended sediment transport may be in the offshore direction, while the bedload will be onshore. The net annual longshore sediment transport rate is an order of magnitude larger than the cross-shore sediment transport rate.

Depth	Rivers and estuaries	Coastal seas		
		calm to moderate waves $H_s/h<0.3$	**minor storm waves $H_s/h=0.3\text{-}0.6$**	**major storm (breaking) waves $H_s/h>0.6$**
Shallow depths 1 to 3 m	Many data sets (P)	20 sets Maplin Sand, UK, 73-75; 20 sets Boscombe, UK, 77-78 (0.2-0.3 mm; qsc; W1)	10 sets Egmond 89-92; (0.2-0.4 mm; c; qsc; K, W2) 6 sets Egmond 98;(0.2-0.4 mm; c ;qsc; qsw; S)	10 sets Delta flume of DH (0.15-0.35 mm; c; S)
Medium depths 3 to 10 m	Many sets Mississippi (P) Nile (G,A1,2) Rijn-Waal (G) E+W Scheldt (V)	5 sets Delta flume of DH (0.15-0.35 mm; c; qsw; S)	10 sets Duck 94 (0.15-0.3 mm; c; qsc; M1)	5 sets Duck 94 (0.15-0.3 mm; c; qsc; M2)
Large depths 10 to 30 m	Many sets Mississippi (P) Outer Thames (W1) E+W Scheldt (S)	5 sets Canadian shelf 1999 (0.2-0.3 mm; qb; A3); 1 set Hauraki Gulf, N. Zealand (0.3-0.5 mm; c; H) 1 set North Sea (0.2 mm; c, qb and qs; V2)	1 set Duck shelf (0.1-0.2 mm; c; M1); 1 set Flemish Banks, N. Sea (0.4-0.5 mm; c; W3)	none
Wave tunnel data	-	some data (Japan)	many data sets in flat sheet flow regime; regular waves (0.13-1 mm; S)	none

S= **SEDMOC database**

A1= **Abdel-Fattah, S., 1997.** Field measurements of sediment load transport in the Nile River at Sohag. Technical report, HRI, Delta Barrage, Egypt

A2= **Amin, A.A., 1999.** Experimental approach to bed load transport of slightly non-uniform sediment. M.Sc. Thesis, IHE, Delft, The Netherlands

A3= **Amos, C.L. et al., 1999.** Ripple migration and sand transport under quasi-orthogonal combined flows on the Scotian Shelf. Journal of Coastal Research, Vol. 15, No. 1, pp 1-14

G= **Gaweesh,, M.T.K. and Van Rijn, L.C., 1994.** Bed load sampling in sand-bed rivers, Journal of Hydraulic Eng., ASCE, Vol. 120., No. 12, p 1364-1384

H= **Hume, T. et al., 1999.** What happens at the seabed off a headland during a tropical cyclone. Coastal Sediments, Long Island, USA, pp. 1836-1851

Vincent, C.E. et al., 1999. The control of resuspension over megaripples on the continental shelf. Coastal Sediments, Long Island, USA, pp. 269-280

K= **Kroon, A., 1994.** Sediment transport and morphodynamics of the beach and nearshore zone near Egmond, The Netherlands. Doc. Thesis, Dep. of Physical Geography, Univ. of Utrecht, The Netherlands

M1= **Miller, H., 1999.** Field measurement of longshore sediment transport during storms. Coastal Engineering, Vol. 36, p. 301-321

M2= **Madsen, O.S. et al., 1993.** Wind stress, bed roughness and sediment suspension on the inner shelf during an extreme storm event. Continental Shelf Research, Vol.13, No. 11, p. 1303-1324

P= **Peterson, A.W. and Howells, R.F., 1973.** Compendium of solids transport data for mobile boundary data. University of Alberta, Dept. of Civil Eng., Report HY-1973-ST3

V1= **Voogt, L., Van Rijn, L.C. and Van den Berg, J.H., 1991.** Sediment transport of fine sands at high velocities. Journal of Hydraulic Eng., ASCE, Vol. 117, No. 7, p 869-890

V2= **Van de Meene, 1994.** The shoreface-connected ridges along the central Dutch coast. Doc. Thesis, Dep. of Physical Geography, University of Utrecht, Utrecht, The Netherlands

W1= **Whitehouse, R.J. et al., 1996, 1997a, 1997b** Sediment transport measurements at Maplin Sands, Outer Thames Estuary. Report TR 15, HR Wallingford, England

Sediment transport measurements at Boscombe Pier, Poole Bay. Report TR 27, HR Wallingford, England

Sediment transport measurements at Foulness, Outer Thames Estuary. Report TR 33, HR Wallingford, England

W2= **Wolf, F.C.,J., 1997.** Hydrodynamics, sediment transport and daily morphological development of a bar-beach system. Doc. Thesis, Dep. of Physical Geography, Univ. of Utrecht, The Netherlands

W3= **Williams, J.J. et al., 1996.** Observed suspended sediments in storm conditions. Coastal Engineering 3, p. 3257-3269

Table 1.3.3 *Overview of field data sets of sand transport in the coastal zone (qb=bed load transport; qsc=current-related suspended transport; qsw=wave-related suspended transport; c=time-averaged concentration)*

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1.4 Dynamics (flow, waves and morphology) of pits

1.4.1 Introduction

The present summary is based on an extensive review of the dynamics and morphology of sediment mining pits (**Van Rijn and Walstra, 2002**).

1.4.2 Currents

The influence of the deepened area (pit) on the local current pattern (tide and wind driven) depends on the:

- pit dimensions (length, width, depth),
- angle between the main pit axis and direction of approaching current,
- strength of local current,
- bathymetry of local area (shoals around pit).

Generally, the dimensions of the deepened area are so small that there is no significant influence of the deepened area on the macro-scale current pattern. In most cases the current pattern is only changed in the direct vicinity of the area concerned.
Basically, three situations can be distinguished:

1. Main pit axis parallel to current

When the deepened area is situated parallel to the local current, the velocities in the deeper zone may increase considerably due to the decrease of the bottom friction, depending on the length and width of the deeper zone. Just upstream of the pit, flow contraction will occur over a short distance yielding a local increase and decrease of the flow velocity (order of 10% to 20%, depending on thel width W and upstream flow depth h_o). Flow contraction will be minimum for $W>>h_o$. The flow velocity in the contraction section can be estimated from ($h_1>h_o$):

$$u_{con}=[2h_o/(h_o+h_1)]u_o \qquad (1.4.1)$$

The length of the contraction section is of the order of the width W. Similarly, flow contraction will occur just before the downstream end of the pit. The flow velocity distribution along the main axis of the pit is shown schematically in Figure 1.4.1.
Assuming the water surface slope to be constant, the equilibrium flow (u_1) inside the pit can be described by the Chézy equation, yielding the following far-field expression ($h_1>h_o$):

$$u_1=u_o(C_1/C_o)(h_1/h_o)^{0.5} \qquad (1.4.2)$$

with u_1=equilibrium flow velocity inside pit and u_o= equilibrium flow velocity upstream of pit, h_1= water depth in pit. Generally, the flow inside the pit is somewhat larger than outside the pit ($u_1>u_o$), except when the length of the pit is relatively small (L<2W). The adjustment length λ of the flow to the new equilibrium value inside the pit is of the order of $C_1^2h_1/(2g)$, yielding values of 3 to 4 km. The exponential adjustment process can be expressed as:

$$u_x^2=u_1^2+(u_o^2-u_1^2)e^{-x/\lambda} \qquad (1.4.3)$$

Generally, the new equilibrium flow velocity in the pit is reached for L>10W and $W>>h_o$.

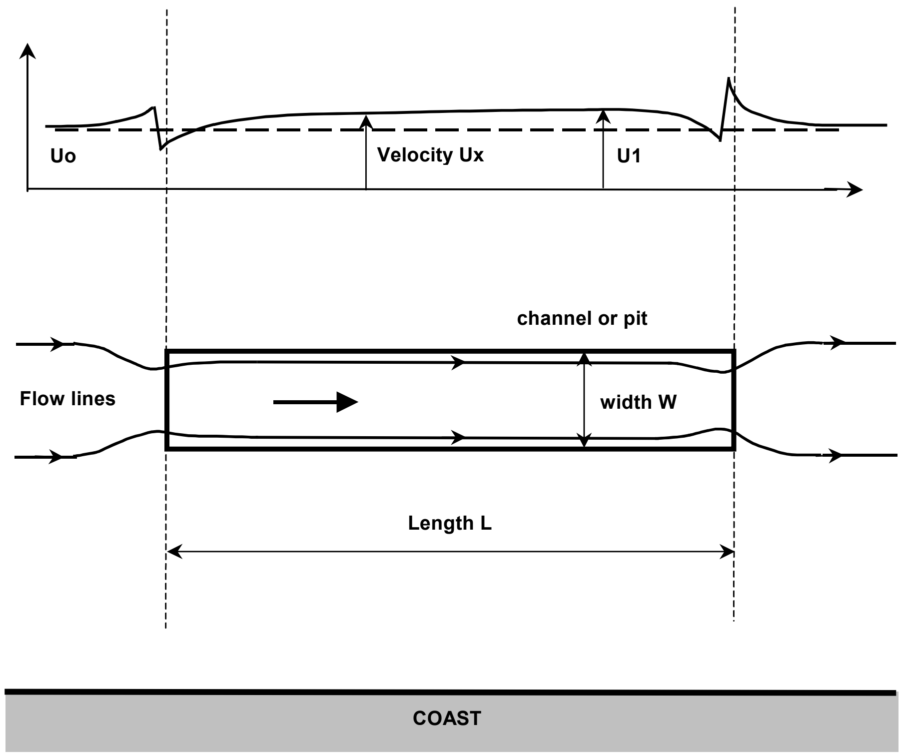

Figure 1.4.1 *Main pit axis parallel to current*

2. Main pit axis perpendicular to current

When the deepened area is situated perpendicular to the local current, the velocities in the deeper zone are reduced due to the increased water depth. This influence is most significant in the near-bed layer of the deceleration zone where adverse pressure gradients are acting, causing a strong reduction of the flow. In case steep side slopes (1:5 and steeper) flow separation and reversal will occur introducing a rather complicated flow pattern. The velocities in the recirculation zone are small compared with those in the main flow. The flow velocities in the near-water surface layers are hardly influenced by the presence of the pit (inertial effect). The depth-averaged flow velocity inside the pit (u_1) can be determined from the continuity equation (q is constant):

$$u_1=u_o(h_o/h_1) \tag{1.4.4}$$

The layer between the near surface region and the recirculation region is dominated by production of turbulence energy (mixing layer). When the deepened area is wide enough, the flow will reattach at the end of the downsloping side, as shown in Figure 1.4.2. More uniform velocity profiles will be present at the upsloping (downstream) side of the pit. In case of a narrow pit (channel) the flow pattern will be completely dominated by flow separation and flow reversal phenomena. Most probably, a large vortex will generated in the pit. Detailed mathematical studies have been done by **Alfrink and Van Rijn (1983)** and recently by **Jensen et al. (1999)**. The latter studied the morphological behaviour of small scale, steep-sided pipeline trenches using a full 3D model for oblique flow conditions. The flow in these types of trenches can only be described by a detailed 3D model as the flow pattern is strongly dominated by the flow separation zone in the steep-sided trench, often extending onto the downstream side slope. The separation zone is largest for an approach angle of 90^o but becomes weaker for smaller angles.

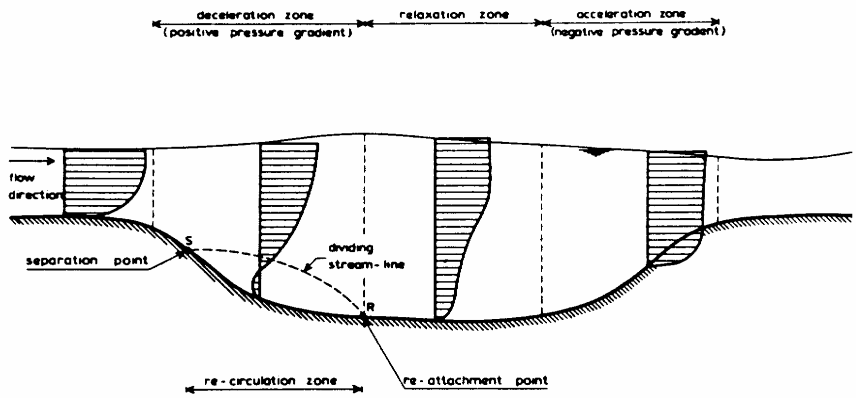

Figure 1.4.2 *Pit perpendicular to current*

3. Main channel/pit axis oblique to current

When the deepened area is situated oblique to the local current, the effects of parallel and perpendicular flow patterns are occurring simultaneously. The velocity component perpendicular to the pit is inversely proportional to the local water depth, while the velocity component parallel to the pit may increase due to a reduction of bottom friction. As a result, the streamlines show a refraction-type pattern in the pit (see Figure 1.4.3). This effect is more pronounced in the bottom region where the velocities are relatively small. Usually, there is an overall increase of the velocities in the pit when the angle α_o between the approaching current and the pit axis is smaller than about 20^o to 30^o, depending on channel dimensions and bottom roughness.
The equilibrium (depth-averaged) flow velocity vector in the pit v_l can be estimated as (Jensen et al., 1999):

$$v_l=v_o\,[(h_o\,\sin\alpha_o)/(h_l\,\sin\alpha_l)] \qquad (1.4.5)$$

with v_o=equilibrium approaching flow velocity, h_o=water depth upstream of pit, h_l=water depth in pit, α_o=approach angle, α_l= equilibriun angle in pit (**Jensen et al., 1999**). The equilibrium values will be reached if the cross width W is larger than approximately $500h_o$, ($W>500h_o$). The equilibrium flow velocity will exceed the upstream velocity if the pit depth is relatively large and the approach angle is relatively small ($<30^o$).

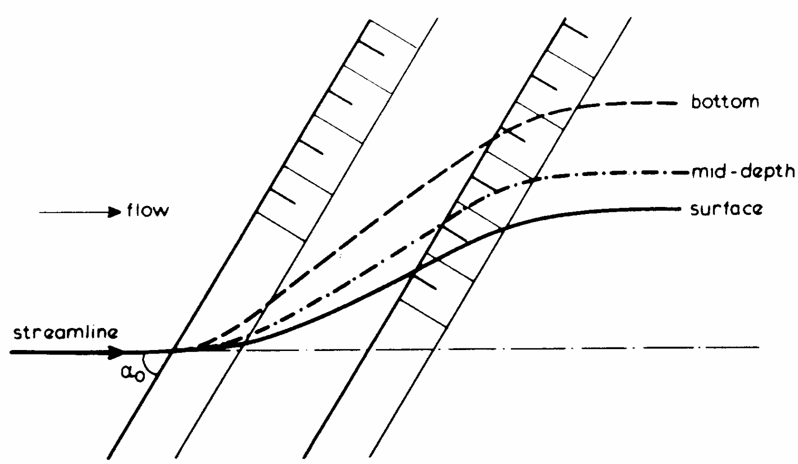

Figure 1.4.3 *Pit parallel to current*

Physical model studies have been performed at the **Hydraulic Research Station Wallingford** (**HR Wallingford, 1973**) and at **Delft Hydraulics (Boer, 1985)**. The tests at HRS Wallingford were conducted in relatively wide channels inclined to the current at angles of 0^o to 30^o. The water depth on the banks ranged

from 0.015 to 0.1 m and the difference in the depth between the channel and the banks was 0.02 m. Based on these results, it can be concluded that for angles smaller than 30°, the velocity in the channel is generally larger than that outside the channel.
The tests at Delft Hydraulics were conducted in relatively wide channels inclined to the flow at angles of 45° to 90°. The water depth outside the channel ranged from 0.1 to 0.2 m. The difference in depth between the channel and the banks was 0.1 m. In all cases, the measured velocities inside the channel were smaller than those outside the channel. **Boer (1985)** has shown that the depth-averaged velocity in an oblique channel of infinite length can be computed by the following relatively simple mathematical model based on depth-averaged parameters. Based on a numerical solution of this set of equations, Boer has presented design graphs for the current velocity and deflection angle in the middle of a channel oblique to the flow (see **Boer, 1985 and Van Rijn and Walstra, 2002**). The water depth outside the channel is in the range between 2.5 and 10 m. The channel depth with respect to the surrounding bottom is 5 m. The side slopes of the channels are 1 to 10. The bottom width is 200 m. The approach velocity is 2 m/s. The bed roughness is k_s= 0.1 m.

Jensen et al. (1999) has studied the oblique flow over a pit/channel using a 3D mathematical model (rigid lid approach), including a 3D turbulence model (K-Epsilon model). The equations are transformed onto a curvilinear coordinate system and solved using the finite volume method. The model was verified using the experimental results of HR Wallingford (1973). The model was found to satisfactorily represent the effect of refraction increasing towards the bottom.

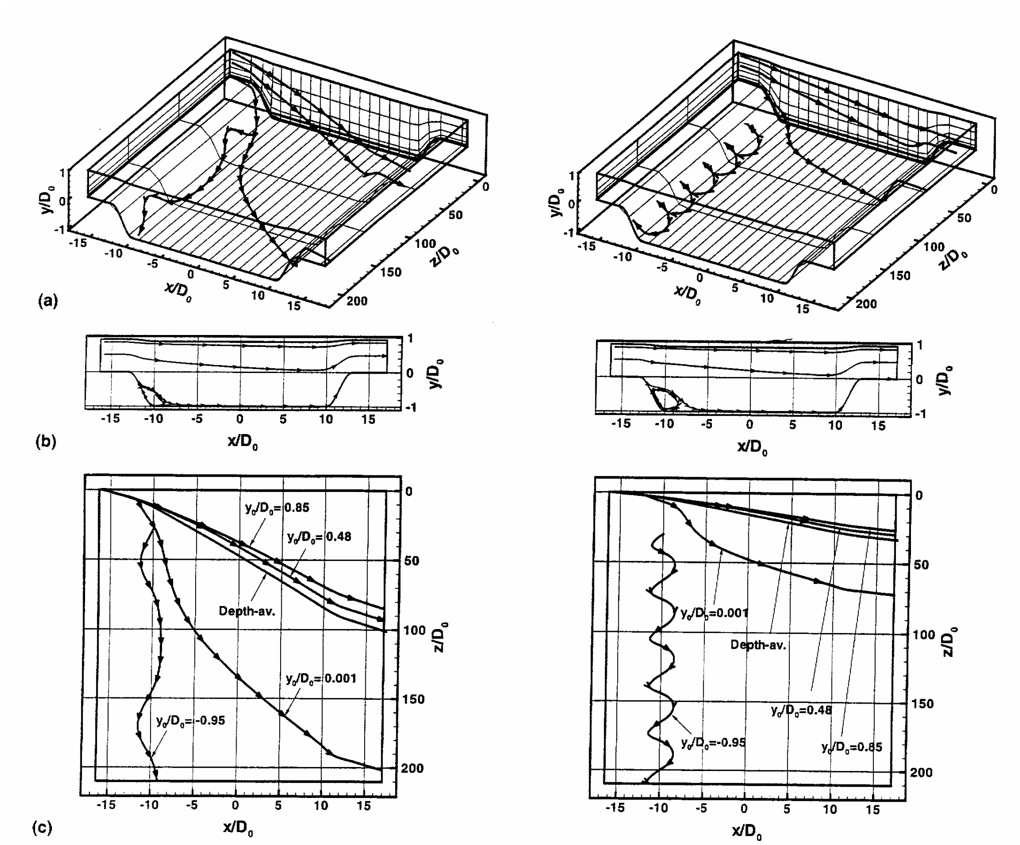

Figure 1.4.4 *Visualisation of refracted streamlines;*
Ratio of depth in channel (D_1) and depth outside channel (D_0)=2;
Ratio of channel width (W) and depth outside channel (D_0) =23
Left: Approach angle=30°
Right: Approach angle=60°

Figure 1.4.4 shows the refracted streamlines for two situations. The streamlines represent the paths of three fluid particles released at three different heights above the bed (just above the bed, at mid-depth and near the water surface). The streamline near the bed has the largest deflection. The variation of the current refraction (deflection) over the vertical may be seen as a secondary motion coexisting with the depth-averaged flow. The strongest secondary motion (strongest fraction effect) occurs for approach angles in the range of 60^o to 80^o. In that case the difference between the depth-averaged flow angle and the angle of the bed-shear stress reaches values up to 30^o. In Figure 1.4.4 a fourth streamline is shown for a fluid particle inside the flow separation zone. The slope of the channel is relatively steep; thus a large separation zone is generated. The fluid particle inside the separation zone is carried in the longitudinal direction of the pit.

1.4.3 Waves

Waves are important for the sediment transport processes due to the stirring action of the orbital fluid motions on the sediment particles. This effect is most pronounced in shallow depths where the wave motion penetrates to the sea bottom. Wave motion in the nearshore zone is influenced by refraction, diffraction, shoaling, and energy dissipation by wave breaking and bottom friction. Wave breaking in the nearshore zone may result into the generation of additional currents. Waves breaking outside the pit results in a wave-induced set-up on both sides of the pit generating a flow towards the pit. Due to the presence of the coast the flow in the pit will be deflected in offshore direction (similar to rip currents, see Figure 1.4.5).

Three situations of wave propagation over the pit are herein distinguished:

1. Main pit axis parallel to waves

When the waves are propagating parallel to the main axis of the deepened area, the wave height in the deeper area will be reduced somewhat due to the increased water depth. The wave celerity will increase in the deeper area yielding curved wave crests. Wave energy on the side slopes will be reduced by diffractional effects. Waves with a very small approach angle entering the deeper area will be refracted out of the area, see Figure 1.4.5.

2. Main pit axis perpendicular to waves

When the waves are propagating perpendicular to the main axis of the deepened area, the wave height in the deeper zone will reduce due to the increased depth. Secondary effects are reflection phenomena at the edges of the side slopes. This effects are however of minor importance for gentle side slopes (less than 1:7).

3. Main pit axis oblique waves

When the waves are propagating oblique to the main axis of the deepened area, shoaling and refraction effects will occur and a rather disturbed wave pattern may be generated above the deeper zone depending on the depth and wave approach angle. In case of a relatively small approach angle, the incoming waves may be trapped (refracted/reflected backwards) on the slope of the deepened area resulting in a significantly smaller wave height in the deepened area, see Figure 1.4.5. A cross wave pattern consisting of incoming and outgoing waves will be generated outside the pit. The critical wave approach angle (with respect to the channel axis) is about 25^o to 30^o for wave periods of 7 to 9 s, depths of 15 to 20 m, side slopes of 1 to 7 and pit depth of 5 to 10 m. Incoming waves with an approach angle larger than the critical value will cross the deeper area; the wave direction of the waves leaving the pit will more or less the same as that of the incoming waves due to compensating refractional effects on both side slopes (in case of a relatively narrow pit, Figure 1.4.5). Linear or weakly non-linear wave theory can be used to estimate these effects for each particular case.

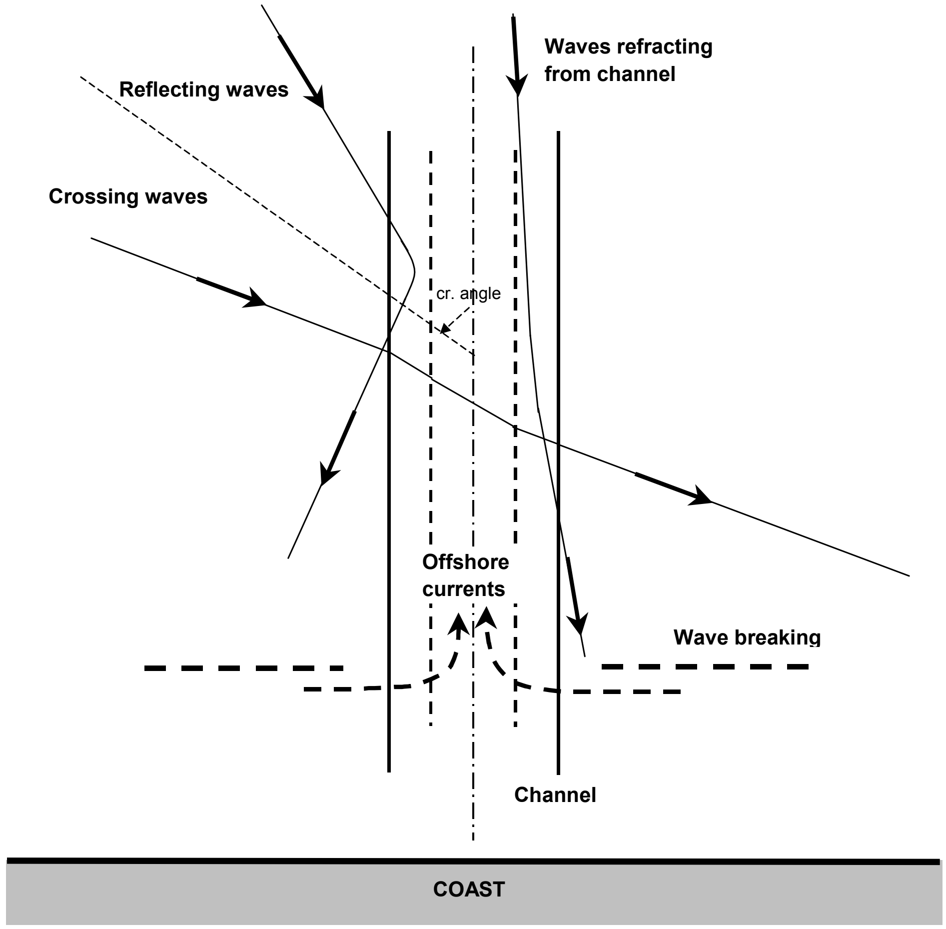

Figure 1.4.5 *Effect of deeper area on wave propagation*

1.4.4 Morphological processes

Backfilling process

Mining operations in the plain seabed will lead to a deepened area in the form of a pit, channel or trench. The morphological behaviour of the deepened mining area (pit, channel, trench) in coastal waters shows the following basic features, depending on the bed composition, the strength of the flow and the orientation of the pit to the flow direction. Assuming mobile conditions, three cases are herein distinguished (Fig. 1.4.6):

- **Unidirectional flow perpendicular or oblique to the main pit axis:** deposition at the upstream slopes and erosion of the downstream slopes of the pit resulting in migration of the deepened area in the direction of the dominant flow (mainly bed load transport); deposition in the deeper area of the pit by reduction of the sand transport capacity (mainly suspended load transport);
- **Tidal flow perpendicular or oblique to the main pit axis:** erosion of both side slopes due to bi-directional flow; deposition in the deeper area of the pit by reduction of the sand transport capacity (mainly suspended load transport);
- **Tidal flow parallel to the main pit axis:** flattening of the slopes by transport of sediment from the slopes into the deeper area of the pit by gravitational slope effects (mainly bed load transport in parallel flow).

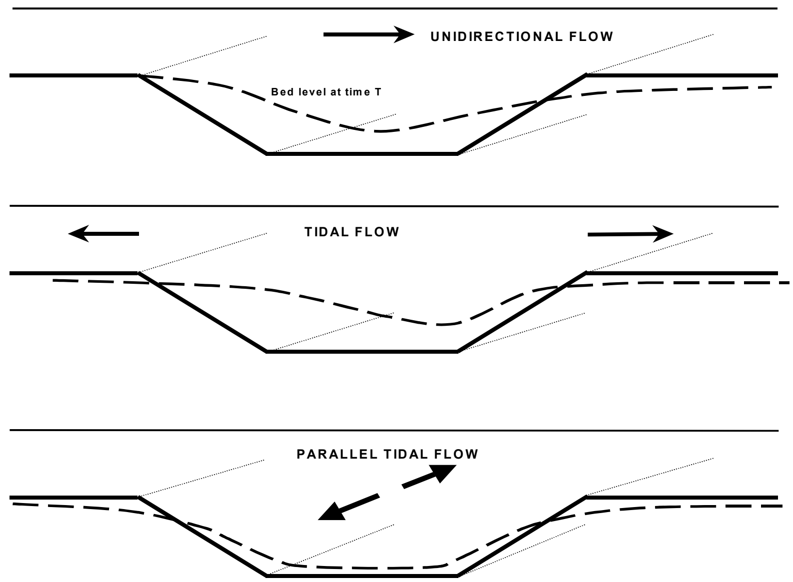

Figure 1.4.6 *Morphological development of pit*
Top: Migration in unidirectional flow perpendicular to main axis
Middle: Deposition and erosion in tidal flow perpendicular to main axis
Bottom: Flattening of slopes in tidal flow parallel to main axis

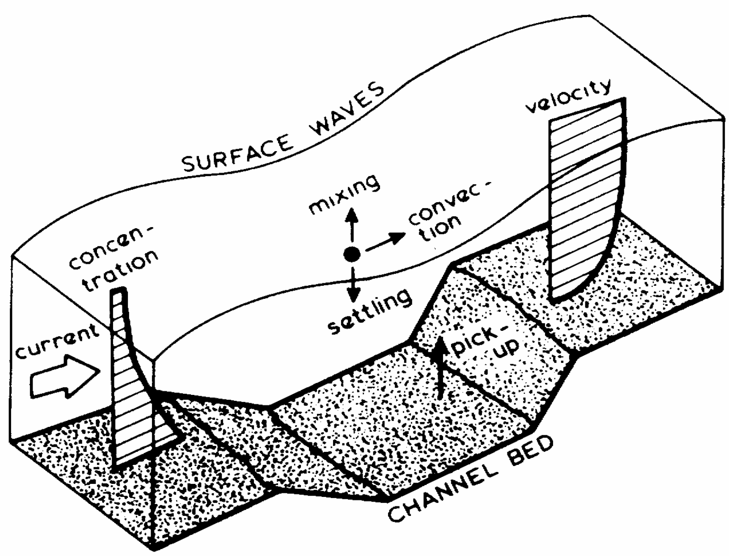

Figure 1.4.7 *Sediment transport processes in a pit perpendicular to the flow*

When a current passes a pit, the current velocities decrease due to the increase of the water depths in the pit or channel and hence the sediment transport capacity decreases (**Van Rijn, 1986a,b, 1987**). As a result the bed-load particles and a certain amount of the suspended sediment particles will be deposited in the pit. The settling of sediment particles is the dominant process in the downsloping (deceleration) and in the middle section and of the pit. In the case of a steep-sided pit with flow separation resulting in the generation of additional turbulence energy, the settling process may be reduced considerably. In the upsloping (downstream) section of the pit the dominant process is sediment pick-up from the bed into the accelerating flow, resulting in an increase of the suspended sediment concentrations. The most relevant processes in the deposition and erosion regions are: convection of sediment particles by the horizontal and vertical fluid velocities, mixing of sediment particles by turbulent and orbital motions, settling of the particles due to gravity and pick-up of the particles from the bed by current and wave-induced bed-shear stresses. The effect of the waves is that of an intensified stirring action in the near-bed region resulting in larger sediment concentrations, while the current is responsible for the transportation of the sediment. The stirring effect diminishes with increasing water depth. Thus, this effect is less important in the pit itself. These processes are schematically shown for cross flow over a long, narrow pit in Figure 1.4.7.

In case of oblique flow over a pit the sediment transport in longitudinal direction may increase considerably with respect to the undisturbed longitudinal transport outside the pit. **Jensen et al. (1999)** reports an increase of a factor 2 for an approach angle of 45^o.

In case of flow parallel with the pit axis, the side slopes of the pit are flattened and smoothed due to gravitational effects. When a sediment particle resting on a side slope is set into motion by waves or currents, the resulting movement of the particle will, due to gravity, have a component in downwards direction. By this mechanism sediment material will always be transported to the deeper part of the pit yielding reduced depths and smoothed side slopes. These effects are particularly important in conditions with a relatively narrow pit and flow parallel to the longest axis of the pit.

The prediction of sedimentation in mining pits (backfilling) basically consists of two elements:
- sediment transport carried by approaching flow to the pit, depending on flow, wave and sediment properties;
- trapping efficiency of the pit, depending on pit dimensions, channel orientation and sediment characteristics.

Effect of sand waves
Sand waves can affect the behaviour of mining pits in two ways:
- generation of sand waves inside the pit thereby reducing the depth;
- sand waves and shoals on bed just outside the pit and migrating towards the pit causing infill of the pit.

Katoh et al. (1998) studied the generation and migration of sand waves over a shoal inside a navigation channel (Bisanseto channel, Japan). The sand waves had a height of about 5 m and a length of about 100 m and were most pronounced and mobile in depths of about 15 to 25 m. The crestlines of the sand waves are essentially normal to the direction of the predominant tidal currents. The bed consisted of sand in the range of 0.5 to 2 mm. The sand wave crests were removed in the period 1981-1983 by dredging to obtain a larger depth for navigation (dredged volume of about 2 million m^3). During a period of 10 years after dredging the sand wave pattern was restored similar to the initial situation.
Rijkswaterstaat in The Netherlands studied (1981) the regeneration of a sand wave in the North Sea after removal of the crest of the sand wave in a water depth of about 20 to 25 m by dredging. The crest of one sand wave within a large-scale sand wave field was removed in 1974 by about 1.5 m over a length of about 1 km (total dredging volume of about 350,000 m^3). The sand wave field consisted of waves with a height of about 5 to 6 m and spacings of 400 to 500 m at about 30 km from the coast and about 3 km north of the navigation channel to the Port of Rotterdam. The sand wave crests are superimposed by active megaripples with a height of about 0.5 to 1 m and length of about 20 to 30 m. The maximum tidal current velocities during ebb and flood in this area are about 0.4 to 0.6 m/s. Storm waves with a height of about 3 to 4 m occur during the winter season during 3% of the time. The mean sediment size is about 0.2 to 0.3 mm. A regeneration trend with a clear increase of the sand wave height in time could not be detected within the dredged section over a period of 6 years. The sand waves were quite stable, although significant vertical and horizontal fluctuations were observed in the crest regions due to the movements of the megaripples.

Effect of mud
In muddy conditions, high concentrations of mud in suspension with accompanying high settling rates and relatively thick deposits may exist along the bed. These deposits appear to be due to the deposition of fluid mud flocs convected into the region from both upstream and downstream as a result of drifts near the bed.

These drifts can be set up by wave-driven forces, and/or horizontal density gradients induced by differences in salinity between the fresh water flow and sea water. The vertical distribution of the sediment concentrations can be characterized as a three-layer system with clear interfaces (lutoclines), as follows (**Van Rijn, 1993**):

- consolidated mud at the bottom with concentrations largber than about 300 kg/m^3; flocs and particles are supported by drag forces exerted by the escaping fluid (hindered settling effects);
- mobile (fluid) mud suspension layer (0.1 to 1 m) with concentrations in the range between 10 and 300 kg/m^3; the fluid mud can be transported by tide-, wind-, wave- and gravity-driven forces (near-bed drift velocities of 0.05 to 0.5 m/s);
- mobile dilute mud suspension (up to water surface) with concentrations in the range between 0.1 and 10 kg/m^3; the flocs and particles are supported by turbulence-induced forces; the mud suspension can be transported by tide-, wind-, wave-driven forces (velocities of 0.3 to 1 m/s)

When the sea bed contains relatively large muddy and silty sediment fractions, cohesive forces become important. Generally, the strength of the consolidated soil against erosion increases depending on the type of clay minerals, the presence of organic materials, the stage of consolidation, etc. Flume experiments show that the sand concentrations are significantly reduced (factor 30) in conditions with wave motion over a bed consisting of 25% mud and 75% sand compared to the sand concentrations above a 100% sand bed (**Van Rijn, 1993**). The stage of consolidation of the mud bed is an important factor in the erodibility of the bed material. Fresh mud deposits have a very loose texture of mud flocs which already have a low density themselves. The wet bulk density of fresh mud deposits is in the range of 1050 to 1100 kg/m^3 (95% water). The cohesive forces in fresh deposits are still very low and the material can easily be eroded. The density of fresh deposits gradually increases as interstitial water is pressesd out by gravity forces. This process of consolidation initially goes relatively fast, but gradually slows down. The strength against erosion increases with increasing consolidation and density.
Wave action over a muddy bed may generate a high-concentration fluid mud layer close to the bed. The sediment concentrations in this layer may be of the order of 100 to 300 kg/m^3. The sediment concentrations above this layer generally are an order of magnitude smaller. Tide-driven, wave-driven, wind-driven or gravity-driven (on slopes) currents are able to transport the fluid mud layers towards a nearby pit resulting in excessive deposition on short term time scales (storms, monsoon waves).

Information on modelling of fluid mud transport is given by **Odd and Cooper (1989), Kranenburg and Winterwerp (1997), Ali et al., (1997) and Winterwerp et al. (2002)**.
Recent studies (**Winterwerp, 1999, 2001, 2002**) suggest that the formation of fluid mud from suspensions of cohesive sediments in depressions in the sea bed is the result of supersaturation of the suspension, that is that the transport capacity of the flow is exceeded. These studies also suggest that the generation of fluid mud already takes place at relatively small concentrations (< 1 kg/m^3). **Winterwerp et al. (2001)** estimate that the saturation concentration in the Dutch coastal zone amounts to a value in the range of 0.1 to 0.5 kg/m^3.
The transport capacity of mud (or saturation concentration) in a 20 m deep sandpit, located at 20 m water depth, would decrease by a factor 5 to 10 as a first estimate, assuming a decrease in velocity proportional to the increase in cross section (continuity considerations). This would imply that fluid mud may already be formed in such pits at concentrations of about 0.05 kg/m^3. Assuming a fluid mud concentration of 100 kg/m^3, such a suspension would yield a fluid mud thickness in the range of 0.01 to 0.02 m. Hence, these sediment deposits may become significant due to accumulation in time. If the hydrodynamic conditions favour the formation of such fluid mud layers, it is not likely that the tidal flow will be able to re-erode these deposits. If these layers are not eroded anymore, they will consolidate, resulting in bed concentrations in the range of 100 to 500 kg/m^3.

Slope instabilities
The side slopes of a dredged pit, channel or trench should remain stable after construction.
Three types of slope instability can occur, depending on local conditions (Figure 1.4.8):

- slope collapse,

- liquefaction and successive flow slide,
- breaching and retrogressive slope erosion

The maximum slope is a function of depth or slope height, grain size and density or porosity but also to operational aspects concerning for instance the dredging activity.
Slope collapse can only occur at very steep slopes (<1:2 to 3). Liquefaction can occur in loosely packed sand layers only, and may result in a instantaneous mass flow creating very gentle slopes (> 1:10 to 15). If sufficient geotechnical data are available, the risk for slope instability can be computed by relatively simple models. Retrogressive breaching is well known in the dredging practice, as mentioned in relation to deep suction dredging, but as a mechanism of slope instability in densely packed sand it is not commonly recognised so far (**Van den Berg et al., 2002**).
Breaches under water produce a quasi-steady turbidity current, that may be the origin of massive sands deposited in channels. The characteristic bed profile in the case of breaching is a steep top and successive gentler slopes decreasing with slope height.
Risk on slope instability and slope development due to breaching, which may be of importance for mining of sand in deep pits, can be simulated with a 1D stationary model (**Mastbergen en Van den Berg, 2002**).

Factors affecting the side slope instability are: (i) excavation method applied, (ii) excavation depth and (iii) sediment properties of the deeper layers.

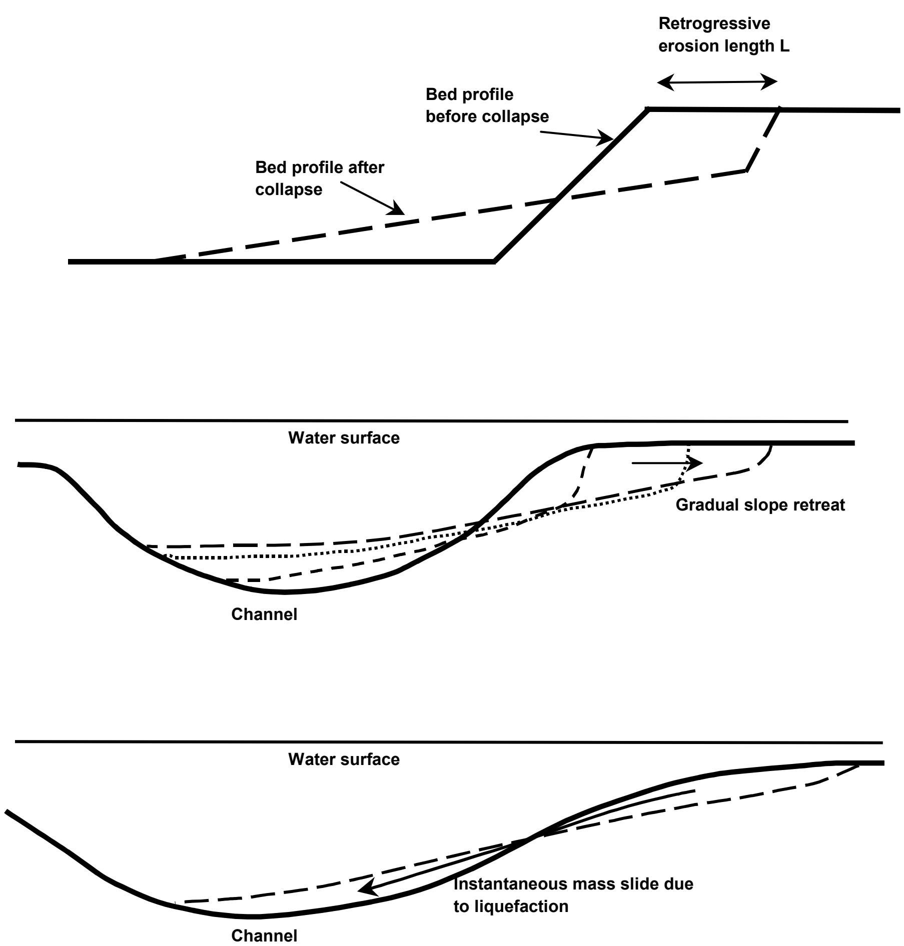

Figure 1.4.8 *Slope failure mechanisms*
Top: Slope collapse
Middle: Slope breaching and retrogressive erosion
Bottom: Slope liquefaction and mass slide

Generally applied excavation methods are: trailing suction hopper dredges, cutter suction dredges and deep suction dredge. The most suitable method is dredging with large capacity trailing suction hopper dredges, nowadays operational at large water depths. The new generation of jumbo trailing hopper dredges has a capacity of over 20,000 m^3. Cutter suction dredges are applied only in shallow water in rehandling areas. The most efficient way of sand mining, however, is the stationary or slowly displaced deep suction dredge, creating a deep, crater-like sand pit. However, its application is restricted to on-shore locations or well protected off-shore areas only, due to intolerable wave action at sea. The deep suction dredge benefits of the unique self eroding and transporting property of sand along the resulting slope due to gravity and density differences (breaching). For this reason sand and gravel extraction in lakes, along the rivers and even on land extractioning areas is performed hydraulically with deep suction dredges. New technology is being developed, but still not fully operational (fluidisation and sand by-passing systems, punaise system being a submerged deep suction dredge, not affected by surface waves).

Until recently the excavation depth allowed on the Dutch area of the North Sea is not more than 2 m. This implies that for large quantities of sand, very large sea bottom areas have to be explored. Theoretically, very deep sand extraction depths are feasible at the North Sea with deep suction dredgers until -75 m NAP due to the presence of thick layers of sand. Predicted slopes in sand of 0.2 mm are about 1 to 4 in a water depth of 25 m and 1 to 10 in a depth of 50 m.
In the case of (submerged) sand deposits, sand rehandling areas, artificial islands, etc. slope stability also plays an important role. Hydraulically placed sands but also freshly sedimentated natural sands, are loosely packed and therefore susceptible to liquefaction. In the case of deep suction dredging and slope instability due to breaching, sand is transported and resedimentates downslope, creating large zones of loosely packed sand. The underwater slope development (**De Groot et al., 1988**) turned out to be a function of grain size and sand supply per unit width and by the slope height, defining if liquefaction is likely to occur, reducing the slope dramatically. Slopes observed in loosely packed sands (0.1 to 0.25 mm) are in the range of 1 to 6 and 1 to 30.

1.4.5 Near-field and far-field impact of pits

Results of many studies related to pits in water depths of 10 to 20 m show that the physical dredging effects in the direct vicinity (within about 500 m) of the pit are minor. There is no evidence of suspended sediments falling to the seabed beyond a zone of about 500 m and causing significant changes, which may be manifested as infilling of pits by fine sediments (**Hitchcock and Bell, 2004**).
The impact of a extraction pit on the coast can at the present stage of knowledge only be estimated in rough way from the available data of existing extraction pits made in the coastal waters of the USA, Japan, UK and The Netherlands.

Four zones are distuinguished and the impact of a pit in each zone is briefly described.

- **pit at foot of beachface (-2 to -5 m depth contour);**
  - cheap and attractive method for sheltered coasts (mild wave regimes; small littoral drift);
  - infill from beachside and from seaside (annual infill rate is not more than about 3% of initial pit volume; infill rates are between 5 and 15 m^3/m/yr, depending on wave climate; filling time scale is 20 to 30 years);
  - local recirculation of sand; no new extraction sand is added to beach system;
- **pit in upper shoreface zone (-5 to -15 m depth contour);**
  - relatively strong impact on inshore wave climate due to modified refraction and diffraction effects;
  - relatively strong modification of gradients of littoral drift in lee of pit resulting in significant shoreline changes (growth of beach salients);
  - relatively rapid infill of extraction pit with sediments from landside (beach zone); annual infill rates up to 20% of initial pit volume in shallow water (filling time scale is 5 to 10 years);
  - local recirculation of sediment; no new extraction sand is added to nearshore system;
- **pit in middle shoreface zone (-15 to -25 m depth contour);**
  - negligible impact on nearshore wave climate;

- negligible effect on nearshore littoral drift;
- no measurable shoreline changes;
- new extraction sand is added to nearshore morphological system (nourishment);
- infill of extraction pit mainly from landside with sediments eroded from upper shoreface by near-bed offshore-directed currents during storm events; annual infill rate is about 1% of initial pit volume (filling time scale is 100 years);
- trapping of mud in pits (negative ecological effect);
- particle tracer studies show small but measurable transport rates, mainly due to storm waves;
- long-term deficit of sand for upper shoreface;

• **pit in lower shoreface zone (beyond -25 m depth contour);**
- no impact on nearshore wave climate;
- no effect on nearshore littoral drift;
- no measurable shoreline changes;
- new extraction sand is added to nearshore morphological system (nourishment);
- minor infill of sand in extraction pit; only during super storms;
- trapping of mud in pits (negative ecological effect);
- particle tracer studies show minor bed level variations (of order of 0.03 m over winter period) during storms.

Extraction pits in the middle and lower shoreface should be designed with their longest axis normal to the shore to minimize the trapping of sand from the nearshore zone during storm events.
The estimated time scales for the middle and lower shoreface are extremely uncertain due to lack of sand transport data at these locations.

1.4.6 Available laboratory and field datasets for model verification

The number of data sets (in cluding all boundary conditions) available for hindcast studies of the morphological pit behaviour in coastal conditions (combined current and wave conditions in depths>10 m) is rather small. Various extraction pits in the USA and in Japan have been described in the literature, but these data sets do not provide sufficient data (bathymetries and boundary conditions, see **Van Rijn and Walstra, 2002**) for reliable verification. The available data sets are summarized in Table 1.4.1. The two laboratory cases of Delft Hydraulics and the three field cases have been used in the SANDPIT Project to verify the available models.

Test Cases	Hydrodynamic data sets	Morphodynamic data sets
Laboratory	2D flume tests (Alfrink and Van Rijn, 1983; Delft Hydraulics, R1267-III,1980)	2D flume tests with bakelite (Migniot and Viguier, 1980)
		3D flume tests with bakelite (Migniot and Viguier, 1980)
		2D flume tests with sand (Delft Hydraulics, 1985)
		3D basin tests with sand (Delft Hydraulics, Z2378, 1998)
Field	PUTMOR in North Sea, Netherlands (Svasek, 2001)	Trial trench Scheveningen in North Sea, Netherlands (Svasek, 1964)
		CNEXO pit in English Channel near mouth of Seine river (Gomi and Sergent, 2004)
		Artificial sand ridge in North Sea near Hoek van Holland, Netherlands (Rijkswaterstaat, 1996; Delft Hydraulics, 1998)

Table 1.4.1 *Summary of available data sets*

1.4.7 References

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1.5 Prediction models of bed forms, sand transport and morphology

1.5.1 Bed form prediction models

Introduction

Coastal regular bedforms can be roughly classified on the basis of their geometric characteristics into large scale (wavelength of *O*~1 km), medium scale (wavelength of *O*~10 m) and small scale bedforms (wavelength of *O*~10 cm). Small scale bedforms, frequently referred to as ripples, appear when the bottom is subjected to oscillatory driving forces that, in presence of a rough bottom, induce the formation of recirculating cells producing ripple growth. The importance of studying ripples arises as a consequence of the considerable modification of bottom roughness, which in presence of such sedimentary structures is not controlled by the grain size anymore but by the ripple height. For example, from field observations (Sandy Duck'97 Nearshore Field Experiment at the Outer Bank of North Carolina), it was noticed that as unbreaking waves approached shallow waters, in some cases up to 60-70% of their energy was dissipated. This dissipation cannot be justified according to the usual estimate criteria of bottom friction factor, unless an appropriate and enhanced friction factor, which accounts for the presence of bedforms, is considered.

The consequences of small scale bedform appearance are therefore extremely important whenever the bedload sediment transport, the damping of wave motion associated to the offshore-onshore propagation, the dispersion of pollutants, etc. are considered.

As a consequence of such relevance in coastal problems, ripples have been widely studied both theoretically and experimentally in order to provide reliable predictions about their appearance, their equilibrium features, as well as their migration.

In the following a brief state of the art description of the existing models regarding ripple prediction is reported.

Prediction models

Since **Bagnold's (1946)** early experiments with oscillating trays, a large number of studies have been carried out. Most of them refer to measurements of ripple marks, carried out in laboratory or in the field. Nevertheless, a significant gap still exists between laboratory tests performed under simplified hydrodynamic conditions, and field experiments, performed in a very complex environment.

Laboratory measurements can be divided into wave flume tests and oscillating water tunnel tests. Since the former have the limits related to a reduced scale representation (i.e. limited water depths, short period and high amplitude waves in order to mobilize the sandy bottom), the latter try to overcome such limits reproducing the bottom boundary layer dynamics in a scale closer to the one of the prototype.

Wave flume tests

Inman and Bowen (1962) performed measurements of sand transport caused by waves plus currents over a rippled bed. They observed that in absence of superimposed flow, ripples were fairly symmetrical, but due to the strong suspended load associated with the wave propagation, the maximum ripple height was higher under a wave crest than under a trough. When current was superimposed, the asymmetry in the orbital velocity resulted in a flattening of the up-wave face of the ripple and in a steepening of the down-wave one, where the vortex shed became stronger and the suspended material had an initial velocity.

Homma and Horikawa (1962) showed that the ripple length versus the wave orbital amplitude ratio is basically a function of the Reynolds number defined on the basis of the wave orbital amplitude.

Kennedy and Falcon (1965) observed that for given fluid and sediment properties, once the wave orbital amplitude was fixed, there was a range of maximum bed velocities for which ripples could form with a characteristic wavelength. When the kinematic conditions are those for which ripples may appear, they spread all at once and as soon as they reach a certain height, vortices are shed from the crests; the equilibrium wavelength was observed to form at an early stage of bedform growth, increasing with the wave orbital amplitude and decreasing with the period; the equilibrium height is limited by the angle of repose of the sediments. The disappearance of ripples was found for a value of the ratio between the wave orbital amplitude A and the median sediment grain d_{50} approximately equal to 8000.

Homma et al. (1965) related ripple characteristics to wave properties, water depth and sediment grain size, their irregularity increasing with the increase of wave height. Indicating with η the ripple height, with λ the ripple length, with ν the kinematic viscosity of water and with w_s the settling velocity, the dimensionless parameters involved in the study of ripple characteristics were the following: η/λ, $\eta/2A$, $\lambda/2A$, $2U_oA/\nu$ and w_sD_{50}/ν. The first three parameters are directly related to ripple shape, the other two are respectively the Reynolds number of the flow and a Reynolds number of the sand particle size. Ripple steepness η/λ, resulted to be independent of the sediment size and is expressed as a function of $\eta/2A$, while ripple wavelength is expressed as a function of the flow Reynolds number.

Mogridge and Kamphuis (1972) performed a set of experiments in a wave flume changing the fluid viscosity, the wave orbital amplitude, the wave period T and the relative density of sediments, $s=\rho_s/\rho$, (ρ_s being the sediment density and ρ being the water density). According to their dimensional analysis, the parameters that describe the process of ripple formation are the following: $X_1=[(s-1)gd_{50}^3]/\nu^2$ that takes into account the viscosity of water, where g stands for the gravity acceleration; $X_2=d_{50}/[(s-1)T^2]$, which is directly related to the wave period; $X_3=s$, that is the sediment relative density; $X_4=\overline{A}/d_{50}$ where $\overline{A}$ is a modified orbital diameter that takes into account also the mass transport. They observed the changes in ripple geometry due to the variation of such quantities and concluded that the main effects on bedform characteristics basically depend on wave variables, while other conditions may be neglected because of their secondary importance. By using the experimental data they proposed some design curves for ripple height and wavelength, which take into account the wave orbital amplitude and the wave period. Later on, Mogridge et al. (1994) on the basis of the previous mentioned work, developed expressions for the maximum ripple length and height in the range $X_2>0.15*10^{-6}$.

Nielsen (1979, 1981) devoted much effort to determine ripple geometry as a function of flow and sediment parameters, by collecting most of the available data. He also showed the influence of wave irregularity on natural ripples. He found that ripple steepness maintains a constant value equal to $\eta/\lambda=0.32\tan\phi$, where ϕ is the angle of repose of the bed material, as far as the flow is in equilibrium under the eroding and depositing effects. At high values of the Shields parameter θ ripples become less steep; different expressions for laboratory and field conditions were derived. The ripple steepness is related to the θ-parameter. Wavelength and height were described as a function of the mobility number $U_o^2/((s-1)gd_{50})$.

Wiberg and Harris (1994) made a distinction among three types of ripples. Adopting as reference parameter d^* being the ratio between the orbital diameter and the median grain diameter.
For low values of d^*, ripple length is proportional to the orbital diameter, and ripples are defined as orbital.
For high values of d^* ripple length scales with the sediment size, and, due to the independence of orbital diameter, ripples are called anorbital.
Between these two ripple types, suborbital ripples occur, whose length depends both on the orbital diameter and the grain size.

Faraci and Foti (2001, 2002) performed a lab investigation with regular and random waves. Analyzing the results of the experiments, they found that no matter the nature of the waves, it was possible to describe ripple length and height in terms of three non-dimensional parameters, namely the flow Reynolds number Re ($=U_oA/\upsilon$), the Reynolds number Re_d($=U_od/\upsilon$) and the mobility parameter $U_o^2/((s-1)gd_{50})$.
Ripples were also found to migrate as soon as they appear with a migration velocity that depends on the flow Reynolds number and the sediment Reynolds number.

Oscillating water tunnel tests

As previously mentioned, experiments in water tunnels aim to provide additional observations of ripple characteristics in close-to-prototype situations. **Carstens et al. (1969)** identified the initial conditions of motion when at least 10 per cent of the surface grains move. According to their experience, bedforms can develop and propagate starting from a small disturbance placed on the bed with lower orbital amplitudes than those required for the spontaneous initiation of motion. They defined the incipient motion as the movement of 10-20 grains per square centimeter with a perception of rapid intensification of grain motion at increasing velocities, and found a critical velocity inside the boundary layer, by considering the drag force exerted on the grains. They observed an unstable progression from rolling grain to vortex ripples, characterized by an equilibrium state with a wavelength proportional to the wave orbital amplitude. In their experiments they often observed very irregular patches, different from brick patterns, and indicated the range of two-dimensional patterns as $A/d_{50}<775$. Three-dimensional ripples form when $775<A/d_{50}<1770$, while above the upper limit, bedforms disappear and the bed becomes flat again.
Lofquist (1978) introduced the concept of "*induced ripples*" as the bedforms generated by local perturbations. On the other hand, "*spontaneous ripple formation*" refers to the sudden ripple appearance all over an infinite sand surface at the same growth rate. In this latter case the velocity that is needed in order to observe this spontaneous growth must be at least equal to the critical one. Finally ripples may be said to grow "quasi-

spontaneously" when they are first observed in isolated areas and then spread over the entire surface in a few periods. Three-dimensionality effects were observed to be more frequent in the presence of finer sands. **Lofquist (1978)** analyzed the development in time of ripple height and wavelength. He plotted the non dimensional ripple height and wavelength, η/A and λ/A, versus the non dimensional parameter $(D_{50}/A)*m$, m being the number of wave cycles. As regard to ripple height he found the existence of an asymptote at $\eta/A = 0.24$ for the coarser sand, and at $\eta/A = 0.18$ for the finer one, once ripples had reached the equilibrium condition, thus underlining a slight effect of the grain size. The non dimensional wavelength shows one asymptote, independently of which size of sand had been used, for $\lambda/A = 4/3$. At their appearance, ripples are instead characterized by different wavelengths with respect to different grain sizes. According to these plots, ripples appeared to have reached an equilibrium condition, i.e., their height and wavelength do not vary anymore, when the parameter $(D_{50}/A)*m$ equals 1, independently of the sediment grain size. Thus ripples are mature after a number of cycles higher than or equal to A/D_{50}. The ripple equilibrium characteristics seem to confirm Mogridge and Kamphuis (1972) results, apart from a dependence on the grain size, not taken into account in the previous study. Lofquist (1978) performed also an interesting analysis about the hysteresis of a stable profile, characterized by a forced wavelength, by changing the wave orbital amplitude and period. He found out that the stable ranges are characterized by: $1.1<\lambda/A<1.6$ and $0.22>\eta/\lambda>0.125$. Moreover, he suggested that the two-dimensionality of ripples might be dependent on the mobility number and on the ratio $2A/D_{50}$.

Vongvisessomjai (1984) collected many researchers' data, obtained in wave flumes, water tunnels and in the field, and added his own water tunnel data. He distinguished an initial stage of growing ripples, characterized by an enlargement of height and wavelength, along with an increase of bed shear stresses, until optimum conditions are reached. Beyond the optimum conditions, ripple height starts to decay while the wavelength remains approximately constant. While growing ripples are pretty two-dimensional due to small turbulence or vorticity, decaying ripples, due to large turbulence, are highly three-dimensional. The optimum ripple occurs between the two stages. Ripple characteristics are different if sand or lightweight sediments were considered, reflecting a different effect of the fluid shear stress and fluid acceleration on the grains. Lightweight sediment ripples depend only on the parameter $\psi d_{50}/A$, implying no dependence on the grain size but a direct interaction with the imposed oscillatory flow. While high fluid accelerations force sand ripples to grow, experiencing a small turbulence, oscillating bed ripples, as decaying ripples, experience a high turbulence. Design curves have been proposed for growing, decaying and optimum sand ripples.

Sato (1986) and later **Sato and Horikawa (1988)** carried out a thorough analysis of sand ripple geometry by using existing data obtained in wave flumes, oscillatory tunnels and natural beaches along with new test results from an oscillatory water tunnel under both regular and irregular waves. The experimental conditions were such to investigate the effects of wave irregularity and asymmetry on the ripple characteristics. The relevant parameters for the study of ripple characteristics were the Shields parameter and the ratio of the orbital diameter to the median grain size. At high Reynolds numbers the lee vortices at the ripple crests did not maintain their coherent structure and tended to collapse into several three-dimensional vortices, thus enhancing the three-dimensionality of the patterns.

Three-dimensional ripples were observed for $2A/d_{50}>1500$ and $\theta>0.9(2A/d_{50})^{-1/4}$. Ripple wavelength, made non dimensional with the orbital diameter, was also analyzed in terms of $2A/d_{50}$ and θ. The scatter present in many researchers' field data could be due to the evaluation of the significant characteristics of the wave spectrum on the basis of the linear theory. In their analysis, instead, hydraulic parameters are directly estimated from the near-bottom velocity time series. Using also field data of natural beaches, they proposed expressions based on A/d_{50} and θ.

Ribberink and Al Salem (1990) carried out an experimental campaign in an oscillatory water tunnel aimed to study the bedform behavior at high velocity and large period regimes. Bedform characteristics were found to increase for increasing values of velocity amplitude and period; in particular, ripple wavelength was found to be proportional to the wave orbital amplitude close to the bottom. However, the transition to the sheet flow regime did not occur along with the transition to plane bed conditions; even for high mobility numbers indeed, bedform length did not show the expected decrease predicted by Nielsen (1981), and sheet flow took place only in a thin layer along the bedforms without flow separation over ripple crests. Nevertheless, when the oscillatory flow became asymmetric, a decrease in ripple length was observed according to Nielsen (1981), until a plane bed configuration occurred and a sheet flow condition took over.

Foti and Blondeaux (1995a) analyzed the case of ripples formed by a mixture of sands. The presence of graded sediments was found to have a stabilizing effect on ripple formation. Moreover a selective sediment transport, which tended to pile up the coarse grains at ripple crests and to keep the finer fraction in the troughs, was observed.

O'Donoghue and Clubb (2001) compared different existing predictive formulae for the geometry of sand ripples generated by oscillatory flow. They performed a set of experiments in an oscillatory water tunnel in order to test which one among the considered models would better predict ripple geometry in an environment close to typical field conditions. The models involved in the comparison are the previously mentioned Wiberg and Harris (1994), Nielsen (1981), Vongvisessomjai (1984) and Mogridge et al (1994) models. The results of this comparison showed that both the Wiberg and Harris (1994) and the Vongvisessomjai (1984) models are unreliable in predicting ripple geometry for field scale conditions. The Nielsen (1981) and the Mogridge et al (1994) models gave fairly good predictions on ripple lengths; however the fall-off in the ripple steepness at high mobility numbers in the Nielsen (1981) model was not observed in their measurements. Lengths and heights of symmetric ripples generated by sinusoidal flow are similar to those of asymmetric ripples generated by equivalent asymmetric flows. Three-dimensional ripples were observed in the presence of fine sand and with long period flows typical of field conditions. Their characteristics cannot be predicted by making use of the two-dimensional traditional methods.

Field data

Most of the field observations have been surveyed by diving researchers, on a limited temporal scale for the sampling. Alternatively photographic systems have been used in order to provide time series of ripple evolution and, occasionally, migration, failing when large suspension was present. More recently rotating sidescan sonar systems, mounted over tripods, have been adopted to map the bedform evolution. **Inman (1957) and Dingler and Inman (1976)**, among others, acquired bed profiles that showed a tendency to respond rapidly to flow conditions, flattening and re-rippling as a consequence of the more or less intense flow. **Nielsen (1984)** measured the concentration profiles over vortex ripples under non breaking waves. He observed that the profiles are straight lines in a semi-logarithmic plot, decaying exponentially far away from the bed. The vertical length scale of this decay is of the same order as ripple height magnitude. **Kos'yan (1988)** related ripple characteristics surveyed at three different sites to sediment grain size only. **Van Rijn (1993)** related the ripple height and ripple length to the orbital excursion A and the mobilty parameter $U_o^2/((s-1)gd_{50})$.

Jetté and Hanes (1996) compared collected bedform profiles with three predictive models due to Nielsen (1981), **Wiberg and Harris (1994)** and **Grant and Madsen (1982)**; all models seemed to overpredict ripple height and wavelength.
Traykovski et al. (1999) periodically mapped the rippled bottom, relating the bedforms characteristics to hydrodynamic conditions. Hysteresis effects were observed when hydrodynamic changes occurred. **Traykovski et al. (1999)** and **Hanes et al. (2001)** investigated also ripple migration. It was found that sediment transport was enhanced a lot due to migration.

Theoretical models

Analytical studies made use of perturbation techniques, providing a solution for wave orbital amplitude much smaller or much larger than ripple wavelength.
The roughness of each single grain of sediment is sufficient to trigger the process leading to ripple formation. Each sand particle indeed generates its own recirculating cells of fluid. When single grains start moving, they are trapped in the cells of the still grains in such a way that ripple formation will quickly take place. **Sleath (1976)** by studying the oscillatory flow over an initially slightly wavy wall, found out that a small disturbance on a flat bottom interacting with an oscillatory flow induces steady streamings, superimposed on the oscillatory flow. These steady streamings can be thought of as recirculating cells whose dimensions, intensity and direction depend on the characteristics of both the oscillatory motion and the bottom waviness. Steady streaming is generated when the non linear terms in the equation of motion are retained, if the simple harmonic flow over a slightly wavy wall is considered.
Ripples will not grow indefinitely: the tendency of the steady drifts to carry the grains and pile them up over the crests is limited by the gravity force acting in the downslope direction. As soon as a critical height will be reached, the gravity will balance the growing tendency. However, for any given set of conditions, a limiting steepness will exist. Rolling grain ripples can grow so steep that vortices start to develop in their lee; vortex ripples will then spread over the bed from the initial rolling grain ripple patterns.

Starting from **Sleath (1976)**, **Blondeaux (1990)** proposed a predictive theory to explain the mechanism of ripple onset on the basis of a linear stability analysis of a flat sandy bottom subject to viscous flow generated by wave action. The theory was developed for arbitrary values of the ratio between the wave orbital amplitude and the ripple wavelength. In particular, he determined the conditions for the decay or the amplification of a sinusoidal infinitesimal disturbance of the sandy bed as a function of four non dimensional parameters, namely the Reynolds number of the bottom boundary layer, the Froude number, the Reynolds number of the sediments and the relative density of the sediments.

This analysis, however, can only predict whether bedforms appear or not. Thus **Vittori and Blondeaux (1990)** applied a weakly non linear development of the previous theory to an unstable perturbation predicted by the linear stability analysis, characterized by values of the parameters that are close to the critical conditions, in order to find out whether a disturbance will grow reaching a finite equilibrium amplitude. Rolling grain ripples represent the equilibrium bedforms corresponding to this condition. For larger amplitudes, the flow separates; the theory can only predict the time development until the flow separates and it cannot follow the further growth of the bedforms in the vortex regime. Indeed the assumption of weakly non linear effects precludes to handle the strong nonlinear process associated with flow separation behind the crests.

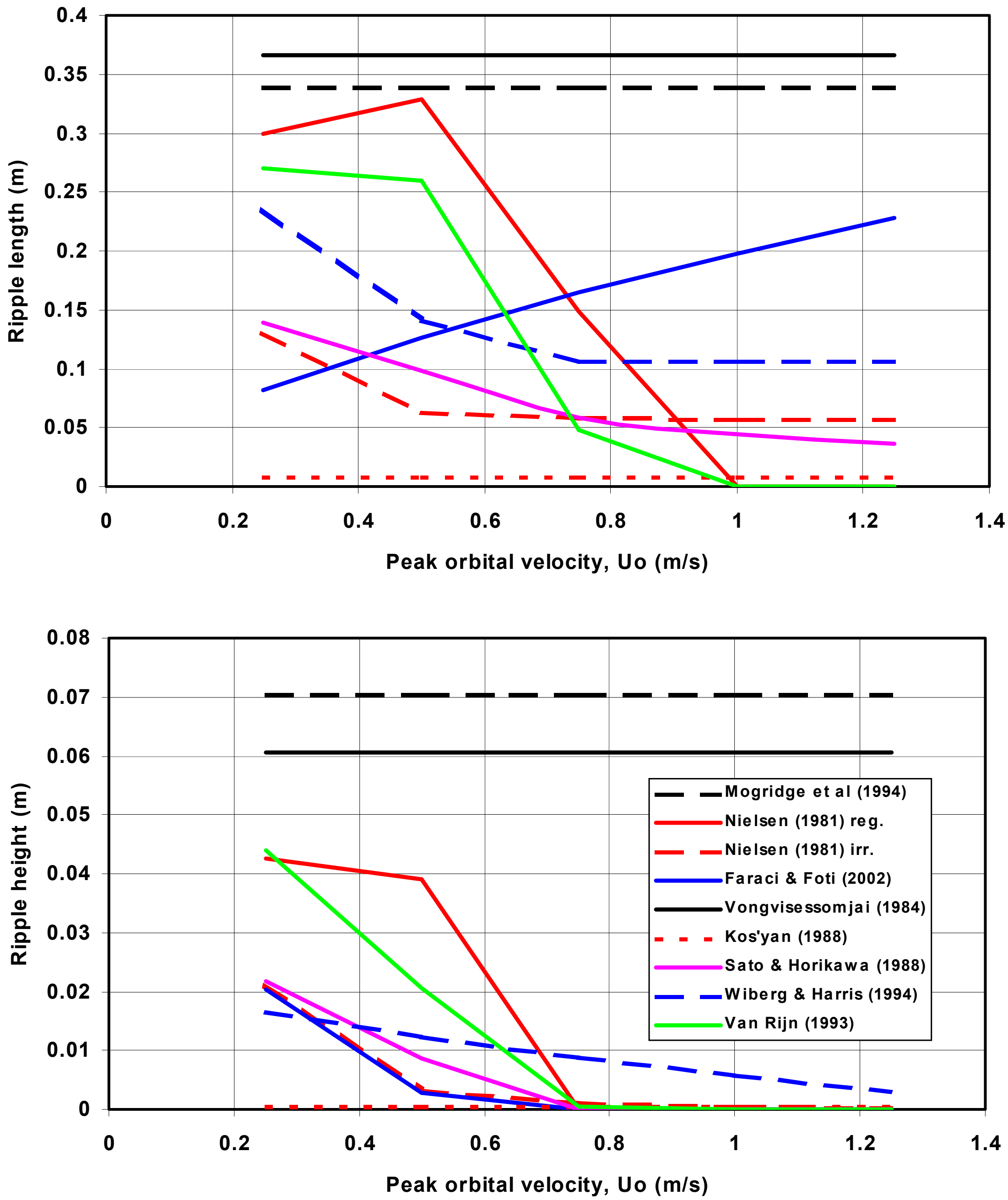
Foti and Blondeaux (1995a,b) extended the linear analysis of **Blondeaux (1990)** to a more relevant range of the non dimensional parameters, in particular accounting for higher Reynolds numbers, adopting a constant value for the eddy viscosity. They found a closed form solution for the turbulent oscillatory flow over a fixed wavy wall. They also gave a theoretical explanation of the sorting process observed experimentally, based on the formulation of a hiding function for graded sediments.

Blondeaux et al. (2000) by means of a linear stability analysis found that ripples migrate in the onshore direction as they appear with a speed which is a function of the sediment Reynolds number Re_d and the boundary layer Reynolds number Re_δ, as confirmed by experimental results by **Faraci and Foti (2002)**.

Andersen (2001) presented a model describing the formation of rolling grain ripples on a flat sandy bed, which treated each ripple as a particle interacting with the neighbouring ones. For each particle a heuristic equation of motion was formulated. From the collision and merging of particles, new larger ripples are formed, until an equilibrium state is achieved. These ridges shield the bed from the full force of the flow, creating a shadow zone, whose length influences the final equilibrium spacing of ripples.

Application of prediction models

The available prediction models have been used to compute the ripple height and ripple length for sediment with d_{50}= 0.2 mm and a range of wave conditions (wave period T= 6 s, peak orbital velocities U_o=0.25, 0.50, 0.75, 1.0 and 1.25 m/s). The results are shown in Figure 1.6.1. As can be observed, there is a wide range in computed ripple heights and lengths. The ripple heights vary between 0.07 and 0 m; the ripple lengths vary between 0.37 and 0.01 m. Most methods yield a ripple height decreasing with increasing peak orbital velocity, as the ripples will be washed out for large peak orbital velocities (> 1 m/s). Some methods yield a constant ripple height. One method yields an increasing ripple length with increasing peak orbital velocity.



Figures 1.6.1 *Computed ripple height and length; d_{50}=0.2 mm, T=6 s*

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1.5.2 Sand transport prediction models

Introduction

There is an essential difference between the sediment transport processes that occur above rippled and plane sand beds. It is important, therefore, to define the boundary delineating these regimes. Steep ripples are formed by relatively low waves in relatively deep water (i.e. typical offshore conditions). Such ripples tend to be long crested (two-dimensional), and to have steepness (η/λ) greater than about 0.12 (often > 0.15), where η and λ are the ripple height and length, respectively. The 'roughness' (k_s) of a rippled bed is equivalent to about 3-4 ripple heights ($k_s = 3\eta$- 4η). Beneath steeper waves in shallower water (for example, at edge of the surf zone), the ripples start to become washed out; their steepness decreases, causing k_s to decrease, and they may become shorter crested (transitional 2D-3D profiles). Finally, beneath very steep and breaking waves, the ripples are washed away completely, the bed becomes plane ('sheet flow' regime), and the bed roughness k_s decreases still further (scaling on θD where θ is the Shields parameter and D is the sand grain diameter). This sequence is summarised in Table 1 in terms of the non-dimensional 'relative roughness' A_0/k_s, where A_0 is the near-bed orbital excursion amplitude. As the waves become larger (increasing A_0), causing ripples to wash out (decreasing k_s), the relative roughness (A_0/k_s) increases.

Equivalent approximate ranges of the wave Reynolds number RE ($= A_0^2\omega/\nu$), where ω = wave angular frequency and ν = kinematic viscosity, are also shown in Table 1.6.1. The type of oscillatory boundary layer flow expected in different ranges of A_0/k_s and RE was reviewed by **Davies and Villaret (1997)**. In practice, much of our existing knowledge has been derived from laboratory experiments, and also modelling studies, that have been carried out either for the rippled bed regime ($A_0/k_s \sim 1$) or for the plane bed regime (large A_0/k_s). Rather little is known about the intermediate, 'very rough bed' regime (A_0/k_s = 5-30), which is unfortunate since many situations of practical importance in the sea occur in this regime.

Bed form characteristics	**2D Steep Ripples**	**2D and 3D Low Ripples**	**Washed-out Ripples**	**Plane Bed**
Ripple steepness η/λ	$\eta/\lambda \geq 0.15$	$0.05 < \eta/\lambda \leq 0.15$	$\eta/\lambda<0.05$	$\eta/\lambda=0$
Relative roughness A_0/k_s	O(1)	O(1-10)	O(10-100)	O(100-1000)
Reynolds number RE	$O(10^3\text{-}10^4)$	$O(10^4\text{-}10^5)$	$O(10^5)$	$O(10^6\text{-}10^7)$

Table 1.6.1 *Bed form characteristics in the coastal zone*

Fundamentally different physical processes determine sand transport rates above plane and rippled sand beds in oscillatory flow. Above plane beds, momentum transfer occurs primarily by turbulent diffusion. In contrast, in the bottom part of the boundary layer above rippled beds, momentum transfer and the associated sediment dynamics are dominated by coherent periodic vortex structures; above this, in a layer of thickness of about 1-2 ripple heights the coherent motions break down and are replaced by random turbulence. The effect of this is that sand is entrained into suspension to considerably greater heights above rippled beds than above plane beds. In addition, the phase of sand pick-up from the bottom during the wave-cycle is also significantly different above rippled beds, with pick-up being linked to the phase of vortex shedding. This has potentially important consequences for the net sand transport rate beneath asymmetrical waves, which can be in the negative ('offshore') direction despite the larger positive ('onshore') orbital velocities.

A range of models has been developed to simulate sediment transport in the different regimes. The most fundamental difference in the model types is between 'research' and 'practical' formulations (see **Davies et al., 2002**). Essentially, 'research' models aim to represent the detailed physical processes involved in sand transport by waves and currents, including intra-wave transport processes. They are developed for a particular transport regime (e.g. sheet flow above a plane bed), and they normally resolve the vertical structure (and also, in some cases, the horizontal structure) of the velocity and sediment concentration fields. Since research models normally require lengthy computation times, they tend not to be implemented in coastal morphological models involving many grid points. Instead, more 'practical' models are used for this purpose; these are simpler prediction schemes that either do not resolve the spatial and temporal structure of the velocity and concentration fields or, if they do so, employ simplified and prescriptive approaches for this purpose. By their nature, practical models are usually aimed at covering a wide range of conditions; they are robust and easy to compute, and they can therefore be implemented relatively easily in coastal morphological models.

An important distinction between research and practical models is that, while research models determine both the 'wave-related' and 'current-related' components of the suspended load transport, practical models usually determine only the 'current-related' component. The current-related transport, which is often the dominant component, is in the direction of the mean flow, subject to 'veering' effects in the oscillatory boundary layer (**Davies and Villaret, 2002**). The wave-related component, which arises from intra-wave processes, can alter the net transport rate substantially in both magnitude and direction, however, particularly for coarser sand grains confined to the oscillatory boundary layer.

The respective components of the total sand transport rate are defined as follows. If the instantaneous velocity component (*U*), with which the sediment grains are assumed to be transported horizontally, and concentration (*C*), are written, respectively:

$$U = u + u_p + u' \qquad \text{and} \qquad C = c + c_p + c'$$

where both U and C are defined at height z above the bed. Here $u = \langle U \rangle$ and $c = \langle C \rangle$ where angle-brackets denote averaging over a large (integral) number of wave periods, subscript p denotes the periodic component and a dash denotes the turbulent component. Although U is written here as the velocity component in a given direction, it may alternatively be regarded here as a 2D vector quantity defining the horizontal velocity field. The cycle-averaged flux at level z is given approximately by:

$$\langle UC \rangle \approx uc + \langle u_p c_p \rangle$$

where the small turbulent contribution $\langle u'c' \rangle$ has been neglected. The net suspended flux, averaged over the depth from the reference level $z = a$ to the mean surface level $z = h$ is then given by :

$$q_s = \int_a^h \langle uc \rangle \, dz \approx \int_a^h [uc + \langle u_p c_p \rangle] \, dz$$

in which the term uc corresponds to the 'current-related' contribution to the net transport, and the term $\langle u_p c_p \rangle$ to the 'wave-related' contribution. [In a field context, it is sometimes relevant to further subdivide the wave-related transport into high frequency and low frequency components (**Van Rijn, 1993**).] Finally the total net transport rate (q_t) is given by:

$$q_t = q_s + q_b$$

where q_b corresponds to the net bed load transport in the height range $0 < z < a$. Here level $z = 0$ usually represents the notional, undisturbed, (mean) bed level. However, if the sheet flow layer is taken into account explicitly in a model, the integration range may then involve levels below $z = 0$ (e.g. **Malarkey et al., 2003**).

Flat bed phenomena (upper regime) and modelling

Research models

Sand transport in combined wave and current flows over a flat bed involves fluid velocities and sand concentrations that vary strongly with time during the wave cycle. Here ***unsteady 'intra-wave' models***, based on analytical or numerical solution of the basic fluid momentum and continuity equations, in combination with the sediment mass balance (advection-diffusion) equation, have been developed to determine the velocity and sand concentration fields as functions of space and time. These models include the potentially important 'wave-related' component of the transport.

Instantaneous sand concentrations ***in suspension*** above a flat bed are usually obtained from the solution of the one-dimensional vertical (1DV) advection-diffusion equation, namely:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial z}\left(\varepsilon_{s,z} \frac{\partial C}{\partial z} + w_s C \right)$$

where C = instantaneous, volumetric sand concentration; w_s = sediment settling velocity; $\varepsilon_{s,z}$ = sediment mixing coefficient (diffusivity) in the vertical z-direction; and t = time. The fluid motion in and above the sheet flow layer is described by the usual momentum balance equation.

Analytical or numerical solution of these equations requires expressions for the fluid and sediment diffusivity and boundary conditions for velocity and sand concentration at the bed ($z = 0$). The simplest approach for the definition of the diffusivity coefficients is the traditional mixing length concept. A simple approach for the bed boundary condition for sediment concentration is then the application of an empirical or semi-empirical expression for concentration as a function of the bed-shear stress (e.g. **Englelund and Fredsøe, 1976; Zyserman and Fredsøe, 1994**). Analytical unsteady models have been presented by **Madsen and co-workers** (**Madsen and Grant, 1976; Grant and Madsen, 1986; Glenn and Grant, 1987**) and, more recently, by **Bosboom et al. (1998)**. Overviews of numerical models, mainly for the suspension layer, have been given by

Davies et al. (1997), Dohmen-Janssen (1999) and Davies et al. (2002). The available numerical models, for the parameter ranges of practical importance, vary in complexity from mixing length models (**Ribberink and Al-Salem, 1995; Dohmen-Janssen et al., 2001**) and eddy viscosity models (**Fredsøe et al., 1985**), through one- and two-equation, Reynolds-averaged, turbulence-closure models (**Davies and Li, 1997; Brors, 1999; Utnes and Meling, 1998; Guizien et al., 2001**), to more elaborate 2-phase models (see below).
The various model types include a range of detailed physical processes and/or parameterisations of these processes; for example, **Li and Davies (2001)** developed a 1DV, one-equation turbulent kinetic energy closure model that incorporates the effects of (i) graded sediment sizes, (ii) turbulence damping by sediment in suspension and (iii) hindered settling. This model was used to study transport beneath large symmetrical and asymmetrical waves. **Guizien et al. (2003)** have used a 1DV two-equation closure model to investigate sediment entrainment events at reversal in oscillatory flow; this phenomenon can potentially affect the wave-related component of the transport (**Murray et al., 1991**). **Davies and Villaret (2002)** used a one-equation closure model to investigate the process of mean current 'veering' in cases in which waves are obliquely incident on a current. They presented a case study which demonstrated that the directions of the mean flow, the bed load transport, the current-related suspended load transport, and the true suspended load transport, could all be significantly different from the direction of the mean pressure gradient driving the current. They argued that veering effects have clear implications for grain size sorting; for example, if the mean current is in the longshore direction off a beach, outside the breaker zone say, coarser grains will tend to migrate onshore, while finer grains will tend to migrate offshore. These trends are consistent with the common observation that beach material includes little or no fine sand.

The ***'sheet-flow'*** layer is the thin layer of high sediment concentration that occurs above plane, non-cohesive, sediment beds in intense wave and current flow conditions. Sheet flow contributes significantly to the net sediment transport beneath large waves. The importance of grain collisions in the sheet-flow layer requires modelling approaches that are different from the standard diffusion concepts that are applied in the outer, low-concentration, suspension layer. **Bagnold (1956)** developed the concept of a 'dispersive stress' in the sheet-flow layer, which absorbs the difference between the fluid stress applied at the top of the layer and the critical threshold stress for sediment movement, which may be inferred to exist at the bottom of the layer. **Sayed and Savage (1983)** produced an analytical description of the velocity and concentration profiles in a steady sheet-flow layer by splitting the stress into viscous and plastic components. **Wilson (1989)** and, more recently, **Sumer et al. (1996)** related empirically the properties of a steady sheet-flow layer to the applied bed shear stress. In particular, they related the thickness of the sheet-flow layer (δ), and its the effective roughness (k_s), to the Shields parameter (θ).
A complete modelling description of the sheet-flow layer requires, in principle, the use a two-phase model in which the full collisional nature of the sediment transport process is taken into account (e.g. **Asano, 1990; Li and Sawamoto, 1995; Dong and Zhang**). The earlier two-phase model studies of **Kobayashi and Seo (1985)** and **Asano (1990)**, among others, were based on Bagnold's classical concepts. Later, **Greimann et al. (1999)** used a more general granular stress model (limited to dilute suspensions), while **Jenkins and Hanes (1998)** included collisional effects to study particle-particle interactions within the sheet flow layer. **Villaret et al. (2000)** presented 1DV computations for steady flow using a two-phase model based on the general formulation proposed by **He and Simonin (1994)** for industrial applications. This model, which includes the effect of particle-particle interactions and particle-turbulent flow interactions, allows a complete description of the flow both within the near-bed, high concentration region, and also in the dilute suspension region. **Villaret and Davies (1995)** initially used this model to reproduce 'starved bed' experimental data sets; **Villaret et al. (2000)** later extended the model to account for interactions with an erodible rough bed and applied it to predict saturation conditions above an equilibrium bed in steady flow. Despite the conceptual advantages of two-phase models, however, their complexity makes them unsuitable for practical sediment transport prediction. In fact, most 'research' models do not attempt to represent the sheet-flow layer in any detail, instead relying upon some form of parameterisation of the 'bed-load' transport (**Davies et al., 1997**). In contrast, the suspended load is often represented in considerable detail, for example by use of numerical turbulence-closure schemes coupled to the turbulent diffusion of sediment.
While many studies of the sheet flow layer have been conducted for steady flow, experiments on oscillatory sheet-flow have been carried out in wave tunnels by **Horikawa et al. (1982), Ribberink and Al-Salem (1995), Dohmen-Janssen et al. (2001), McLean et al. (2001), Ahmed and Sato (2001)**, and others. By analogy with steady flow, **Dohmen-Janssen et al. (2001)** used the peak stress in the wave cycle to characterise the sheet-flow layer thickness in oscillatory flow. Unsteady sheet flow modelling has been undertaken by **Bakker and Van Kesteren (1986)** who developed an analytical approach based on an extension of **Bagnold's (1956)**

concepts to explain the behaviour of the time-varying concentration profile. They assumed that the mean dispersive force is balanced by the weight of the supported grains, while the second harmonic of the dispersive force is balanced by the vertical drag on the grains.

Kaczmarek (1991) adapted the steady sheet-flow description of **Sayed and Savage (1983)** for use in unsteady flow, utilising the instantaneous stress at the top of the layer, and connecting this to a diffusive model of the suspension layer. More recently, **Kaczmarek and Ostrowski (2002)** included a third 'contact layer' between the sheet-flow layer and suspension layers, where both collisions between sediment grains and turbulent diffusion of sediment are important. A rather simpler, quasi-steady, 1DV model of oscillatory sheet flow, similar to that of **Kaczmarek (1991)**, has been proposed by **Malarkey et al. (2003)**. Despite its simplicity this model achieves greater realism than is found in classical models based on the reference-concentration approach, by 'tracking' the erosion and deposition of the bottom sediment layer in relation to the amount of sediment present in the sheet-flow and suspension layers. The prescriptions used to represent the sheet-flow layer are based on **Wilson (1989)** and **Sumer et al.'s (1996)** steady-flow concepts. This new model yields continuous, instantaneous, vertical profiles of velocity and sediment concentration from the stationary bed, through the high-concentration sheet-flow layer, up into the outer suspension layer. Conditions in the suspension layer are modelled using a standard k-ε turbulence-closure scheme, together with the sediment continuity equation, subject to matching conditions at the interface between the sheet-flow and suspension layers.

The model of **Kaczmarek (1991)**, as well as many of the classical reference-concentration models described by **Davies et al. (1997)**, are 'quasi-steady' in that the characteristics of the sheet-flow layer are defined strictly in terms of the *instantaneous* bed shear stress. For oscillatory flow, **Sleath (1994)** quantified the importance of local acceleration in the flow by means of the parameter, S, given by:

$$S = \frac{\omega U_0}{g(s-1)}$$

where U_0 is the near-bed velocity amplitude, ω is the wave angular frequency (= $2\pi/T$, where T is the wave period), s is relative density of the sediment (= ρ_s/ρ where ρ_s is the sediment density and ρ is the fluid density) and g is the acceleration due to gravity. The parameter S is a measure of the importance of the horizontal pressure gradient in the vicinity of the bed compared with the stress due to gravity. **Sleath (1994)** and **Zara Flores and Sleath (1998)** demonstrated empirically that for $S < 0.2$ local acceleration could be neglected and that, therefore, a quasi-steady approach should thus be valid in the sheet flow layer. In practice, for sands having $s = 2.65$, this condition is almost always satisfied. However **Dohmen-Janssen et al. (2002)** found in some experiments that, even for such modest values of S, phase effects appeared to be important in the sheet-flow layer. They developed a more stringent condition, specifically for sand, based on a 'phase-lag' parameter, P, given by:

$$P = \frac{\omega \delta}{w_s}$$

where δ is the sheet-flow layer thickness (order of 0.01 m), which is proportional to the peak shear stress in the wave cycle, and w_s is the settling velocity of the sand.

Practical models

The more 'practical' transport models for oscillatory flow above flat beds may be subdivided, following **Dohmen-Janssen (1999)**, into:

quasi-steady models based on the assumption that the instantaneous sand transport rate within the wave cycle is proportional to some power of the instantaneous near-bed orbital velocity or bed-shear stress (with phase lag effects neglected). Such models have been proposed by various researchers for bed load transport (**Madsen and Grant, 1976; van Rijn, 1993; and Ribberink, 1998**). For combined bed load and suspended load transport, the quasi-steady Bagnold-Bailard model is commonly used (**Bagnold, 1966; Bailard, 1981; Bailard and Inman, 1981**).

semi-unsteady models based on the quasi-steady approach, but with a correction factor to represent phase lag effects between the velocity and concentration fields. Such models have been proposed by **Dibajnia and Watanabe (1992) and Dohmen-Janssen (1999)**. **Dohmen-Janssen (1999)** showed that net transport rates due to combined oscillatory flows and a net current are reduced considerably by the influence of phase-lags between sediment concentration and flow velocity. **Dibajnia and Watanabe (1992)** proposed an expression for the total (bed load and suspended load) net transport rate, which is the difference between the amount of sand transported in the forward (onshore) and backward (offshore) directions. The amount of sand transported in the forward direction consists of two parts: sand entrained and transported during the forward half cycle; and sand entrained during the preceding backward half cycle, but also transported forward having not yet settled back to the bed. Similar processes occur during the backward half wave cycle. The net transport may thus be in the offshore direction (against the wave direction) for fine bed material (< 0.2 mm) and small wave periods (< 5 s). According to **Dohmen-Janssen (1999)**, phase-lag effects represented by the 'phase-lag' parameter P (defined above) are important for fine sediment, large peak orbital velocities and small wave periods (P > 0.5).

In relation to sand transport prediction, for example within coastal-area morphological models, a range of practical models, including those above, may be implemented. These approaches include the model of **Bijker's (1992)**, the model TRANSPOR2000 of **van Rijn (1993, 2000)**, and the model SEDFLUX (**Damgaard et al., 1996, 2001; Soulsby, 1997**). Each of these models represents both the bed load and suspended load but, with the exception of TRANSPOR 2000, does not include the wave-related component of the transport. The relative performance of these and other models is summarised later. The continuing appeal of **Bijker (1971, 1992)** model (e.g. the implementation by **Davies and Villaret (2002)** including a prediction scheme for the bed roughness k_s) lies in its simplicity and ready implementation. Moreover, its predictions are broadly similar to those of more recent practical models (see **Van Rijn et al., 2001**). It also has the appeal of being based on classical sediment transport concepts for the bed load and suspended load, rather than being based purely upon empirical curve fitting to transport data.

Rippled bed phenomena (lower regime)

Introduction

Reviews concerning sand concentrations and sand transport in oscillatory flow above rippled beds have been provided by **Fredsøe and Deigaard (1992), Nielsen (1992), van Rijn (1993)** and, more recently, **Van Rijn (2000)**. The oscillatory flow above steep ripples is characterised by the generation, advection and diffusion of near-bed vortices, which dictate the behaviour of the sediment grains suspended in the near-bed layer. Since detailed, intra-wave, field data on sand transport in the rippled bed regime is scarce, much of our knowledge is based on laboratory studies. The basic characteristics of instantaneous, local sand concentration over ripple profiles were demonstrated by in the laboratory by **Bosman (1982), Block et al. (1994) and Villard and Osborne (2002)**, and in the field by **Vincent et al. (1999)**, among others. However, it is only comparatively recently that measuring techniques have become available to properly quantify the process of vortex formation and shedding (e.g. Particle Image Velocimetry (PIV) used by **Earnshaw (1996)** and **Ahmed and Sato (2001)**). **Earnshaw (1996), Marin (1988) and Ranasoma (1992)** produced vorticity contours over fixed ripples, at various instants in the wave cycle, while **Lofquist (1980)** and **Rankin and Hires (2000)** have produced time series of the force on movable ripples. A central issue in the modelling of boundary layer flows above ripples is the specification of the bed roughness (k_s) (see, for example, **Nielsen, 1992; Van Rijn, 1993**). Laboratory (wave flume) measurements of k_s in wave and current flows suggest that $k_s \approx 3\eta\text{-}4\eta$, where η is the ripple height. In contrast, measurements made beneath waves and also wave-current flows in larger-scale wave basins tend to yield significantly lower values ($k_s \approx 0.5\eta\text{-}\eta$) (**Van Rijn, 1993**). These lower values appear to be associated with irregularities in the ripples formed on larger laboratory (basin) scales, e.g. along-crest irregularities in the ripple profile that promote 2D-3D flow structures, in contrast to the predominantly 2D flow above steep longer-crested ripples. Use of the lower range of k_s values is probably to be preferred in field applications. **Mathisen and Madsen (1996a, 1996b; 1999)** carried out detailed studies of the bed roughness in the laboratory using a fixed 2D bed roughness. **Styles and Glenn (2002)** proposed models for the prediction of both the ripple dimensions and the bed roughness, which they validated using field data; their approach is based on an extended version of the **Grant and Madsen (1979)** wave-current boundary layer model proposed by **Wikramanayake and Madsen (1991)**, and later extended by **Styles and Glenn (2002)**.

In relation to sediment transport, intra-wave measurements of sediment entrainment and suspension, made over a full ripple wavelength were carried out at small laboratory scale by **Sato and Horikawa (1986)** and **Sato et al. (1987)**; their results were obtained by moving a light-absorption probe over a two-dimensional horizontal-vertical grid of 100 points. More usually, on small laboratory scales, only a limited number of point measurements have been obtained, typically made above the ripple crest and trough using optical probes (**Nakato et al., 1977; Block et al., 1994**). More recently, **Sleath and Wallbridge (2002)** have used a digital video camera to study pick-up from rippled beds in oscillatory flow. Their main finding, which has significant implications for the modelling of sediment pick-up in 2DHV formulations, is that there are two maxima in the sediment transport rate over the ripple crest during the course of a wave half cycle. The first corresponds to sediment entrained at flow reversal, while the second corresponds to the jet of sediment passing over the ripple crest at around the time of peak velocity in the free-stream. Interestingly, this second maximum was found to be well predicted by the steady-flow transport formula of **Meyer-Peter and Muller**, provided that the calculation was based on an estimate of shear stress in the vicinity of the ripple crest. These measurements above ripples confirmed the earlier findings of **Zara Flores and Sleath (1998)** that the flow in the mobile layer on a ripple surface is quasi-steady at low values of the acceleration parameter *S* (see earlier), except in the vicinity of flow reversal.

In larger scale experiments, acoustic probes have been used to measure the instantaneous concentration and velocity components beneath regular and irregular waves above rippled beds (**Chung and Van Rijn, 2003**), and high-resolution acoustic backscatter systems (ABS) have been used beneath wave groups to study time-variations in concentration above ripples on longer group-scales (**Vincent and Hanes, 2002**). **Thorne et al. (2002a; 2002b; 2003a; 2003b)** have reported the results of ABS measurements made in the large-scale Deltaflume of Delft Hydraulics. These measurements were made non-intrusively above steeply rippled sand beds beneath weakly asymmetric surface waves. The experiments reported by **Thorne et al. (2003b)** comprise detailed intra-wave measurements made above different locations on a (migrating) ripple profile, which reveal the process of sediment trapping by growing vortices, and the shedding of these sediment-laden vortices at flow reversal. These intra-wave results have also been horizontally- (ripple-) averaged in order to provide a basis for comparison with the results of a new 1DV modelling approach for rippled beds (**Davies and Thorne, 2002**)

Modelling approaches

Unsteady modelling of the suspended sediment transport above a sand bed covered with (long-crested) ripples requires the simultaneous (numerical) solution of the time-dependent momentum equation for the oscillatory fluid flow and the time-dependent advection-diffusion equation for suspended sediment in the two-dimensional horizontal-vertical 2DHV plane. The advection-diffusion equation reads as follows:

$$\frac{\partial C}{\partial t}+\frac{\partial}{\partial x}\left(UC-\varepsilon_{s,x}\frac{\partial C}{\partial x}\right)+\frac{\partial}{\partial z}\left((W-w_s)C-\varepsilon_{s,z}\frac{\partial C}{\partial z}\right)=0$$

where *C* is the instantaneous sand concentration; *U* and *W* are the horizontal and vertical instantaneous fluid velocity components; w_s is the settling velocity; $\varepsilon_{s,x}$ and $\varepsilon_{s,z}$ are the sediment mixing coefficient in the horizontal *x*- and vertical *z*-directions; and *t* is the time.

The numerical simulation of detailed vortex motions requires the application of sophisticated (e.g. turbulence-closure or discrete vortex) models on a fine spatial grid. Furthermore, the shape and dimensions of the ripples should be known a priori in order to provide appropriate boundary conditions. The instantaneous fluid flow and suspended transport in a combined steady and oscillatory flow over a rippled bed can then be solved in an integrated way, which is the great benefit of this approach. However, a major drawback is the relatively large computational time involved; this makes existing 2D (or 3D) rippled bed models unsuitable for application in numerical morphological modelling systems. Moreover, the detailed unsteady modelling of vortex formation and shedding above ripples remains at an early stage of research and, so far, few attempts have been made to use such models to simulate sediment in suspension.

The need for a complex 2D (or 3D) model to represent the oscillatory flow above ripples depends, in the first place, upon the ripple steepness. If the steepness (ripple height to length ratio) is greater than about 0.1, the process of eddy shedding is likely to be fully developed. For lower ripple steepnesses, including many 'rippled bed' situations of practical importance in the field, it is probably quite reasonable to use 1DV 'flat bed' models, provided that the formulation allows the model to be run with an appropriately enhanced bed roughness (k_s) (**Davies and Villaret, 2002**). It is important to note that the residual flows above ripples, which determine the current-related component of the transport, are fundamentally different from those above plane beds. **Huang and Dong (2002)** presented 2DHV residual flow computations above ripples beneath surface progressive waves and also solitary waves. **Davies and Villaret (1999)** developed a 1DV analytical model to predict the Eulerian drift induced by weakly-asymmetric progressive waves above rippled and very rough beds in the turbulent regime. The main result of their study was that the Eulerian drift is characterised by a pronounced near-bed jet in the direction of wave advance, beneath a layer extending to the edge of the boundary layer in which the drift is in the opposite direction.

2DHV models

For beds that are steeply rippled, several 2DHV modelling studies have sought to represent the formation and shedding of vortices, and the subsequent trajectories of the (decaying) vortices. Some of these studies have also investigated integrated quantities such as the wave friction factor (drag coefficient), and the time-mean residual flow structures above the ripples. For example, numerical solutions of the governing vorticity equation were developed by **Sleath (1973, 1982)** and **Blondeaux and Vittori (1991)**. **Longuet-Higgins (1981)**, and subsequently **Block et al. (1994)**, proposed essentially analytical, inviscid (i.e. without diffusion of vorticity) discrete vortex models of the oscillating flow above steep ripples, while **Hansen et al. (1994), Perrier (1996) and Malarkey and Davies (2002)** developed numerical cloud-in-cell (CIC) discrete vortex models (including the diffusion of vorticity). Turbulence-closure models have also been used (see the intercomparison of **Lewis et al. (1995)**; for example, **Perrier (1996)** developed a Reynolds stress closure model, and **Andersen and Fredsøe (1999)** used a Reynolds-averaged k-ω model, to investigate the hydrodynamic and sediment transport processes above ripples. **Andersen et al. (2001)** presented instantaneous distributions of the Shields parameter over a full ripple wavelength based on this k-ω model. They noted that the instantaneous stresses over a ripple surface are typically several times larger than on a flat bed. Trouw et al. (2000) used a k-ε model (within the CFD package PHOENICS) to investigate wave-induced suspension above ripples for regular waves, asymmetrical waves and wave groups. Their model used **Nielsen's (1992)** sediment pick-up function, applied locally over the rippled bed surface, and solved the 2DHV convective-diffusion equation for the suspended sediment. The effects of graded sediment size were also discussed. Intra-wave aspects of **Trouw et al.'s (2000)** solution were presented as a series of animations.

Andersen (1999) further demonstrated that, while the ripple actually changes shape during the wave-cycle, the approximation of a fixed ripple shape in a model is reasonable unless ripple stability is being considered. The k-ω model has been used more recently by **Andersen and Faraci (2003)** to study wave-current interaction above ripples in general angular cases. They found that veering of the near-bed current was more pronounced than over flat beds due to the bed topography turning the mean flow parallel to the ripple crests. The near-bed current-related sand transport tends, therefore, to be in the direction of the ripple crest lines; this would appear to be in qualitative agreement with the veering effect illustrated by **Davies and Villaret (2002)** for flat rough beds (see earlier), though the veering would appear to be more pronounced above ripples due to the *combined* effect of wave-current interaction and topographic steering. [Laboratory experiments on the effects of waves and currents combined at a general angle of incidence above rippled beds have been reported by **Barrantes and Madsen (2000)**. They found that, close to the bottom, there is a transverse component of the mean velocity which is of the same order as the main flow.] At a more fundamental level, Large Eddy Simulation (LES) has started to be used. The potential of this approach has been demonstrated, for steady flow above ripples, by **Zedler and Street (2001)**. They found instantaneous correlations between regions of the near-bed flow with a large vertical velocity component and high sediment concentration; their 3D-results provide evidence of the importance of coherent flow structures for sediment entrainment, and suggest a mechanism by which the sediment diffusivity is greater than the eddy viscosity above rippled beds (c.f. **Nielsen (1992)** for oscillatory flows).

Three dimensional effects over ripples, and the transition from two-dimensional long-crested ripples to brick-pattern ripples, have been investigated by several workers. **Sleath (1984)** suggested that, while the shape of vortex ripples depends upon the scale and intensity of the primary vortices formed in the lee of the crest, the formation of brick-pattern ripples may be caused by the breakdown of the two-dimensional vortex structure, and the formation of bridges across the ripple crest by horseshoe vortices. **Hara and Mei (1990)** showed how centrifugal instability can result in a vertical flow structure that possibly accumulated sediment particles to form the bridges. **Vittori and Blondeaux (1992)** investigated the formation of brick-pattern ripples in oscillatory flow on the basis of weakly nonlinear stability analysis. Subsequently, **Scandura et al. (2000)** computed a three dimensional boundary layer flow over steep ripples, at higher Reynolds number, and discussed a spanwise instability mode occurring in oscillatory flow.

Despite the development of these models, our knowledge remains fragmentary, not least because the models have been applied in rather limited parameter ranges, including cases for which detailed experimental data exists. Nevertheless, several models have succeeded in simulating the vortex shedding process in oscillatory flow convincingly. **Perrier (1996)** compared the results of a discrete-vortex model (essentially the model of **Hansen et al. (1994)**) and a Reynolds stress closure model, and found that both models simulated the formation and shedding of vortices realistically. **Perrier** also found that the results from a k-ε closure model were less convincing; high turbulence intensity seemed to be confined to the ripple crest region, and vortex shedding did not become well established. Greater success was achieved by **Andersen and Fredsøe (1999)** using a k-ω closure model, i.e. a model of essentially the same complexity as the k-ε model in the hierarchy of turbulence schemes.

Using a simpler, analytical, discrete-vortex model, **Block et al. (1994)** showed that the phase angle of eddy shedding depends upon (d_0/λ), the ratio between the near-bed orbital diameter (d_0) and the ripple wavelength (λ). [For steep ripples, $d_0/\lambda \approx A_0/k_s$.] In simulations of two flume experiments involving regular waves above steeply rippled beds of very fine sand, **Block et al. (1994)** found that the lee-slope eddy was shed (and moved over the ripple crest) *after* flow reversal for $d_0/\lambda = 1.6$, and *before* flow reversal for $d_0/\lambda = 2.6$, in accordance with experimental observations. [**Villard and Osborne (2002)** found in the laboratory that, beneath irregular waves, 73% of the observed events involving shedding and advection of sediment-laden vortices occurred before flow reversal, while 27% occurred after flow reversal.] For larger values of d_0/λ, and correspondingly smaller ripple steepnesses (i.e. with the ripples starting to become washed out), this coherent eddy shedding process starts to break down so that, ultimately, discrete-vortex models cannot be used (for, say, $A_0/k_s \geq 5$). Since the wave-related component of the net sediment transport depends upon the presence or absence of eddy shedding effects, it is important to establish the boundary between the 'steep-ripple' regime in which vortex shedding dominates in respect of both momentum and sediment transfer, and the 'low-ripple' ('very rough') regime in which coherent eddy shedding starts to become replaced by more random turbulent processes. Two dimensional research models can be used to improve our understanding of this transition.

As far as sediment in suspension above ripples is concerned, Lagrangian particle tracking has been used in several previous oscillatory flow models (e.g. **Hansen et al., 1994; Block et al., 1994; Perrier, 1996**). **Longuet-Higgins (1981)** analytical discrete-vortex model showed how a vortex pair might be formed above each ripple in each wave half-cycle, and the predicted upward movement of this pair (prior to decay) suggested a potentially important mechanism for lifting sediment into suspension. However, systematic vortex pairing has not been found to occur in **Block et al.'s (1994)** analytical model, or in any of the more recent numerical models, though **Trouw et al. (2000)** have speculated that the mechanism might be important. In practice, **Block et al.'s** model does not allow sediment to be entrained to sufficient heights above the bed, which can probably be attributed to the lack of diffusion in their formulation. However, in his numerical discrete-vortex model, **Perrier (1996)** had rather greater success in this respect. He allowed sediment to be entrained both directly from the bed through application, locally along the ripple surface, of **Fredsøe's (1984)** oscillatory boundary layer model, and also by means of the jet of sediment leaving the bed in the region of the ripple crest. He then followed the fate of these two suspended sediment populations during the wave cycle. The results showed reasonable agreement with experimental data in respect of suspended concentrations and also transport rates.

In the models referred to above, the mechanism of sediment 'trapping' in the strong vortex above the ripple lee-slope, followed by the shedding of this vortex across the crest, is all important. In the case of asymmetrical waves above symmetrical or asymmetrical ripples (having their steep face directed shorewards), the wave-related suspended transport is in the onshore direction in a very thin near-bed layer, but in the offshore direction above this as a result of the (vortex-) trapping of a greater amount of sediment beneath the wave crest, and the 'pumping' of this sediment offshore beneath the trough (see, for example, **Trouw et al. (2000)**). It is interesting to note that **Malarkey and Davies' (2002)** discrete-vortex model suggests that the vortex that forms above the gently-sloped side of an asymmetrical ripple may actually be stronger than the vortex that forms above the steeply-sloped side. However, the former vortex is more dispersed, and its sediment transporting capability is therefore likely to be relatively weak, consistent with the general physical picture outlined above.

1DV models

Although 2D-models have achieved reasonable success in representing the main features of vortex dynamics and the associated sediment transport above rippled beds, these models are unduly complex and computationally demanding from an engineering point of view. For the most part, existing, more practical, 1DV models have attempted to represent ripples by enhancing the bed roughness (k_s) used in standard, one-dimensional (vertical) 'flat bed' formulations; after the governing equations have been horizontally-averaged, the effects of the ripples on the flow and on sand concentrations are simply represented by an effective bed roughness and an effective sediment diffusivity. **Nielsen (1992), Chung et al. (2000) and Chung and van Rijn (2003)** have shown that this unsteady, ripple-averaged approach yields good wave period-averaged sand concentrations in the ripple regime. However, the 1DV approach does have potentially severe limitations, at least if conventional (time-invariant) formulations are used for the sediment diffusivity and eddy viscosity. For example, **Chung et al. (2000) and Chung and Van Rijn (2003)** found that the wave-related (oscillating) suspended transport cannot be simulated accurately using a conventional diffusivity formulation. Appropriate time-mean formulations for the eddy viscosity and sediment diffusivity above rippled beds, for use in a horizontally-averaged 1DV framework, have been proposed by **Nielsen (1992) and Sleath (1991)**. They presented empirical evidence to show that, in contrast to the plane bed case, a *height-independent* viscosity is appropriate in the near-bed vortex layer above ripples. [**Sleath's** eddy viscosity formulation has been compared with field observations of the rate of turbulence decay with height above the bed by **Smyth et al. (2002)**.] **Van Rijn (1993)** also found that use of a height-independent sediment diffusivity in the lower layer of a two-layer formulation, in which the diffusivity then increased linearly with height, was essential to produce accurate mean concentration profiles above ripples. Subsequently, it was shown by **Davies and Villaret (1997, 1999)** that the time-variation in eddy viscosity is more pronounced above ripples than above plane beds, with peaks in viscosity occurring near times of flow reversal. Their conclusions were based on an analysis both of experimental data (**Ranasoma, 1992**) and also the results of **Perrier's (1996)** discrete-vortex model. This new 1DV eddy viscosity approach has been extended by **Davies and Thorne (2002)** to study sediment transport by waves and currents above ripples. Here the time-varying sediment diffusivity ($\varepsilon_{s,z}$) has been related to the eddy viscosity ($\varepsilon_{m,z}$) by a height-varying factor $\beta = \varepsilon_{s,z}/\varepsilon_{m,z}$ with $\beta \approx 4$ in a near-bed vortex-dominated layer (c.f. **Nielsen (1992)**) and $\beta \rightarrow 1$ above this in a turbulent layer. The model includes also a time-varying sediment pick-up function that is linked to the phase of eddy shedding from the ripples during the wave cycle. The initial results presented by **Davies and Thorne (2002)** based on this approach were for the time-averaged features of the concentration field and sand transport rate in wave and current flows, and included computations for graded sand sizes in suspension. It should be noted that, in cases in which the waves and currents are combined at some general angle of attack, a direction-dependent bed roughness should probably be employed (**Barrantes and Madsen, 2000**). Such an approach was used by **Davies and Thorne (2002)**.

Although most simplified 1DV engineering formulations have tended to be based upon turbulent-diffusion concepts, a rather different argument has been developed by **Nielsen (1992)** involving combined convection and diffusion. It is argued by Nielsen that convection should be important for steeply rippled beds above which eddy shedding occurs, since the transport of sediment in discrete eddies (convection) represents a qualitatively different process from transport by random turbulent processes (diffusion). The distinction between the convection and diffusion processes, as represented by Nielsen, is that the *same* time-mean (relative) concentration profile is obtained for all grain sizes (i.e. regardless of settling velocity w_s) in the case of pure convection, whereas a different profile is obtained for each grain size in the case of pure diffusion. In practice, the shape of the mean concentration profile in Nielsen's model is forced, at least in part, by the bottom (pick up) boundary condition. However, using the tools that the combined convection-diffusion approach provides, Nielsen has shown that it is possible to obtain concave-up or convex-up concentration profiles for different grain sizes in the same flow conditions, as found in his field observations. The convection-diffusion approach has been used by, for example, **Lee and Hanes (1996)** in connection with field data obtained off the coast of Florida. It has also been critically assessed by **Thorne et al. (2002b)** in connection with acoustic (ABS) data obtained above ripples in the Deltaflume. They found that measured mean concentration profiles could be represented well by use of (a rescaled version of) **Nielsen's (1992)** pure convection and combined convection-diffusion approaches. They argued further, however, that the same results could be obtained, at least for the time-mean concentration profile, by use of diffusion concepts, provided that an eddy viscosity structure appropriate for rippled beds (c.f. **Van Rijn, 1993**) was used.

Practical models

Quasi-steady models developed for the wave-related bed-load and suspended load transport in the flat bed regime are commonly used also in the ripple regime, e.g. the Bagnold-Bailard model (**Bailard, 1981; Bailard and Inman, 1981**). However, this type of model generally yields net transport in the forward (onshore) direction (e.g. **Houwman and Ruessink, 1996**). An exception is the model of **Sato and Horikawa (1986)** which produces net offshore-directed transport beneath asymmetrical waves in the ripple regime. This difficulty was addressed by **Nielsen (1988)** who proposed three simple, fairly practical modelling approaches for the prediction of net sand transport rates by asymmetrical waves above rippled beds. These included (i) a diffusion-type model involving sediment pick-up peaks of unequal size following the passage of the wave crest and trough; (ii) a heuristic, convection-type, entrainment model; and (iii) a simple, but very effective, 'grab and dump' model. In comparisons with data, this third model turned out to be the most reliable for very coarse sands. Nielsen pointed out that the net transport in each model was in the 'offshore' direction, i.e. in the direction opposite to that of the largest (absolute) velocity. This is in good agreement with experimental evidence, but contrary to quasi-steady models like that of **Bailard and Inman (1981)**, which generally predict net transport in the 'onshore' direction above rippled beds. Nielsen concluded that the validity of such quasi-steady models is restricted to flat bed conditions.

Local sand transport prediction

Prior to the start of the **EU SandPit Project**, the predictive ability of various research and practical sand transport models was assessed, over wide ranges of combined wave and current conditions of practical importance, as part of the earlier **EU SEDMOC Project (1998-2001)**. The 'state-of-the-art' results, which were reported by **Davies et al. (2002)**, were summarised as follows.

Initially, seven 'research' models were intercompared over a wide range of wave and current conditions above a sand bed of given grain size. The models were of differing complexity, including one-dimensional (1DV) and two-dimensional (2DV) turbulence models and also a two-phase flow formulation (see Table 1.6.2).

Model	Type	Reference
1DV 'STP' eddy viscosity	1	Fredsøe et al. (1985)
1DV mixing length	1	Ribberink and Al-Salem (1995) Dohmen-Janssen et al. (2001)
1DV One-equation k- closure	2	Davies and Li (1997), Davies and Villaret (2000)
1DV Two-equation k-L closure	1	Huynh-Thanh et al. (1994), Guizien and Silva (2000)
1DV two-phase flow	-	Villaret et al. (2000)
2DHV Two-equation k-ε closure	3	Brors (1999), Utnes and Meling (1998)
2DHV Two-equation k-ω closure	3	Andersen and Fredsøe (1999)

Table 1.6.2 *Research models intercompared by* ***Davies et al. (2002)***
Type1: Standard plane bed modelling approach with enhanced roughness (k_s) for rippled beds
Type 2: Two-layer model including eddy viscosity to represent ripple-related vortex effects
Type 3: Ripple-related vortices included explicitly in flow model in 2DHV domain

A key element in the comparison was that the bed roughness (k_s) was specified, allowing the performance of the different models to be assessed for essentially the same input conditions. The results of the comparison showed that, in plane bed cases (involving large waves and/or strong currents), the research models agreed to well within an order of magnitude, the differences between individual models being attributable to differences between the respective turbulence-closure schemes. However, in cases involving rippled beds (lower waves and weaker currents), the agreement between the models was less convincing, with predicted transport rates differing by up to two orders of magnitude in some cases. This disagreement was probably attributable to the different ways in which the bottom boundary condition for suspended sediment was implemented in the respective models.

A second intercomparison, carried out between five 'practical' sand transport models, namely the model of **Bijker (1992)** (together with the implementation of the Bijker model by **Davies and Villaret (2002)**), the model of **Dibajnia and Watanabe** (as implemented by **Silva and Temperville (2000)**, the TRANSPOR2000 model of **Van Rijn (2000)**, the Bagnold-Bailard model (**Bagnold, 1966; Bailard, 1981**), and the SEDFLUX model of **Damgaard et al. (1996; 2001)** and **Soulsby (1997)**. The results of this comparison showed a similar level of agreement between the predicted sand transport rates. For plane beds, the transport rates predicted by the models varied by a factor of between 10 and 30 in the individual cases. [This is somewhat higher than the equivalent variation found for the research models.] For the rippled beds, the variation was in the range 50 to 200.

A final set of comparisons was carried out between (mainly) practical sand transport models and field data obtained at a variety of sites with differing grain size characteristics. Here the results showed that suspended sand concentrations in the bottom metre of the flow were predicted within a factor of 2 of the measured values in 13 to 48% of the cases considered, and within a factor of 10 in 70 to 83% of the cases. There was a tendency for low concentrations to be underpredicted, due probably to a residual amount of suspended sediment in quiescent conditions (e.g. near slack water) caused by turbulence not included in the model formulations. However, higher concentrations were predicted more convincingly. Use of a multiple grain size approach appeared to provide considerable advantages. As far as the prediction of sand transport rates is concerned, these comparisons were more encouraging. Estimates of the longshore (current-related) component of suspended sand transport made by five models yielded agreement within a factor of 2 of the measured values in 22% to 66% of cases, and within a factor of 10 in 77% to 100% of cases.

In all of the above comparisons, *untuned* research and practical models were used. The research models tended to have been validated in laboratory conditions, and then adapted for field use, while the practical models were designed and calibrated for field use. It is not surprising, therefore, that the more sophisticated research models did no better in some of the comparisons than the relatively simple practical methods. It was noted also that the research models produced results that appeared to be more volatile than those from the simpler practical approaches through the parameter ranges considered. This was probably due to the greater sensitivity of the research models to variations in the bed roughness (k_s).

While the research models were shown in the intercomparisons to be capable of predicting transport rates at field scales as well as practical models, it was noted by **Davies et al. (2002)** that their true benefit lies in the diagnostic analysis that they make possible. The study of the detailed processes included in research models, of which the 'wave-related' component of suspended transport is an obvious example, is still needed to improve our understanding of sediment transport phenomena. However, the difficulty in implementing research models in, for example, coastal morphological systems, mitigates against their use; the use of more practical models still provides an easier and more robust means of estimating sediment transport rates for this purpose.

Measurements of sand transport rates in the field are still rather sparse, making it difficult to validate local sand transport models. From the comparisons presented by **Davies et al. (2002)** it was concluded that the emphasis in future work should be directed towards improved models for *rippled* bed conditions. These conditions, which are prevalent in offshore sandy regions on the continental shelf, represent the main focus of sand transport research in the SandPit Project. **Vincent and Hanes (2002)** have pointed out that, in the field, considerable variability (factor of 2 or more) should be expected in the suspended concentration due to turbulence produced by wave boundary layers, where bedforms are frequently continually evolving as the waves change their height, period and direction. **Davies et al. (2002)** highlighted the need for improved prediction methods for bed form dimensions and, hence, the bed roughness (k_s). The most difficult quantity to specify in some models is the bed roughness, which is usually not known from measurements. The specification of the roughness has a profound effect on results for the net sand transport rate, a relatively small change in k_s having a large effect on the computed transport. Unfortunately, the accuracy of the modelling procedures used cannot be readily checked from most field observations. This remains an important topic for future research.

The comparisons presented by **Davies et al. (2002)** concentrated primarily on the differences between the absolute magnitudes of the transport rates predicted by different models, and on the differences between model predictions and field measurements of suspended concentrations and transport rates. Although such differences were found to be substantial, **Davies et al. (2002)** emphasised that it is not only the ability of models to make accurate *absolute* predictions that is important in sediment transport research. From the point of view of the morphological modeller, what is equally important, and possibly more so, is the *relative* behaviour of models. In particular, it is important that a transport model shows the correct behaviour i) as a function of the input parameters (waves, current and grain size) and ii) over a wide range of conditions involving several orders of magnitude in the transport rate. The correct behaviour of a transport model in this sense is a necessary condition for calibrating a morphological model and obtaining the correct morphodynamic behaviour. Since there was much more agreement in the relative behaviour of many of the models than in their ability to produce the same absolute magnitudes for the transport rate, it was felt that the outcome of the comparisons could be viewed as encouraging from the point of view of morphological modellers. It should be added, however, that, even if a local sand transport model is highly accurate, it does not necessarily follow that correct predictions will result from its use in a coastal morphological scheme. Horizontal advection-diffusion effects on the suspended sand transport, which are not addressed in local transport modelling (effectively at sub-grid scales), also need to be taken into account on the appropriate spatial scales. This requires the use of either a fully three-dimensional morphological modelling scheme, or a sufficiently sophisticated 'lag-function' approach in respect of the suspended transport in a two-dimensional (2DHV) morphological model.

Davies et al. (2002) concluded finally that, if a user selects a research or practical model randomly 'off the shelf', and then uses it in an 'untuned' manner to make predictions for field conditions, considerable uncertainty is to be expected. The state-of-the-art in sand transport research still required some knowledge of conditions on site, allowing the user to carry out model validation and/or tuning and, hence, make an informed judgement about the optimum choice of model for use in sand transport computations.

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1.5.3 Morphological prediction models

This section gives an overview of the available models to simulate sediment transport and morphodynamic behaviour of mined (deepened) areas on the lower shoreface.
Various methods are discussed: simple engineering rules, analytical models and mathematical.
For each type of model an overview is given of the modelled processes, the basic simplifications and the types of boundary conditions which have to be specified. Furthermore, the suitability of the models for the various characteristic morphological areas (e.g. surfzone, lower shoreface) is indicated.

Basic equations

The most general description of the transport processes and associated morphological bed evolution (deposition, erosion and migration) can be obtained by a three-dimensional approach (see Figure 1.6.2) based on numerical solution of the advection-diffusion equation, which reads as:

$$\partial c/\partial t+\partial(uc)/\partial x+\partial(vc)/\partial y+\partial((w-w_s)c)/\partial z-\partial(\varepsilon_{s,x}\partial c/\partial x)/\partial x-\partial(\varepsilon_{s,y}\partial c/\partial y)/\partial y-\partial(\varepsilon_{s,z}\partial c/\partial z)/\partial z=0$$

with: c=suspended sediment concentration, u,v,w= fluid velocities in x,y and z directions, z=vertical direction, w_s= settling velocity, $\varepsilon_{s,x}$, $\varepsilon_{s,y}$, $\varepsilon_{s,z}$= sediment mixing coefficients in x, y and z directions.

The depth-integrated (from the top of the bed load layer to the water surface) suspended transport rates in x and y-directions are defined by:

$$q_{s,x}=\int(uc-\varepsilon_{s,x}\partial c/\partial x)dz \text{ and } q_{s,y}=\int(vc-\varepsilon_{s,y}\partial c/\partial x)dz$$

The bed level evolution follows from:

$$\partial z_b/\partial t+\partial(q_{t,x})/\partial x+\partial(q_{t,y})/\partial y=0$$

with z_b= bed level to horizontal reference plane, $q_{t,x}$=total depth-integrated sediment transport (bed load plus suspended load transport) in x-direction, $q_{t,y}$= total depth-integrated sediment transport (bed load plus suspended load transport) in y-direction.

These equations can be solved for given flow velocities, sediment mixing coefficients, settling velocity, sediment concentrations at all boundaries and at initial time (t=0).

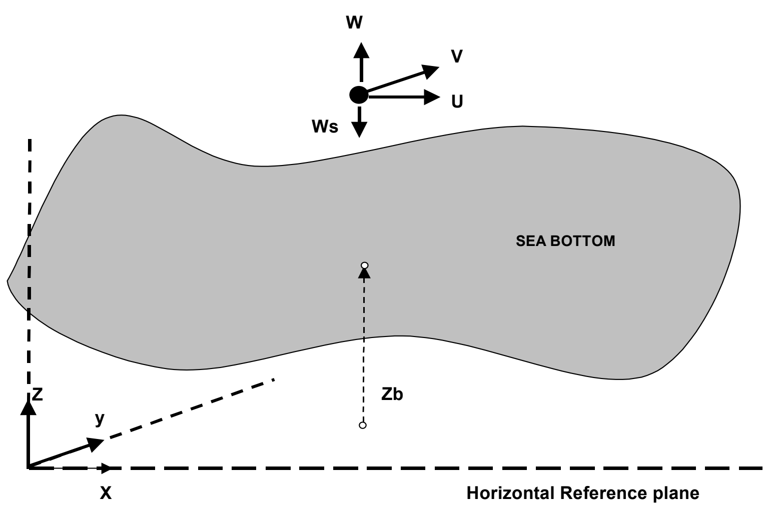

Figure 1.6.2 *Definition sketch for three-dimensional conditions*

A two-dimensional vertical approach can be applied in case of a relatively long deepened area (channel or trench). This situation basically refers to the modelling of one streamtube crossing the channel. The advection-diffusion equation reads as:

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

with: b=width of the streamtube.

Simple engineering rules

The reduction of the suspended sediment in a relatively long channel (1D approach) due to decrease of the transport capacity can schematised quite simply by an exponential decay curve. Assuming an oblique flow direction and a rectangular cross-section (Figure 1.6.3), the reduction of the suspended sediment transport ($q_{s,x}$) is:

$$q_{s,x}= (b_0/b_1)q_{s,0} - [(b_0/b_1)q_{s,0} - q_{s,1}](1-e^{-Ax})= (b_0/b_1)q_{s,0} (e^{-Ax})+q_{s,1}(1-e^{-Ax})$$

with: b_0= streamtube width of approaching flow; b_1=streamtube width in channel, $q_{s,0}$=suspended transport capacity of approaching flow (per unit width), $q_{s,1}$= suspended transport capacity in channel (per unit width), x= coordinate along streamtube, A=coefficient.

This approach has been frequently used to estimate the backfilling of relatively narrow navigation channels and pipeline trenches **(Mayor et al, 1976; Lean, 1980; Bijker, 1980 and Eysink-Vermaas, 1983)**. The trapezoidal cross-section is schematised to a rectangular cross-section making intersections halfway the side slopes. The effective deposition length is the length between the intersection points, yielding L=B/(sinα_1).

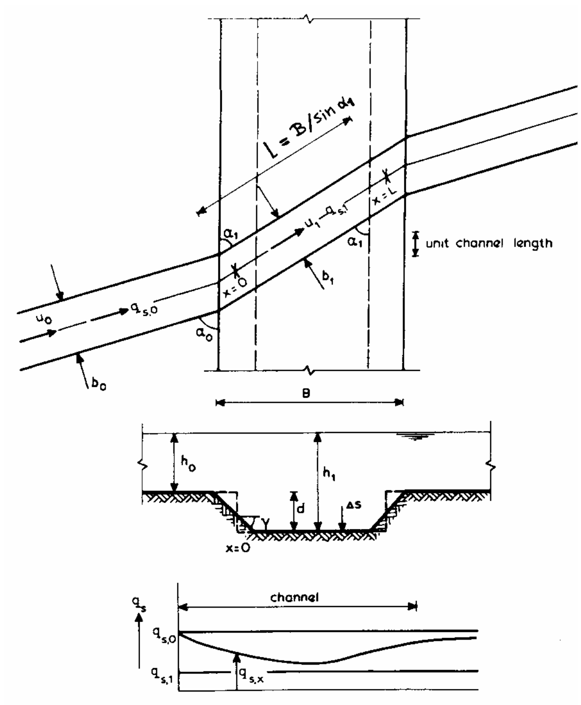

Figure 1.6.3 *Definition sketch*

Van Rijn (1987) has presented trapping efficiency graphs, which can be used to determine the trapping efficiency of sediments from oblique and cross flow over an infinitely long channel. The graphs are based on simulations (about 300) using a 2DV-model.

The trapping efficiency (e) is defined as the relative difference of the incoming suspended load transport ($q_{s,o}$) and the minimum suspended load transport in the channel ($q_{s,1,minimum}$).

$$e=(b_o q_{s,o}-b_1 q_{s,1,minimum})/(b_o q_{s,o})$$

The basic parameters determining the trapping efficiency of a channel, are: approach velocity ($v_{r,o}$), approach angle (α_o), approach depth (h_o), approach bed-shear velocity ($u_{*,o}$), particle fall velocity (w_s), wave height (H), channel depth (d), channel width (B), channel side slopes ($\tan\gamma$) and bed roughness (k_s).
The influence of the relative wave height (H/h) and relative bed roughness (k_s/h) on the trapping efficiency (e) is relatively small and has therefore been neglected. The error of the e-parameter is about 25% for an approach velocity in the range of 0.8 to 1.2 m/s, H/h varying in the range of 0 to 0.3 and k_s/h varying in the range of 0.02 to 0.06.
The sedimentation rate (ΔS) per unit channel length immediately after dredging can be determined from:

$$\Delta S= (e_s\, q_{s,o} + e_b q_{b,o}) \sin\alpha_o$$

with: e_s= trapping efficiency for suspended load, e_b= trapping efficiency (about 1) for bed load, $q_{s,o}$ = incoming suspended load transport per unit width, $q_{b,o}$ = incoming bed load transport per unit width.

Analytical models for flow perpendicular to pit

Analytical models based on relatively simple equations representing the basic physics have been developed to study the morphological behaviour of schematised mining pits.
Van der Kreeke et al. (2001) have studied the morphological behaviour of a channel (initial cross-section schematised as Gaussian function, see Figure 1.6.4), which is subject to tidal currents and waves. The tidal currents are rectilinear and perpendicular to the longitudinal axis of the pit. The tidal velocity comprises the residual velocity M_0 and the harmonic constituents M_2 and M_4. Separate equations have been derived for bed load and suspended load transport.
In case of dominant bed-load transport conditions the bed level changes are represented by:

$$\partial a/\partial t+\partial q_b/\partial x=0$$

with a=channel depth below the surrounding bed and q_b= bed load transport.
The bed load transport (q_b) is represented by:

$$q_b=fU^3[(1+|U|)\lambda\partial a/\partial x]$$

with U= depth-averaged current velocity, f= coefficient (including friction effect), λ= coefficient. The effect of waves on the sediment transport is incorporated by use of a stirring term. The equations are averaged over the tidal time scale assuming that the morphological time scale is large compared to the tidal period. To determine the leading order terms, the variables are scaled and the equations are written in dimensionless form. The relative magnitude of the terms is determined by a small parameter, being the ratio of channel depth and undisturbed water depth. Retaining only leading order terms, the equation for the bed level reduce to an advection-diffusion equation with constant coefficients:

$$\partial a/\partial t+c(\partial a/\partial x)-K(\partial^2 a/\partial x^2)=0$$

with c= migration velocity and K=diffusion coefficient. The initial Gaussian cross-section (see Figure 1.6.4) migrates with speed c and the width σ increases (widening) at rate $d\sigma/dt=(0.5K/t)^{0.5}$.

In case of dominant suspended load transport the bed level changes are represented by:

$$\partial a/\partial t+\alpha(E-S)=0$$

with E= erosion function, S= deposition function, E-S= $w(c_a-c)$, c_a= near-bed concentration, c= concentration, w= fall velocity, α= coefficient (including sediment density and porosity). The settling term is described as: $wc_a = \gamma U^2$ with γ=coefficient.
The suspended load transport is modelled as:

$$\partial c(h+a)/\partial t+\partial Uc(h+a)/\partial x-D(h+a)\partial^2 c/\partial x^2=E-S$$

with D= dispersion coefficient (about 10 m^2/s). The first order solution can also be described by an advection-diffusion equation with constant coefficients.

The analytical solution shows that the channel cross-section migrates, widens and shallows. The velocity of migration is a function of the residual velocity and the amplitudes and phases of the M_2 and M_4 constituents. Widening and shallowing is a result of diffusion. For bed-load transport, diffusion derives from the effect of the bed slope on the sediment transport. For suspended load transport, diffusion is assumed to be the result of velocity shear and vertical turbulent mass exchange (shear dispersion). When accounting for higher order non-linear terms, an initially symmetric cross-section becomes asymmetric. Application to the access channel (water depth h= 17 m, maximum channel depth a_{max}= 3 m, U_{max}= 0.55 m/s, fall velocity= 0.02 m/s) to the port of Amsterdam yields channel displacement velocities of c= 1.3 m/year for bed-load transport case and c= 1.8 m/year for suspended load transport case. The diffusion coefficient is about K= 300 m^2/year.

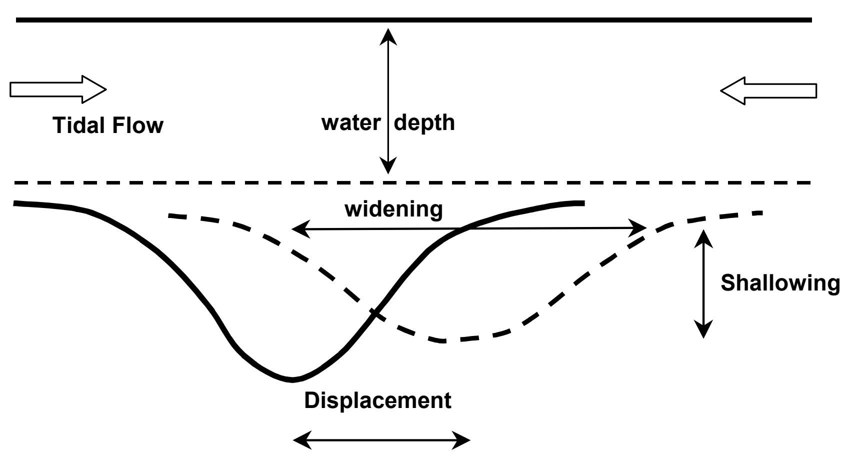

Figure 1.6.4 *Migration of Gaussian-shaped pit*

Analytical models for flow parallel to pit

Fredsøe (1978) has proposed a method to compute the morphological behaviour of a channel parallel to the flow by considering the effects of gravity forces and drag forces on the bed-load particles moving on the side slopes of the channel. The resulting force is inclined towards the channel bottom and thus causes infill of the channel (see Figure 1.6.5). The angle ψ between the direction of the force on the particle and the flow line is assumed to be equal to the ratio of the transverse side slope angle γ and the dynamic friction angle ϕ of the bed material particles, which reads as:

$$\tan \psi = \tan \gamma / \tan \phi$$

Assuming that the variation of the bed-load transport over the side slope is relatively small, a diffusion-type equation for the bed level can be derived, as follows:

$$\partial z_b/\partial t= \zeta\, \partial^2 z_b/\partial x^2$$

with: $\zeta=q_{bo}/[(1-p)(\tan\phi)]$= constant diffusion coefficient (in m^3/ms), q_{bo}= bed-load transport in flow direction on middle of slope (in m^3/ms), $\tan\phi$=dynamic friction angle (about 1.5), p= porosity factor (0.4), x=coordinate transverse to channel axis, z_b= bed level to channel bottom (Fig. 1.6.5).

The volume of sand transported from the side slopes into the channel after time t can be expressed as:

$$\Delta S= 2d\, (\zeta/\pi)^{0.5}\, [(t+t_o)^{0.5} - t_o^{0.5}]$$

with ΔS= infill volume of sand (in m^3/m) after time t, d= initial channel depth (below surrounding bottom), $t_o=(\pi d^2)/(64\,\zeta\,\tan\gamma)$=coefficient (in seconds), γ= initial transverse side slope angle, t= time (in seconds).

According to **Fredsøe (1978)**, this approach is also valid for currents crossing a channel under a small angle relative to the channel axis. In that case the longitudinal component of the bed-load transport on the side slope should be used into the equation for channel sedimentation.

Another approach is based on the transverse bed-load transport ($q_{b,n}$) on the side slope of a straight channel. This was studied by **Ikeda (1982)**, yielding the following formula:

$$q_{b,n}=[1.5(\tau_{b,cr}/\tau_b)^{0.5}\tan\gamma]q_{bo}$$

with $\tau_{b,cr}$= critical bed-shear stress and τ_b= bed-shear stress exerted by the flow (and/or waves).

The sedimentation (in m^3/m) in the horizontal bottom section of the channel by lateral bed-load transport from two side slopes after time t (in s) can be determined as:

$$\Delta S= 2\,(1-p)^{-1}\, t\, [1.5(\tau_{b,cr}/\tau_b)^{0.5}\tan\gamma]q_{bo}$$

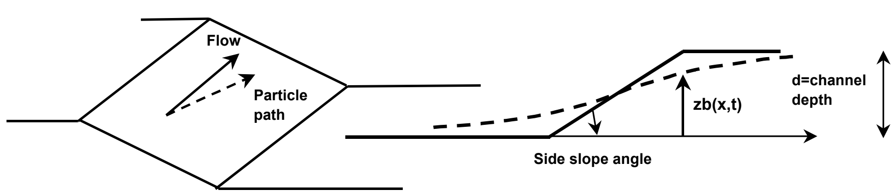

Figure 1.6.5 *Morphological behaviour of side slope in parallel flow*

Two dimensional vertical numerical model for flow perpendicular or oblique a pit

This type of approach can be used for the simulation of bed-load and suspended load transport under conditions of combined quasi-steady currents and wind-induced waves over a pit/channel/trench oblique to the flow, as shown in Figure 1.6.6. Such a model has been developed at Delft Hydraulics **(Van Rijn and Tan, 1985; Van Rijn, 1986a,b; Delft Hydraulics, 1985)** and elsewhere. All processes (parameters) in the direction normal to the streamtube direction are assumed to be constant. These models can be applied at a space-scale of 1 to 5 km and at a time scale of 1 to 100 years.

Basic processes taken into account, are:

Hydrodynamics

- modification of velocity profile and associated bed-shear stress due to presence of waves,
- modification of velocity and associated bed-shear stress due to the presence of 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.

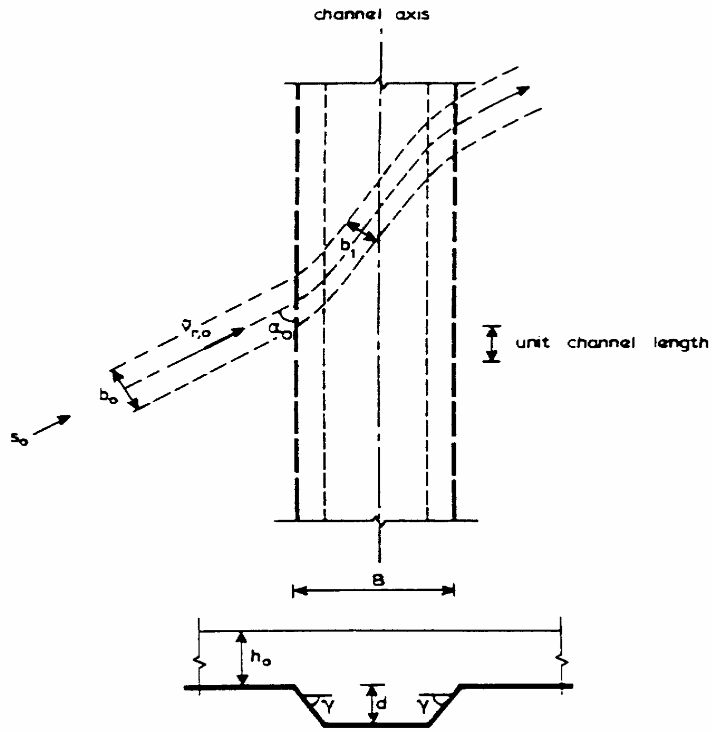

Figure 1.6.6 *Oblique flow and transport across a channel/pit (two-dimensional streamtube approach)*

Basic simplifications are:

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

Boundary conditions to be specified, are:
- water depth, flow width (stream tube width, discharge is constant) and bed level along computation domain,
- wave heights along computation domain,
- equilibrium or non-equilibrium sediment concentrations at inlet (x= 0); model has option to generate equilibrium concentrations,

- bed concentration or bed concentration gradient is prescribed as function of bed-shear stress and sediment parameters,
- sediment, settling velocity, and bed roughness.

Two dimensional vertical numerical model for oblique wave conditions

This type of approach can be used for the simulation of bed-load and suspended load transport and associated morphological changes under conditions of wind-induced (breaking) waves over a shoreparallel pit/channel/trench, see Figure 1.6.7. All longshore processes (parameters) are assumed to be constant (uniform coast). These models can be applied at a space-scale of 1 to 5 km and at a time scale of 1 to 10 years. These models have primarily been designed to model the (cross shore) morphodynamic behaviour of the surfzone under the influence of (breaking) waves, wind and tidal currents.

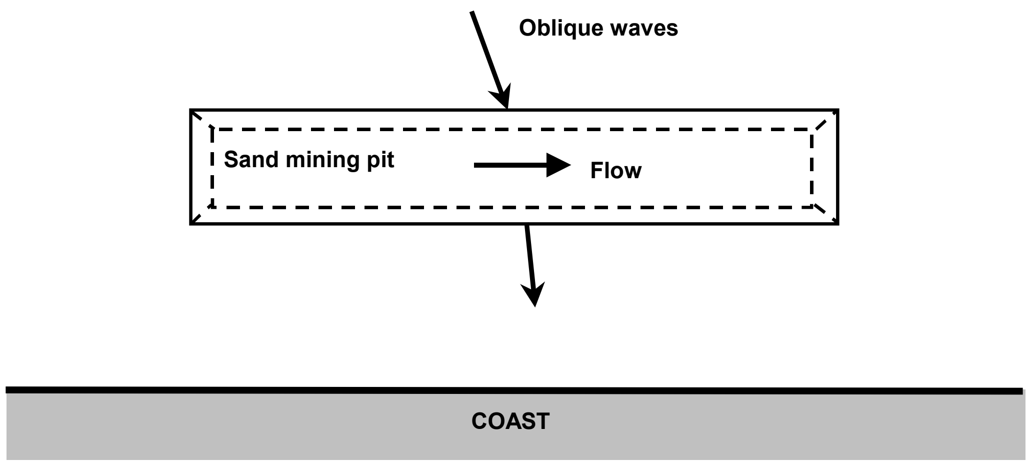

Figure 1.6.7 *Oblique waves over a channel/pit in cross-shore direction (alongshore uniform conditions)*

Basic processes taken into account, are:

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

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

Basic simplifications are:

Hydrodynamic
- deterministic spectral approach for wave field,
- linear wave theory,
- parameterised approach for long-period waves,
- no effect of pit on longshore tidal current,
- quasi-steady approach for mean currents,

Sediment transport

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

Boundary conditions to be specified, are:

- mean water depth and bed level along computation domain,
- rms-wave height and period at inlet (x=0),
- mean longshore current at inlet,
- sand transport at outlet (beach boundary),
- sediment, bed roughness and calibration parameters.

Two dimensional numerical model for shoreparallel conditions

This type of approach can be used for the simulation of bed-load and suspended load transport and associated morphological changes under conditions of shoreparallel flow in a pit/channel/trench, see Figure 1.6.8. All cross-shore processes (parameters) are assumed to be constant (horizontal bottom). These models can be used at a space-scale of 1 to 10 km and at a time scale of 1 to 50 years.

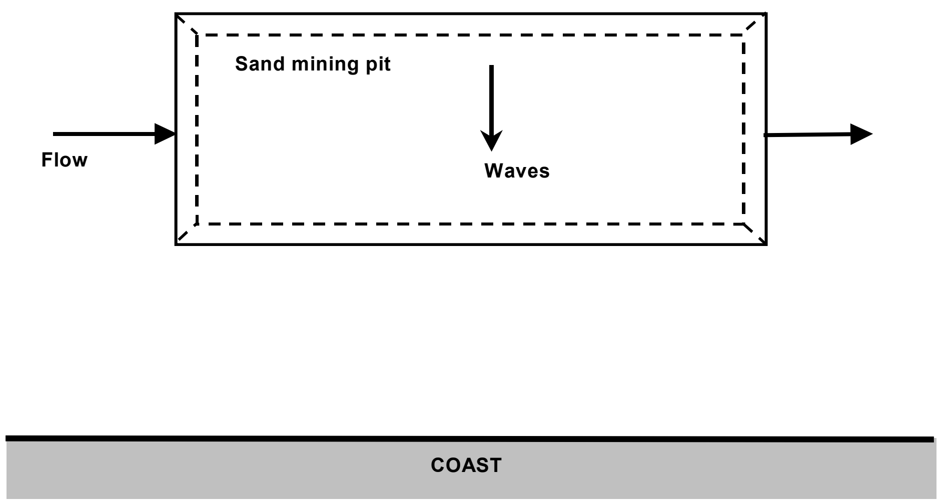

Figure 1.6.8 *Shoreparallel flow over channel/pit in longshore direction (cross-shore uniform conditions)*

Basic processes taken into account, are **(Rijkswaterstaat, 1990)**:

Hydrodynamic

- depth-averaged longshore tidal flow including advection and bottom friction effects,
- acceleration and deceleration effects at upstream and downstream slopes,
- constant wave height, near-bed orbital velocity based on linear wave theory.

Sediment transport

- bed-load transport due to combined current and wave velocities based on Bailard-Bagnold approach,
- horizontal advective suspended load transport,
- equilibrium suspended load transport based on Bailard-Bagnold approach,
- spatial lag effects (Galapatti method).

Basic simplifications are:
Hydrodynamic
- constant wave height in pit,
- depth-averaged flow without wave-current interaction,
- quasi-steady approach for mean currents (tidal cycle schematised into quasi-steady periods),

Sediment transport
- no vertical and longitudinal mixing (diffusion),
- no wave-related transport (no oscillatory transport components),
- no slope effect on bed load transport,
- uniform grain size (no mixtures),

Boundary conditions to be specified, are:
- mean water depth and bed level along computation domain,
- mean longshore current at upstream boundary (x=0),
- rms-wave height and period,
- sediment, bed roughness and calibration parameters.

Two dimensional depth-averaged and three dimensional numerical models

These types of area models consist of a set of submodels, simulating waves, currents, sand transport and bed evolution on a 2DH or 3D grid. These models are based on the numerical solution of the hydrodynamics and sediment dynamics on short time scales (seconds to hours) and associated morphological bed level changes in a loop (feed back) system.
The model system consists of four main modules: Flow, Wave, Transport and Bottom.
The sand transport computations can be operated either in an 'off-line' or an 'on-line' mode. Numerical solution methods are based on finite differences (rectangular of curvilinear grids) or finite elements.
Using a detailed process approach, the prediction horizon is generally limited to a few years at a spatial scale of 10 to 100 km at present stage of research An efficient method to increase the prediction horizon is the integration of the sediment transport processes over longer time scales **(Roelvink et al., 1998; Roelvink et al., 2001)**.
Basic processes taken into account, are:
Hydrodynamic
- wave energy propagation, shoaling, diffraction and refraction,
- wave energy dissipation by bottom friction and wave breaking,
- wave growth by wind input,
- non-steady fluid motion (homogeneous and non-homogeneous) based on continuity and momentum equations,
- advanced turbulence models for modelling of internal shear stresses,
- modification of bed-shear stress due to presence of waves,

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

Basic simplifications are:
Hydrodynamic
- deterministic spectral approach for wave field,
- linear wave theory,

Sediment transport and morhology
- relatively uniform grain sizes (no mixtures and no segration effects in surface bed layer),
- relatively small concentrations (no stratified flow).

1.6 Overview and results of existing pit studies

Several studies have been performed related to the behaviour of navigation channels and sand mining pits in the Dutch sector of the North sea, being (see also **Van Rijn and Walstra, 2002**):

Morphology of mining pits and mining from IJ-channel; Rijkswaterstaat (1990).
Mining of sand from the IJ-channel (navigation channel to Port of Amsterdam) by widening and/or deepening of the channel is found to have no significant effect on the far-field shoreline. The local shoreline close to the breakwaters may be modified slightly due to changes of the local wave-current conditions.

Morphology of mining from EURO-MAAS channel; Delft Hydraulics (1992).
The morphological changes are found to be limited to an area of one or two kilometres on both sides of the widened channel. The effects on the local wave and current patterns are so small, that appreciable effects on the shoreline are not to be expected.

Morphology of large scale mining pits near EURO-MAAS channel; Delft Hydraulics-Alkyon (1997).
Based on the long-term morphological model results, the migration velocity of the mining pits was found to be 10 to 15 m/year. The morphological changes remain within the local surrounding of the mining pits. On the time scale of 100 years the overall migration of the mining pit will be of the order of 1 to 2 km, which is smaller than the width (about 3 km) of the deep mining pit. The effect of the mining pit scenarios on the nearshore coastal zone is found to be negligibly small. The morphological effect of the construction of the MAASVLAKTE 2 extension on the nearshore coastal zone is found to be substantially larger than the effects of the sand mining pits.

Flow in large scale mining pits (L=20 km, W=2 km, pit depth= 30 m, outside depth= 20 m); Svasek (1998).
The maximum flow velocity in the middle section of the pit is smaller than the upstream velocity for a pit length smaller than two times the pit width (L<2W).
The maximum flow velocity in the middle of the pit increases (10% to 20%) with increasing Length-Width ratio, in case of constant pit depth.
The maximum flow velocity in the middle of the pit increases (10% to 20%) with decreasing Width-Pit depth ratio, in case of a constant pit length.
The maximum flow velocity in the pit decreases for approach angles of 30^o and 60^o; the residual velocities at the inflow and outflow sections are largest for an angle of 60^o.

Morphology (2DV) of mining pits; Hoitink (1997), University of Twente and Delft Hydraulics (1998).
Model runs (sand of about 0.2 mm) show that the net displacement of centre of the side slopes is of the order of 500 m,; the total area of influence in the direction of the tidal flow is about 10 km on a time scale of 50 years. The cross-shore migration rates of the pit (with depth of 2 m) is about 2 m/year.
A shallow channel (2 m) shows a relatively large horizontal displacement and a relatively large sedimentation after 50 years.
A deep channel (14 m) shows a relatively small horizontal displacement and sedimentation after 50 years.
A wide channel (1200 and 2400 m) is characterised by sedimentation on the upstream side slope and erosion on the downstream slope; the sedimentation in the middle of the channel is minimum after 50 years.
A relatively narrow channel (300 m) shows relatively large sedimentation in the middle; the channel depth is reduced by about 40% after 50 years.
A channel with a side slope of 1:50 yields a wider channel and hence less sedimentation in the middle of the channel compared to a channel with a slope of 1:12.5.
The water depth outside the channel has a relatively large effect on the channel sedimentation; a small depth of 15 m yields a much larger wave effect resulting in a significant increase of the incoming sand transport.

Flow and morphology (2DH) in large scale mining pits (lengths between 2 and 100 km, width between 5 and 10 km, pit depth= 30 m, outside depth=20 m); Klein (1999), Delft University of Technology, Netherlands
The side slopes of the pits are flattened (erosion).
A pit parallel (0^o) to the flow or under a positive angle (45^o) migrates in the direction of the main longitudinal axis; the maximum migration distance after 1000 years is about 5 km.
A pit under a negative angle of -45^o migrates in the direction normal to the main longitudinal axis (Coriolis

effect); the maximum migration distance after 1000 years is about 3 km.
The depth of a pit parallel to the flow or under a positive angle (45^o) tends to become somewhat larger; deposition is remarkably small after 1000 years.
The rectangular and square planforms of the pits are transformed into less regular shapes.

Morphology (2DH) of large scale mining pit using stability analysis; Hulscher et al. (1993), Németh (1998) and Roos et al. (2001), University of Twente, Netherlands.
The presence of a sand pit triggers the formation of circulation cells resulting in the development of a sand bank pattern. As time evolves, the sand bank pattern spreads out and migrates, alternatingly generating trough and crest zones. The pit itself deepens and the pattern spreads at a rate of 10 to 100 m/year. The migration rate of the centre of the pit is of the order of 1 to 10 m/year.

The outcome of these studies generally is that the flow and morphodynamics can be simulated quite well provided that the boundary conditions of flow, waves and sediment transport at the model inlet are accurately known. In the absence of such data the uncertainty margins are relatively large (up to factor 5). Furthermore, the models have not yet been verified extensively due to lack of field data. Field data sets at deeper water are almost completely missing.

The results of various other studies in the UK and in the USA are:

Shoreline changes due to modification of wave refraction patterns over borrw pits; Motyka and Willis (1974) and Price et al. (1978).
It is concluded that extraction pits beyond the 14 m depth contour donot lead to any significant shoreline erosion.

Sand extraction in central Gulf of St. Lawrence, Canada; Anctil and Quellet (1990). Shoreline effects are found to be minimum if the dredging activities are performed in as deep water as possible and the dredging operations are planned in such a way that the seabed is excavated as gradual as possible (no multiple dredging at the same site) to allow ample time for the morphological system to adjust.

Sand extraxtion site offshore of Ocean City, Maryland, USA; Anders and Hansen (1990).
Numerical wave refraction studies have been conducted to examine potential erosional effects of extraction site mining on adjacent shorelines. The sites with sediment compositions similar to the required beach nourishment materials have finally been selected for dredging of sediment.

Sand extraction site offshore of Harwich, England; HR Wallingford (1983).
Based on all model results (physical and numerical), it is concluded that even for a very severe storm at the worst possible angle of incidence, the effect of dredging of a layer of 1 m from the bed is negligible on wave conditions at the shoreline.

Sand extraction sites offshore of Alabama, USA; Byrnes et al.(2004a).
Five sand resource areas in depths varying between about 10 and 20 m offshore of the Alabama coast (USA) were studied. Nearshore wave, current and sediment transport patterns were modelled for existing and post-dredging conditions, with extraction site sand volumes ranging from 2 to 8 million m^3. Wave transformation modelling results indicated that minor changes will occur to wave fields under typical seasonal conditions and sand extraction scenarios. Localized seafloor changes at extraction sites are expected to result in negligible impacts to prevailing wave climate at the coast. For all potential sites, the maximum variation in annual littoral transport related to the post-dredging bathymetry was predicted to be approximately 10%. Predicted infilling rates (annual maximum of 2% of initial volume) and sediment types were relatively small and within natural variations. Impacts on benthic communities were evaluated on the basis of earlier studies. The levels of of infaunal abundance and diversity are predicted to recover within 1 to 3 years, but recovery of species composition may take longer.

Sand extraction sites offshore of New Yersey, USA; Byrnes et al.(2004b).
Eight sand resource areas in depths varying between about 10 and 20 m offshore of the New Yersey coast (USA) were studied. Nearshore wave, current and sediment transport patterns were modelled for existing and post-dredging conditions, with extraction site sand volumes ranging from 2 to 9 million m^3. The study results are similar to those for the Alabama coast (see under 5.)

Sand extraction sites offshore of Maryland and Delaware, USA; Maa et al. and Diaz et al. (2004). Two offshore shoals in water depths between 10 and 20 m were identified as potential sand sources for beach-quality sand with a maximum cumulative volume of about 25 million m^3 (one-time removal of 2 million m^3). Wave transformation modelling was based on a series of runs of the REF/DIF-1 model with boundary conditions from a nearby, offshore wave gauge. Model calibration was applied using measured nearshore wave data. The modelled, post-dredging data indicated an increase in wave height up to a factor of 2 in the area between the dredged shoals and the shore. Current modelling indicated that dredging related changes are negligible. The effects on sediment transport patterns were not modelled. The impacts on benthos communities were studied by field sampling surveys. A mining scenario that removed the top meter of the shoal will disturb about 8 km^2 with the potential acute impact on about 150 million individuals representing about 300 kg of wet biomass. To minimize the impact, the total removal of the substrate should be avoided. Small areas within the project area should be left alone to serve as refuge patches. Project timing is recommended to lessen impacts (end mining activities in time for Spring/Summer recruitment of certain species).

Sand extraction site offshore of South-Central Louisiana, USA; Stone et al. (2004). A shoal located at the 10 m depth contour off south-central Louisiana was studied as a potential source of sand (about 1 billion m^3). Numerical wave modelling was applied (over prediction up to about 25%). Removal of the shoal causes a maximum increase of the significant wave heightby about 50% to 100% in the lee of the shoal complex for two storm events. Increased erosion along the Isles Dernieres is not expected to be caused by the nearshore wave changes. Directional waves, currents and sediment transport using OBS-sensors (optical backscatter) were measured during winter storms using three bottom-mounted arrays deployed on the seaward and landward sides of the shoal. Episodic increases in wave height, mean and oscillatory current speed, shear velocity, and sediment transport rates during storm events were measured. Waves tend to be higher and longer in period on the seaward side of the shoal. Mean currents are generally higher on the landward side

Sand extraction sites on the U.S. Gulf and Atlantic Continental Shelves; Hayes and Nairn (2004). Using a spectral or phase-resolving wave model combined with two-dimensional hydrodynamic and sand transport models, the behaviour of sand ridges and shoals was studied. Sand transport was found to converge along most of the crest of the shoal and net shoreward sand transport towards the steep flank of the shoal. Further model development focussing on the maintenance of these sand ridge features by wave-generated sand transport processes is recommended. At present the knowledge of ridge maintenance processes is too limited to be able to assess properly how much sand can be removed from a ridge or shoal without disrupting the processes that maintain the feature.

Shoreline response to offshore sand extraction at three U.S.A. sites; Kelley et al., (2004) Wave modifications of an idealized extraction site were studied using the STWAVE model to determine the effects of offshore bathymetry changes to wave conditions and resulting sediment transport patterns. The idealized extraction site was located approximately 400 m offshore, centered on the -10 m depth contour (below MSL) with an excavation depth of 3 m below the seafloor. Wave spectra, centered at -30^o, -15^o, 0^o,15^o and 30^o relative to shore normal, were modelled, each with the same significant wave height and peak period and having an equal percentage of occurrence (20%). The longshore range of influence at the breakerline is about 900 m on both sites of the site for wave directions of -30^o and 30^o. The longshore sand transport rate was computed using the CERC formula for different wave conditions and transformed into a normalized curve representing annual conditions. Based on gradients of the longshore sand transport rates, the shoreline changes were computed. Wave modelling was performed for three locations for a 20-year period and for 20 one-year blocks of the wave record. From these model runs sand transport potential curves were derived for average annual conditions and for one year periods. Based on this, the averages and standard deviations (σ) of the computed longshore transport rates were determined for every 200 m section of the shoreline. If any portion of the longshore transport potential curve associated with the extraction site exceeds $\pm 0.5\sigma$ of the normal temporal variability about the longshore transport potential curve determined for existing (pre-dredging) conditions, the site is rejected. Using this method, there is a 62% chance that the mean longshore transport for any given year is outside the $\pm 0.5\sigma$ envelope about the mean. The method was applied to determine the maximum excavation depth giving longshore transport rates just within the $\pm 0.5\sigma$ envelope at three sites: North Carolina, New Yersey and Florida.

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ANNEX 2

OVERVIEW OF NEW LABORATORY AND FIELD DATA

ANNEX 2A: NEW LABORATORY EXPERIMENTS

Data Source	Sand size (mm)	Water depth (m)	Slope	Waves	Current
		flow depth tunnel		2nd order Stokes waves	
Aberdeen Oscilatory Flow Tunnel Sandpit Experiments					
Series 1 (completed)					
Exploratory Trials					
Experimental programme:					
T1	d_{50} = 0.27	0.50	hor.	T = 5 s d = 1.1 m	none
T6	d_{50} = 0.27	0.50	hor.	T = 5 s d = 1.2 m	none
T2	d_{50} = 0.30	0.50	hor.	T = 5 s d = 1.1 m	none
T4	d_{50} = 0.30	0.50	hor.	T = 7.5s d = 1.3 m	none
T3	d_{50} = 0.30	0.50	hor.	T = 10 s d = 2.0 m	none
T5	d_{50} = 0.30	0.50	hor.	T = 10 s d = 2.2 m	none
T7	d_{50} = 0.32	0.50	hor.	T = 5 s d = 1.0 m	none
T8	d_{50} = 0.32	0.50	hor.	T = 5 s d = 1.2 m	none
Available data and parameters: -observations of resulting bed morphology to investigate the occurrence of 2-d and 3-d bedforms					
Regular and Irregular Flows					
Experimental programme:					
R2_10	d_{50} = 0.27	0.50	hor.	T = 10 s d = 1.21 m	none
R2b_10	d_{50} = 0.27	0.50	hor.	T = 10 s d = 1.51 m	none
R1_10	d_{50} = 0.27	0.50	hor	T = 10 s d = 2.2 m	none
R3_3	d_{50} = 0.32	0.50	hor.	T = 3.1 s d = 0.42 m	none
I3b_3	d_{50} = 0.32	0.50	hor.	Irregular flow T = 3.1* s d = 0.42* m	none
R5_3 (mr3)	d_{50} = 0.44	0.50	hor.	T = 3.1 s d = 0.42 m	none
I5_3 (mi3)	d_{50} = 0.44	0.50	hor.	Irregular flow T = 3.1* s d = 0.42* m	none
R4_5	d_{50} = 0.44	0.50	hor.	T = 5 s d = 1.1 m	none
R6_5 (mr5b)	d_{50} = 0.44	0.50	hor.	T = 5 s d = 0.99 m	none
I6_5 (mi5b)	d_{50} = 0.44	0.50	hor.	Irregular flow T = 5* s d = 0.99* m	none
r7_7 (mr7)	d_{50} = 0.44	0.50	hor.	T = 7.38 s d = 1.33 m	none
i7_7 (mi7)	d_{50} = 0.44	0.50	hor.	Irregular flow T = 7.38* s d = 1.33* m	none
r8_10 (mr10)	d_{50} = 0.44	0.50	hor.	T = 10 s d = 1.22 m	none
i8_10 (mi10)	d_{50} = 0.44	0.50	hor.	Irregular flow T = 10* s d = 1.22* m	none
r9_4 (mr4)	d_{50} = 0.44	0.50	hor.	T = 4.11 s d = 0.66 m	none
i9_4 (mi4)	d_{50} = 0.44	0.50	hor.	Irregular flow T = 4.11* s d = 0.66* m	none
r10_3 (fr3)	d_{50} = 0.22	0.50	hor.	T = 3.1 s d = 0.42 m	none
i10_3 (fi3)	d_{50} = 0.22	0.50	hor.	Irregular flow T = 3.1* s d = 0.42* m	none
r11_4 (fr4)	d_{50} = 0.22	0.50	hor.	T = 4.11 s d = 0.66 m	none
i11_4 (fi4)	d_{50} = 0.22	0.50	hor.	Irregular flow T = 4.11* s d = 0.66* m	none
r12_5 (fr5a)	d_{50} = 0.22	0.50	hor.	T = 5.02 s d = 0.72 m	none
i12_5 (fi5a)	d_{50} = 0.22	0.50	hor.	Irregular flow T = 5.02* s d = 0.72* m	none

	d_{50} = 0.22		hor.	T = 7.38 s d = 1.33 m	none
r13_7 (fr7)	d_{50} = 0.22	0.50	hor.	Irregular flow	none
i13_7 (fi7)		0.50		T = 7.38* s d = 1.33* m	
	d_{50} = 0.22		hor.	T = 10 s d = 1.22 m	none
r14_10 (fr10)	d_{50} = 0.22	0.50	hor.	Irregular flow	none
i14_10 (fi10)		0.50		T = 10* s d = 1.22* m	
Available data and parameters: -time series of bed profiles as bedforms grow from flat bed -bedform geometry -bedform migration rates -net sediment transport -size distribution of sediment collected in ‘onshore’ and ‘offshore’ end of the tunnel				* Denotes the irregular flow ‘equivalent’ T and d.	
Series 2 Experiments (ongoing experiments) Data as of 21/04/04: Ripple Development and Evolution					
Experimental programme:					
Exp #, starting bed				T=3.1s d=0.42m	
	d_{50} = 0.44		hor.	T=5s d=0.99m	none
Exp1, Flat Bed	d_{50} = 0.44	0.50	hor.	T=10s d=1.22m	none
Exp1, mr3	d_{50} = 0.44	0.50	hor.	T=5s d=0.99m	none
Exp2, Flat Bed	d_{50} = 0.44	0.50	hor.	T=4s d=0.48m	none
Exp2, mr10	d_{50} = 0.44	0.50	hor.	T=2s d=0.24m	none
Exp3, Flat Bed	d_{50} = 0.44	0.50	hor.	T=4s d=0.48m	none
Exp4, Flat Bed	d_{50} = 0.44	0.50	hor.	T=5s d=0.99m	none
Exp4, mr2	d_{50} = 0.44	0.50	hor.	T=4s d=0.48m	none
Exp5, Flat Bed	d_{50} = 0.44	0.50	hor.	T=9s d=1.30m	none
Exp5, mr5	d_{50} = 0.44	0.50	hor.	T=4s d=0.48m	none
Exp6, Flat Bed	d_{50} = 0.44	0.50	hor.	T=2.8s d=0.34m	none
Exp6, mr9	d_{50} = 0.44	0.50	hor.	T=4s d=0.48m	none
Exp7, Flat Bed	d_{50} = 0.44	0.50	hor.	T=4s d=0.7m	none
Exp7, mr28	d_{50} = 0.44	0.50	hor.	T=4s d=0.48m	none
Exp8, Flat Bed	d_{50} = 0.44	0.50	hor.	T=10s d=2m	none
Exp8, mr4c	d_{50} = 0.44	0.50	hor.	T=5s d=0.99m	none
Exp9, Flat Bed	d_{50} = 0.44	0.50	hor.	T=4.11s d=0.66m	none
Exp9, mr10b	d_{50} = 0.44	0.50	hor.	T=5s d=0.99m	none
Exp10, Flat Bed	d_{50} = 0.44	0.50	hor.	T=4.11s d=0.66m	none
Exp10, mr4	d_{50} = 0.44	0.50	hor.	T=7.38s d=1.33m	none
Exp11, Flat Bed	d_{50} = 0.44	0.50	hor.	T=5s d=0.99m	none
Exp11, mr4	d_{50} = 0.44	0.50	hor.	T=7.38s d=1.33m	none
Exp12, Flat Bed	d_{50} = 0.44	0.50	hor.	T=10s d=2m	none
Exp12, mr5b	d_{50} = 0.44	0.50	hor.	T=7.38s d=1.33m	none
Exp13, Flat Bed	d_{50} = 0.44	0.50	hor.		none
Exp13, mr10b		0.50			
Available data and parameters: -time series of bed profiles as bedforms grow from flat bed -bedform geometry -bedform migration rates					

Data of laboratory experiments on ripples and sand transport in a wave tunnel; University of Aberdeen, UK

Data Source	Sand Size (mm)	Water depth (m)	Slope	Waves	Current
Sandpit Experiments in the Large Oscillating Water Tunnel at WL|Delft Hydraulics.				2^{nd} order Stokes waves	
Experimental programme Series T:					
Test T4-05	D_{50} = 0.35 mm	0.80	hor.	T = 5,0 s, u_{rms}= 0.40 m/s	none
Test T5-05				T = 5,0 s, u_{rms} = 0.50 m/s	
Test T6-05				T = 5,0 s, u_{rms} = 0.60 m/s	
Test T3-07				T = 7.5 s, u_{rms} = 0.30 m/s	
Test T4-07				T = 7.5 s, u_{rms} = 0.40 m/s	
Test T5-07				T = 7.5 s, u_{rms} = 0.50 m/s	
Test T6-07				T = 7.5 s, u_{rms} = 0.60 m/s	
Test T4-10				T=10.0 s, u_{rms} = 0.40 m/s	
Test T5-10				T=10.0 s, u_{rms} = 0.50 m/s	
Test T6-10				T=10.0 s, u_{rms} = 0.60 m/s	
Available data and parameters: - Bed geometry. - Net sand transport rates. - Time-averaged concentrations at 10 elevations (from a few cm to about 0.3 m) above the ripple crest and trough (Transverse Suction System). - Time-dependent concentrations at two elevations above the ripple crest and trough (Carousel Suction System). (data niet geprocessed, dus weglaten) - Ensemble-averaged velocities (EMF) at two elevations above the sand bed.					

Data Source	Sand Size (mm)	Water depth (m)	Slope	Waves	Current
Sandpit Experiments in the Large Oscillating Water Tunnel at WL|Delft Hydraulics.				JONSWAP waves (irregular, asymmetric)	
Experimental programme Series U:					
Test U36-W	D_{50} = 0.35 mm	0.80	hor.	T_p = 6.5 s, u_{rms} = 0.36 m/s	none
Test U36-WC				T_p = 6.5 s, u_{rms} = 0.36 m/s	+0.18 m/s
Test U44-W				T_p = 6.5 s, u_{rms} = 0.44 m/s	none
Available data and parameters: - Bed geometry. - Net sand transport rates. - Time-averaged concentrations at 10 elevations (from a few cm to about 0.3 m) above the ripple crest and trough (ABS, Transverse Suction System). - Time-dependent concentration profiles (ABS). (data niet geprocessed, dus weglaten) - Ensemble-averaged velocities (EMF) at two elevations above the sand bed. (kun je wel ensemble middelen voor onregelmatige golven?)					

Data Source	Sand Size (mm)	Water depth (m)	Slope	Waves	Current
Sandpit Experiments in the Large Oscillating Water Tunnel at WL|Delft Hydraulics.				V25 and V34 Noorwijk field site (nearly symmetric), V38 Egmond field site (asymmetric)	
Experimental programme Series V:					
Test V25-W	D_{50} = 0.22 mm	0.80	hor.	T_p = 10.2 s, u_{rms} = 0.25 m/s	none
Test V25-WC				T_p = 10.2 s, u_{rms} = 0.25 m/s	+0.20 m/s
Test V34-W					none
Test V34-WC2					

Test V34-WC4 Test V38-W **Available data and parameters:** - Bed geometry. - Net sand transport rates. - Time-averaged concentrations at 10 elevations (from a few cm to about 0.3 m) (ABS, Transverse Suction System). - Ensemble-averaged velocities (EMF) at two elevations above the sand bed. (kun je wel ensemble middelen voor onregelmatige golven?)				T_p = 10.2 s, u_{rms} = 0.34 m/s T_p = 10.2 s, u_{rms} = 0.34 m/s T_p = 10.2 s, u_{rms} = 0.34 m/s T_p = 9.7 s, u_{rms} = 0.38 m/s	+0.20 m/s +0.45 m/s none

Data of laboratory experiments on ripples and sand transport in a wave tunnel; University of Twente, NL

Data Source	Sand size (mm)	Water depth (m)	Slope	Waves	Current
		Flow depth in wave tank		Regular waves	"Uniform" current
University of Catania Wave Plus Current SandPit Experiments					
SET 1 Preliminary assessment of the the suitable experimental set-up configuration, localization of a 1m^2 study area were the current characteristics can be considered homogeneous and testing of the experimental repeatability					
Experimental programme:	D_{50} =0.25 mm				
T1		0.30	hor.	-	U_c=0.029 m/s
T2		0.30	hor.	-	U_c=0.133 m/s
T3		0.30	hor.	-	U_c=0.140 m/s
T4		0.30	hor.	-	U_c=0.131 m/s
T5		0.30	hor.	-	U_c=0.131 m/s
T6		0.30	hor.	-	U_c=0.129 m/s
T7		0.30	hor.	-	U_c=0.146 m/s
T8		0.30	hor.	-	U_c=0.021 m/s
T9		0.30	hor.	-	U_c=0.044 m/s
T10		0.30	hor.	T=1.3 s; H=0.09 m	U_c=0.129 m/s
T11		0.30	hor.	-	U_c=0.141 m/s
T12		0.30	hor.	-	U_c=0.134 m/s
T13		0.30	hor.	T=1.0 s; H=0.14 m	U_c=0.134 m/s
Available data and parameters: - 3D Velocity profiles both above and within the boundary layer (ADV) in several points of the study area - Surface elevation time series (only wave plus current experiments) - Bottom roughness (ks) for all the experiments					
SET 4 Investigation of the spatial behaviour of the wave-current interactions within the basin					
M1	D_{50} =0.25 mm				
M2		0.30	hor.	-	U_c=0.049 m/s
		0.30	hor.	T=1.0 s; H=0.09 m	U_c=0.049 m/s
Available data and parameters: - 3D Velocity profiles both above and within the					

boundary layer (ADV) over the entire area interested by the presence of the current - Surface elevation time series (only wave and wave plus current experiments) - Bottom roughness (ks) for all the experiments					
SET 2 Analysis of several current and wave conditions over an horizontal fixed bed					
Experimental programme:					
P62	D_{50} =0.25 mm				
P63		0.30	hor.	-	U_c=0.071 m/s
P64		0.30	hor.	T=1.0 s; H=0.10 m	U_c=0.071 m/s
P65		0.30	hor.	T=0.8 s; H=0.11 m	U_c=0.071 m/s
P66		0.30	hor.	T=0.8 s; H=0.07 m	U_c=0.071 m/s
		0.30	hor.	T=2.0 s; H=0.07 m	U_c=0.071 m/s
P67					
P68		0.30	hor.	-	U_c=0.047 m/s
P69		0.30	hor.	T=1.0 s; H=0.10 m	U_c=0.047 m/s
P70		0.30	hor.	T=0.8 s; H=0.10 m	U_c=0.047 m/s
P71		0.30	hor.	T=0.8 s; H=0.07 m	U_c=0.047 m/s
P72		0.30	hor.	T=2.0 s; H=0.06 m	U_c=0.047 m/s
		0.30	hor.	T=0.8 s; H=0.06 m	U_c=0.047 m/s
P73					
P74		0.30	hor.	-	U_c=0.098 m/s
P75		0.30	hor.	T=1.0 s; H=0.12 m	U_c=0.098 m/s
P76		0.30	hor.	T=0.8 s; H=0.13 m	U_c=0.098 m/s
		0.30	hor.	T=0.8 s; H=0.06 m	U_c=0.098 m/s
P77					
P78		0.30	hor.	-	U_c=0.027 m/s
P79		0.30	hor.	T=1.0 s; H=0.09 m	U_c=0.027 m/s
P80		0.30	hor.	T=1.0 s; H=0.13 m	U_c=0.027 m/s
P81		0.30	hor.	T=0.8 s; H=0.12 m	U_c=0.027 m/s
P82		0.30	hor.	T=0.8 s; H=0.06 m	U_c=0.027 m/s
P83		0.30	hor.	T=0.7 s; H=0.07 m	U_c=0.027 m/s
		0.30	hor.	T=0.9 s; H=0.09 m	U_c=0.027 m/s
Available data and parameters: - 3D Velocity profiles both above and within the boundary layer (ADV) over the center of the study area - Surface elevation time series (only wave plus current experiments) - Bottom roughness (ks) for all the experiments					
SET 3 Comparison of the roughness effects (small roughness and large roughness)					
Experimental programme:					
S1					
S2	D_{50} =0.25 mm				
S3		0.30	hor.	-	U_c=0.044 m/s
S4		0.30	hor.	T=0.8 s; H=0.08 m	-
S5		0.30	hor.	T=1.0 s; H=0.09 m	-
S6		0.30	hor.	T=1.2 s; H=0.08 m	-
S7		0.30	hor.	T=1.4 s; H=0.09 m	-
S8		0.30	hor.	T=1.6 s; H=0.08 m	-
S9		0.30	hor.	T=0.8 s; H=0.10 m	-
S10		0.30	hor.	T=1.0 s; H=0.10 m	-
S11		0.30	hor.	T=1.2 s; H=0.10 m	-
S12		0.30	hor.	T=1.4 s; H=0.10 m	-
S13		0.30	hor.	T=0.8 s; H=0.08 m	U_c=0.044 m/s

S14		0.30	hor.	T=1.0 s; H=0.09 m	U_c=0.044 m/s
S15		0.30	hor.	T=1.2 s; H=0.08 m	U_c=0.044 m/s
S16		0.30	hor.	T=1.4 s; H=0.08 m	U_c=0.044 m/s
S17		0.30	hor.	T=1.6 s; H=0.08 m	U_c=0.044 m/s
S18		0.30	hor.	T=0.8 s; H=0.10 m	U_c=0.044 m/s
S19		0.30	hor.	T=1.0 s; H=0.11 m	U_c=0.044 m/s
		0.30	hor.	T=1.2 s; H=0.10 m	U_c=0.044 m/s
L1	D_{50} =30 mm	0.30	hor.	T=1.4 s; H=0.10 m	U_c=0.044 m/s
L2					
L3		0.30	hor.	-	U_c=0.047 m/s
L4		0.30	hor.	T=0.8 s; H=0.08 m	-
L5		0.30	hor.	T=1.0 s; H=0.08 m	-
L6		0.30	hor.	T=1.2 s; H=0.08 m	-
L7		0.30	hor.	T=1.4 s; H=0.08 m	-
L8		0.30	hor.	T=1.6 s; H=0.09 m	-
L9		0.30	hor.	T=0.8 s; H=0.11 m	-
L10		0.30	hor.	T=1.0 s; H=0.11 m	-
L11		0.30	hor.	T=1.2 s; H=0.10 m	-
L12		0.30	hor.	T=1.4 s; H=0.10 m	-
L13		0.30	hor.	T=0.8 s; H=0.08 m	U_c=0.047 m/s
L14		0.30	hor.	T=1.0 s; H=0.09 m	U_c=0.047 m/s
L15		0.30	hor.	T=1.2 s; H=0.09 m	U_c=0.047 m/s
L16		0.30	hor.	T=1.4 s; H=0.09 m	U_c=0.047 m/s
L17		0.30	hor.	T=1.6 s; H=0.08 m	U_c=0.047 m/s
L18		0.30	hor.	T=0.8 s; H=0.10 m	U_c=0.047 m/s
L19		0.30	hor.	T=1.0 s; H=0.11 m	U_c=0.047 m/s
L20		0.30	hor.	T=1.2 s; H=0.10 m	U_c=0.047 m/s
		0.30	hor.	T=1.4 s; H=0.10 m	U_c=0.047 m/s
Available data and parameters: - 3D Velocity profiles both above and within the boundary layer (ADV) over the center of the study area - Micropropeller velocity measurements between crest and trough. - Surface elevation time series (only wave and wave plus current experiments) - Bottom roughness (ks) for all the experiments		0.30	hor.	T=1.6 s; H=0.10 m	U_c=0.047 m/s

Data of laboratory experiments on hydraulic roughness of ripples in a wave-current basin; University of Catania, Italy

ANNEX 2B: NEW FIELD DATA

Data Source	Sand Size (mm)	Water depth (m)	Slope	Waves	Current
Experimental programme: **Ship-based measurements** -velocity profiles from sailing ship (ADCP) in cross-shore profile - bed soundings (bed forms) from sailing ship in cross-shore profile - sediment composition in cross-shore profile - stratigraphic data (box core samples) in cross-shore profile - density and temperature in cross-shore profile - bed load transport and susp. transport from moored ship (tidal currents) Tripod measurements - bed forms. - velocity and concentration profiles - susp. transport - pressure recordings (water level and wave heights)	d_{50} = 0.21 mm	10 to 20 m	hor.	Irregular waves up to 5 m	Peak tidal currents=0.7 m/s

Data of field experiments of Spring and Autumn 2003 campaigns at Noordwijk site in North Sea (Netherlands), University of Utrecht, The Netherlands

ANNEX 3

ICES GUIDELINES FOR THE MANAGEMENT

OF

MARINE SEDIMENT EXTRACTION

**(International Council for the Exploration of the Sea ICES,
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ANNEX 3: ICES GUIDELINES FOR THE [1]MANAGEMENT OF MARINE SEDIMENT EXTRACTION

Introduction

In many countries sand and gravel[2] dredged from the seabed makes an important contribution to the national demand for aggregates, directly replacing materials extracted from land-based sources. This reduces the pressure to work land of agricultural importance or environmental and hydrological value, and where materials can be landed close to the point of use, there can be additional benefits of avoiding long distance over-land transport. Marine dredged sand and gravel is also increasingly used in flood and coastal defence, and land reclamation schemes. For beach replenishment, marine materials are usually preferred from an amenity point of view, and are generally considered to be the most appropriate economically, technically and environmentally.

However, these benefits need to be balanced against the potential negative impacts of aggregate dredging. Aggregate dredging activity if not carefully controlled can cause significant damage to the seabed and its associated biota, to commercial fisheries and to the adjacent coastlines, as well as creating conflict with other users of the sea. In addition, current knowledge of the resource indicates that while there are extensive supplies of some types of marine sand, there appear to be more limited resources of gravel suitable, for example to meet current concrete specifications and for beach nourishment.

Against the background of utilising a finite resource, with the associated environmental impacts, it is recommended that regulators develop and work within a strategic framework which provides a system for examining and reconciling the conflicting claims on land and at sea. Decisions on individual applications can then be made within the context of the strategic framework.

General principles for the sustainable management of all mineral resources overall include:

- conserving minerals as far as possible, whilst ensuring that there are adequate supplies to meet the demands of society;
- encouraging their efficient use (and where appropriate re-use), minimising wastage and avoiding the use of higher quality materials where lower grade materials would suffice;
- ensuring that methods of extraction minimise the adverse effects on the environment, and preserve the overall quality of the environment once extraction has ceased;
- protecting sensitive areas and industries, including fisheries, important habitats (such as marine conservation areas) and the interests of other legitimate users of the sea; and
- preventing unnecessary sterilisation of mineral resources by other forms of development.

The implementation of these principles requires a knowledge of the resource, an understanding of the potential impacts of its extraction and of the extent to which rehabilitation of the seabed is likely to take place. The production of an Environmental Statement, developed along the lines suggested below, should provide a basis for determining the potential effects and identifying possible mitigating measures. There will be cases where the environment is too sensitive to disturbance to justify the extraction of aggregate, and unless the environmental and coastal issues can be satisfactorily resolved, extraction should not normally be allowed.

It should also be recognised that improvements in technology may enable exploitation of marine resources from areas of the seabed which are not currently considered as reserves, while development of technical specifications for concrete etc. may in the future enable lower quality materials to be used for a wider range of applications. In the shorter term, continuation of programmes of resource mapping may also identify additional sources of coarser aggregates.

[1] These guidelines do not relate to navigational dredging (i.e. maintenance or capital dredging)
[2] It is recognized that other materials are also extracted from the seabed, such as stone shell and maerl, and similar considerations should apply to them.

Scope

It is recognised that sand and gravel extraction if undertaken in an inappropriate way may cause significant harm to the marine and coastal environment. There are a number of international and regional initiatives that should be taken into account when developing national frameworks and guidelines. These include the Convention on Biological Diversity (CBD), EU Directives (particularly those on birds, EIA and habitats) and other regional conventions/agreements in particular the OSPAR and Helsinki Conventions, and initiatives pursued under them. This subject for example has recently been included in the Action Plan for Annex V to the 1992 OSPAR Convention on the 'Protection and Conservation of the Ecosystems and Biological Diversity of the Maritime Area' as a human activity requiring assessment.

Administrative framework

It is recommended that countries have an appropriate framework for the management of sand and gravel extraction and that they define and implement their own administrative framework with due regard to these guidelines. There should be a designated regulatory authority to:

- issue authorisation having fully considered the potential environmental effects,
- be responsible for compliance monitoring, and
- the framework for monitoring, and
- enforcing conditions

Environmental impact assessment

The extraction of sand and gravel from the seabed can have significant physical and biological effects on the marine and coastal environment. The significance and extent of the environmental effects will depend upon a range of factors including the location of the extraction area, the nature of the surface and underlying sediment, coastal processes, the design, method, rate, amount and intensity of extraction, and the sensitivity of habitats, fisheries and other uses in the locality. These factors are considered in more detail below. Particular consideration should be given to sites designated under international, European, national and local legislation, in order to avoid unacceptable disturbance or deterioration of these areas for the habitats, species and other designated features.

To enable the organisation(s) responsible for authorising extraction to evaluate the nature and scale of the effects and to decide whether a proposal can proceed it is necessary that an adequate assessment of the environmental effects be carried out. It is important for example, to determine whether the application is likely to have an effect on the coastline, or have potential impact on fisheries and the marine environment.

The Baltic Marine Environment Protection Commission – (Helsinki Commission) adopted HELCOM Recommendation 19/1 on 26 March 1998. This recommends to the Governments of Contracting Parties that an EIA should be undertaken in all cases before a extraction is authorised. For EU member states, the extraction of minerals from the seabed falls within Annex II of the "Directive on the Assessment of the Effects of Certain Public and Private Projects on the Environment" (85/337/EEC)[3]. As an Annex II activity, an EIA is required if the Member State takes the view that one is necessary. It is at the discretion of the individual Member States to define the criteria and/or threshold values that need to be met to require an EIA. The Directive was amended in March 1997 by Directive 97/11/EC. Member States are obliged to transpose the requirements of the Directive into national legislation by March 1999.

It is recommended that the approach adopted within the EU is followed. Member States should therefore set their own thresholds for deciding whether and when an EIA is required.

Where an EIA is considered appropriate the level of detail required to identify the potential impacts on the environment should be carefully considered and identified on a site-specific basis. An EIA should normally be prepared for each extraction area, but in cases where multiple operations in the same area are proposed, a single impact assessment for the whole area may be more appropriate, which takes account of the potential for any

[3] EIA Directive

cumulative impacts. In such cases, consideration should be given to the need for a strategic environmental assessment.

Consultation is central to the EIA process. The framework for the content of the EIA should be established by early consultation with the regulatory authority, statutory consultees, and other interested parties. Where there are potential transboundary issues it will be important to undertake consultation with the other countries likely to be affected, and the relevant Competent Authorities are encouraged to establish procedures for effective communication.

As a general guide, it is likely that the following topics considered below will need to be addressed.

Description of the physical setting

The proposed extraction area should be identified by geographical location, and described in terms of:

- the bathymetry and topography of the general area
- the distance from the nearest coastlines
- the geological history of the deposit
- the source of the material
- type of material
- sediment particle size distribution
- extent and volume of the deposit
- the stability and/or natural mobility of the deposit
- thickness of the deposit and evenness over the proposed extraction area
- the nature of the underlying deposit, and any overburden
- local hydrography including tidal and residual water movements
- wind and wave characteristics
- average number of storm days per year
- estimate of bed-load sediment transport [quantity, grain size, direction]
- topography of the seabed, including occurrence of bedforms
- existence of contaminated sediment and their chemical characteristics
- natural [background] suspended sediment load under both tidal currents and wave action

Description of the biological setting

The biological setting of the proposed extraction site and adjacent areas should be described in terms of

- the flora and fauna within the area likely to be affected by aggregate dredging , (e.g. pelagic and benthic community structure) taking into account temporal and spatial variability;
- information on the fishery and shell fishery resources including spawning areas with particular regard to benthic spawning fish, nursery areas, over-wintering grounds for ovigerous crustaceans and known routes of migration;
- trophic relationships, (e.g. between the benthos and demersal fish populations by stomach content investigations)
- presence of any areas of special scientific or biological interest in or adjacent to the proposed extraction area, such as sites designated under local, national or international regulations (e.g. Ramsar sites, the UNEP 'Man and the Biosphere' Reserves, World Heritage sites, Marine Protection Areas (MPA's) Marine Nature Reserves, Special Protection Areas (Birds Directive) or the Special Areas of Conservation (Habitats Directive);

Description of the proposed aggregate dredging activity

The assessment should include, where appropriate, information on:

- the total volume to be extracted
- proposed maximum annual extraction rates and dredging intensity

- expected lifetime of the resource and proposed duration of aggregate dredging
- aggregate dredging equipment to be used
- spatial design and configuration of aggregate dredging (i.e. the maximum depth of deposit removal, the shape and area of resulting depression)
- substrate composition on cessation of aggregate dredging
- proposals to phase (zone) operations
- whether on-board screening (i.e. rejection of fine or coarse fractions) will be carried out
- number of dredgers operating at a time
- routes to be taken by aggregate dredgers to and from the proposed extraction area
- time required for aggregate dredgers to complete loading
- number of days per year on which aggregate dredging will occur
- whether aggregate dredging will be restricted to particular times of the year or parts of the tidal cycle
- direction of aggregate dredging (e.g. with or across tide)

It may be appropriate when known to also include details of the following:

- energy consumption and gaseous emissions
- ports for landing materials
- servicing ports
- on-shore processing and onward movement
- project related employment

Information required for physical impact assessment

To assess the physical impacts the following should be considered:

- implications of extraction for coastal and offshore processes, including possible effects on beach draw down, changes to sediment supply and transport pathways, changes to wave and tidal climate
- changes to the seabed topography and sediment type
- exposure of different substrates
- changes to the behaviour of bedforms within the extraction and adjacent areas
- potential risk of release of contaminants by aggregate dredging, and exposure of potentially toxic natural substances
- transport and settlement of fine sediment disturbed by the aggregate dredging equipment on the seabed, and from hopper overflow or on-board processing and its impact on normal and maximum suspended load
- the effects on water quality mainly through increases in the amount of fine material in suspension
- implications for local water circulation resulting from removal or creation of topographic features on the seabed
- timescale for potential physical 'recovery' of the seabed

Information required for biological impact assessment

To assess the biological impact the following information should be considered:

- changes to the benthic community structure
- effects of aggregate dredging on pelagic biota
- effects on the fishery and shellfishery resources including spawning areas with particular regard to benthic spawning fish, nursery areas, overwintering grounds for ovigerous crustaceans and known routes of migration
- effects on trophic relationships (e.g. between the benthos and demersal fish populations)
- effects on sites designated under local, national or international regulations (see above)
- predicted rate and mode of recolonisation, taking into account initial community structure, natural temporal changes, local hydrodynamics and any predicted change of sediment type
- effects on marine flora and fauna including seabirds and mammals
- effects on the ecology of boulder fields/stone reefs

Interference with other legitimate uses of the sea

The assessment should consider the following in relation to the proposed programme of extraction:

- commercial fisheries
- shipping and navigation lanes
- military exclusion zones;
- offshore oil and gas activities;
- engineering uses of the seabed (e.g. adjacent extraction activities, undersea cables and pipelines including associated safety and exclusion zones)
- areas designated for the disposal of dredged or other materials;
- location in relation to existing or proposed aggregate extraction areas;
- location of wrecks and war-graves in the area and general vicinity.
- wind farms
- areas of heritage, nature conservation, archaeological and geological importance
- recreational uses
- general planning policies for the area (international, national and local)
- any other legitimate use of the sea

Evaluation of impacts

When evaluating the overall impact, it is necessary to identify and quantify the marine and coastal environmental consequences of the proposal. The EIA should evaluate the extent to which the proposed extraction operation is likely to affect other interests of acknowledged importance. Consideration should also be given to the assessment of the potential for cumulative impacts on the marine environment. In this context cumulative impacts might occur as a result of aggregate dredging at a single site over time, from multiple sites in close proximity or in combination with effects from other human activities (e.g. fishing and disposal of habour dredgings).

It is recommended that a risk assessment be undertaken. This should include consideration of worst case scenarios, and indicate uncertainties and assumptions used in their evaluation.

The environmental consequences should be summarised as an impact hypothesis. The assessment of some of the potential impacts requires predictive techniques, and it will be necessary to use appropriate mathematical models. Where such models are used, there should be sufficient explanation of the nature of the model, including its data requirements, its limitations and any assumptions made in the calculations, to enable assessment of its suitability for the particular modelling exercise.

Mitigation Measures

The impact hypothesis should include consideration of the steps that might be taken to mitigate the effects of extraction activities. These may include:

- the selection of aggregate dredging equipment and timing of aggregate dredging operations to limit impact upon the biota (such as birds, benthic communities and fish resources)
- modification of the depth and design of aggregate dredging operations to limit changes to hydrodynamics and sediment transport and to minimise the effects on fishing
- spatial and temporal zoning of the area to be authorised for extraction or scheduling extraction to protect sensitive fisheries or to respect access to traditional fisheries
- preventing on-board screening or minimising material passing through spillways when outside the dredging area to reduce the spread of the sediment plume
- agreeing exclusion areas to provide refuges for important habitats or species, or other sensitive areas

Evaluation of the potential impacts of the aggregate dredging proposal, taking into account any mitigating measures, should enable a decision to be taken on whether or not the application should proceed. In some cases it will be appropriate to monitor certain effects as the aggregate dredging proceeds. The EIA should form the basis for the monitoring plan.

Authorisation Issue

When an aggregate extraction operation is approved, then an authorisation should be issued in advance (which may take the form of a permit, licence or other form of regulatory approval). In granting an authorisation, the immediate impact of aggregate extraction occurring within the boundaries of the extraction site such as alterations to the local physical, and biological environment is accepted by the regulatory authority. Notwithstanding these consequences, the conditions under which an authorisation for aggregate extraction is issued should be such that environmental change beyond the boundaries of the extraction site are as far below the limits of allowable environmental change as practicable. The operation should be authorised subject to conditions which further ensure that environmental disturbance and detriment are minimised.

The authorisation is an important tool for managing aggregate extraction and will contain the terms and conditions under which aggregate extraction may take place as well as provide a framework for assessing and ensuring compliance.

Authorisation conditions should be drafted in plain and unambiguous language and will be designed to ensure that:

a. the material is only extracted from within the selected extraction site;
c. any mitigation requirements are complied with; and
d. any monitoring requirements are fulfilled and the results reported to the regulatory authority.

Monitoring compliance with conditions attached to the authorisation

An essential requirement for the effective control of marine aggregate extraction is monitoring on a continuous basis of all aggregate dredging activity to provide a permanent record. This has been achieved in several ways e.g. an Electronic Monitoring System or Black Box. The information provided will allow the regulatory authority to monitor the activities of aggregate dredging vessels to ensure compliance with particular conditions in the authorisation.

The information collected and stored will depend on the requirements of the individual authorities and the regulatory regime under which the permission is granted e.g. EIA, Habitats, Birds Directives of the EU.
The minimum requirements for the monitoring system should include:

- an automatic record of the date, time and position of all aggregate dredging activity
- position to be recorded to within a minimum of 100 metres in latitude and longitude or other agreed co-ordinates using a satellite based navigation system
- there should be an appropriate level of security
- the frequency of recording of position should be appropriate to the status of the vessel i.e. less frequent records when the vessel is in harbour or in transit to the aggregate dredging area e.g. every 30 minutes and more frequent when dredging e.g. every 30 seconds.

The above are considered to be a reasonable minimum requirement to evaluate the regulatory authority to monitor the operation of the authorisation in accordance with any conditions attached. Individual countries may require additional information for compliance monitoring at their own discretion.

The records can also be used by the aggregate dredging company to improve utilisation of the resources. The information is also an essential input into the design and development of appropriate environmental monitoring programmes and research into the physical and biological effects of aggregate dredging including combined/cumulative impacts (see section above)

Environmental Monitoring

Sand and gravel extraction inevitably disturbs the marine environment. The extent of the disturbance and its environmental significance will depend on a number of factors. In many cases it will not be possible to predict, in full, the environmental effects at the outset, and a programme of monitoring may be needed to demonstrate the validity of the EIA's predictions, the effectiveness of any conditions imposed on the authorisation, and therefore the absence of unacceptable impacts on the marine environment.

The level of monitoring should depend on the relative importance and sensitivity of the surrounding area. Monitoring requirements should be site specific, and should be based, wherever possible, on the findings of the EIA. To be cost effective, monitoring programmes should have clearly defined objectives derived from the impact hypothesis developed during the EIA process. The results should be reviewed at regular intervals against the stated objectives, and the monitoring exercise should then be continued, revised, or even terminated.

It is also important that the baseline and subsequent monitoring surveys take account of natural variability. This can be achieved by comparing the physical and biological status of the areas of interest with suitable reference sites located away from the influence of the aggregate dredging effects, and of other anthropogenic disturbance. Suitable locations should be identified as part of the EIA's impact hypothesis.

A monitoring programme may include assessment of a number of effects. When developing the programme a number of questions should be addressed including:

- what are the environmental concerns that the monitoring programme seeks to address?
- what measurements are necessary to identify the significance of a particular effect?
- what are the most appropriate locations at which to take samples or observations for assessment?
- how many measurements are required to produce a statistically sound programme?
- what is the appropriate frequency and duration of monitoring?

The regulatory authority is encouraged to take account of relevant research information in the design and modification of monitoring programmes.

The spatial extent of sampling should take account of the area designated for extraction and areas outside which may be affected. In some cases, it may be appropriate to monitor more distant locations where there is some question about a predicted nil effect. The frequency and duration of monitoring may depend upon the scale of the extraction activities and the anticipated period of consequential environmental changes which may extend beyond the cessation of extraction activities.

Information gained from field monitoring (or related research studies) should be used to amend or revoke the authorisation, or refine the basis on which the aggregate extraction operation is assessed and managed. As information on the effects of marine aggregate dredging becomes more available and a better understanding of impacts is gained, it may be possible to revise the monitoring necessary. It is therefore in the interest of all concerned that monitoring data is made widely available. Reports should detail the measurements made, results obtained, their interpretation and how these data relate to the monitoring objectives.

Reporting Framework

It is recommended that the national statistics on aggregate dredging activity continue to be collated annually by ICES WGEXT.

Definitions

In these Guidelines "marine sediment extraction" is intended to refer to the extraction of marine sands and gravels (or "aggregates") from the seabed for use in the construction industry (where they often directly replace materials extracted from land-based sources), and for use in flood and coastal defence, beach replenishment and land reclamation projects. It is recognised that other materials are also extracted from the seabed, such as stone, shell materials and maerl, and similar considerations to those set out in the Guidelines should also apply to these. The Guidelines do not apply to navigational dredging (e.g. maintenance or capital dredging operations).

In these Guidelines the term "authorisation" is used in preference to "permit" or "licence" but is intended to replace both terms. The legal regime under which marine extraction operations are authorized and regulated differs from country to country, and the terms permit and licence may have a specific connotation within national legal regimes, and also under rules of international law. The term "authorisation" is thus used to mean any use of permits, licences or other forms of regulatory approval.

ANNEX 4

GLOSSARY OF PHYSICAL AND ENGINEERING TERMS USED IN THE SANDPIT PROJECT

ANNEX 4: GLOSSARY OF PHYSICAL AND ENGINEERING TERMS USED IN THE SANDPIT PROJECT

GENERAL TERMS

Bed roughness. The speed of current and wave flows over the sea bed is controlled by friction with the bed, which in turn depends on how rough the bed is; for example, flat gravel is rougher than flat sand, and *rippled* sand is rougher than flat sand. The bed roughness can be quantified in various inter-related ways: e.g. (a) the "Nikuradse roughness" is proportional to grain diameter or dimensions of bedforms; (b) the "bed roughness-length" can be obtained from the *velocity profile* above the bed.

Bedforms. Under different flow and sediment conditions the sea bed may be flat and featureless, or it may be covered with irregularities such as *ripples, hummocks* or *sandwaves.* The general name for any kind of irregularity of the bed is a bedform.

Bedload. A form of sediment transport in which the grains roll, slide or hop along very close to the sea bed in response to current or waves. Especially important for coarse sand and gravel.

Bed shear-stress. A measure of the frictional force exerted by a water flow on the sea bed, expressed as force exerted per unit area of bed (Newtons per square metre). Equally, it is the retarding force the sea bed exerts on the flowing water.

Benchmarking. Different researchers have developed different computational models for predicting *sediment transport* rates, or for *morphodynamic* evolution of the sea bed. The performance of models designed for the same purpose can be compared with each other by comparing their outputs for a specified set of inputs, usually for a case in which the inputs and outputs have been measured in a laboratory or field experiment. The accuracy of the models against the data can be intercompared, as well as the speed and complexity of the models. This is called a benchmark test, and the whole process is called benchmarking.

Entrainment. A strong wave or current flow can lift sand grains from the sea bed up into *suspension* in the water. This is called entrainment of sand.

Friction velocity. A mathematically-convenient quantity for expressing the friction of a flow with the sea bed in the units of velocity (metres per second). Related to the *bed shear-stress* (which is expressed in Newtons per square metre).

Hummock. A type of *bedform* comprising mounds and hollows of sediment arranged in a three-dimensional pattern with 1 to 20m spacing and heights of 0.05 to 0.2 m. Formed by *sheet flow.*

Morphodynamics. The evolution of the shape of a coastline or sea bed; for example, the change in the shape of a sand mining pit or a sandbank with time. Morphodynamic models are designed to simulate this bed evolution in response to the computed wave and current distribution.

Ripple. A type of small *bedform* often seen on a beach or in a river, like a small wave or ripple in the sand with its crests aligned perpendicular to the flow. Its height is a few cm or tens of cm, and spacing between crests is tens to hundreds of cm. Wave ripples generate vortices and thus strongly increase the suspension of sediment.

Sandwave. A type of large *bedform* like a wave with its crests aligned perpendicular to the water flow. Its height is between about 0.5m and several metres, and spacing between crests is tens or hundreds of metres. Moves slowly in the direction of strongest flow.

Sediment transport. The movement of sediment (mud, sand or gravel) by flowing water (currents and waves). Has two main modes in the sea, *bedload* transport and *suspended* transport.

Shear-stress. Strictly, the shear-force exerted between two "layers" of water sliding past each other, as in a *velocity profile*. Loosely, often used as a shorthand for the *bed shear-stress*, ie the shear-force exerted between the lowest "layer" of water and the bed.

Sheet flow. A form of *bedload* transport of sediment caused by very strong currents or large (often breaking) waves, in which any ripples are washed away leaving a flat bed covered in a layer of moving sand. Often associated with intense suspended load transport.

Shields curve. The most widely used method of predicting whether or not sand (or gravel) will be moved by flowing water. A non-dimensional form of the *bed shear-stress* is plotted against a non-dimensional form of the grain diameter as a threshold curve. Points lying above the curve correspond to moving sediment, while those below the curve cannot be moved by that bed shear-stress.

Significant wave height. The most widely used method of measuring the typical height of waves caused by wind at sea. Originally defined as the mean height of the highest one-third of all individual measured waves recorded over about 20 minutes at a particular location at a particular time. Nowadays usually calculated via a different method, but with much the same answer.

Suspension. Strong current or wave flows can carry sand grains (or mud) to heights well above the bed, when they are said to be suspended, or in suspension. The movement of the suspended sand with the current is known as suspended transport, and the mass of sediment suspended over an area of bed is known as the suspended load. Most important for fine and medium sands.

Velocity profile. The variation with height above the sea bed of the velocity due to a current or wave.

INSTRUMENTS AND TECHNIQUES

ABS. Acoustic Backscatter System, for measuring the concentration profile of suspended sand by measuring the intensity of reflected sound pulses.

ADCP. Acoustic Doppler Current Profiler for measuring flow velocity in all three orthogonal directions at several heights between water or bed surface and the sensor.

AOFT. The Aberdeen Oscillatory Flow Tunnel is a rectangular cross-sectional duct in which water flows can be made to oscillate to simulate wave-induced velocities near the seabed at full scale, designed by, and located at, the University of Aberdeen.

ASTM. Acoustic Sediment Transport Meter, for measuring both velocity and suspended sediment concentration using acoustic techniques at a number of points near the seabed.

Burst. A record of data measured on an instrument, usually measured at high frequency (several times a second) for a duration of about 10 to 20 minutes, intermittently (eg one burst per hour, or per 3 hours).

EMF. The Electromagnetic Flowmeter measures two components of water velocity by applying an electromagnet and sensing the small voltages produced.

LOWT. The Large Oscillating Water Tunnel is a rectangular cross-sectional duct in which water flows can be made to oscillate to simulate wave-induced velocities and currents near the seabed at full scale, designed by, and located at, Delft Hydraulics.

OBS. Optical Backscatter Sensor used for measuring suspended sediment concentrations by measuring the strength of scattering of visible light or infra-red (from an LED source) by the sediment. The OBS is much more sensitive to mud than to sand.

OPCON. The Optical Concentration meter senses the concentration of sand suspended in a flow by measuring the reduction in intensity of a beam of light passed through it.

PIV. Particle Image Velocimetry is a method for measuring the instantaneous velocity field over a 2-dimensional area of flow. The flow area is illuminated using a laser. A pair of images of the flow, separated in time by a few milliseconds, is recorded by digital camera. The 2-d velocity field is then obtained by cross-correlating the pair of images. Used in the *AOFT* SANDPIT experiments to obtain detailed velocity measurements of flow over rippled beds.

TSS. The Transverse Suction System pumps sediment-laden water through nozzles located near the bed of a sediment flume, and directed perpendicular to the flow. The water is filtered to obtain a direct measure of suspended sediment concentration. Sometimes used as a primary standard for calibrating indirect methods such as *ABS, ASTM, OBS* or *OPCON*.

UVP. The Ultrasonic Velocity Profiler (manufactured by METFLOW) measures instantaneous *velocity profiles* in liquid flow by detecting the Doppler shift frequency of echoed ultrasound as a function of time. Each profile consists of 128 simultaneous velocity measurements. Used in the *AOFT* SANDPIT experiments to measure velocities over rippled beds.