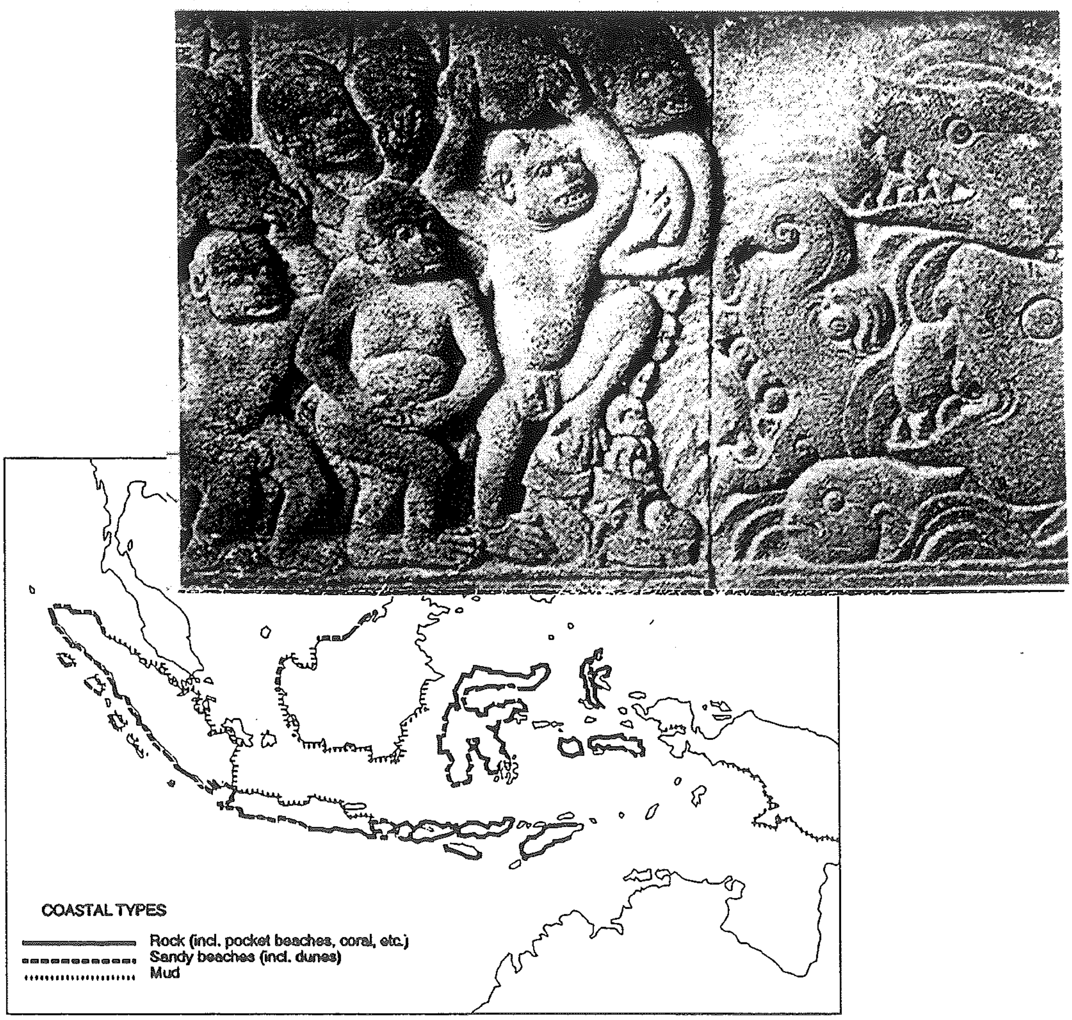
PEDOMAN TEKNIK PANTAI

(COASTAL ENGINEERING MANUAL)



PRELIMINARY VERSION

JUNE 1996

COMPILED IN THE FRAMEWORK OF THE MOU
BETWEEN THE MINISTRIES OF PUBLIC WORKS
OF INDONESIA AND THE NETHERLANDS

At last, the day of departure arrived. What a noble sight it was to see the seven enthousiastic armies marching in close columns to the south, with Hanuman in front as the guide. Their bright weapons glittered in the morning sun and the earth shook with the sound of these rapidly moving multitudes. But their long march came to an abrupt stop when they reached the coast, for the sea was wild with the roar of towering waves. To cross the sea-arm which separated Alengka from the mainland was totally out of the question. For a moment Hanuman stopped to think. Then a brilliant idea struck him. It became suddenly clear to him that 'gotong-royong' should be the answer. Was it not true that countless drops of water together makes a mighty ocean ? Therefore millions of fearless Waranas should be able to conquer this narrow sea-arm. A bridge could be built across it. Hanuman decided to set the example. He seized a piece of rock and plunged it into the raging waters. At this sign, the millions of apes followed suit and, as there were plenty of hard materials on the coast, they were able to perform their task without delay. The numberless apes succeeded in building up a mighty causeway across the sea by means of the stones, tree-trunks and other hard materials which lay at their disposal. They toiled without rest for seven long days and seven long nights, until at last their heroic project was complete. The bridge rose sound and strong above the battering waves. The sea had now been conquered and the way was clear for the seven armies to enter the kingdom of Alengka.

How Hanuman bridges the sea-arm, from RAMAYANA,
compiled by Sunardjo Haditjoroko,
Penerbit Djambatan, Jakarta 1975

Cover picture:
Sandstone relief at Shiva Mahadeva Temple, Prambanan Complex, Java

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Jakarta
june 20 1996

joint statement at the occasion of

the opening session of the seminar on

Development and Construction in Low Lying Areas

We, the minister of Public Works of Indonesia and the minister of Transport, Public Works and Watermanagement of The Netherlands have, with great pleasure, taken notice of the progress being made in the cooperation between the Indonesian Public Works Department and Rijkswaterstaat.

We consider the development of infrastructure and the sustainable exploitation of natural resources as one of the central focuses in any nation's efforts to improve its economy. This is true not only in a small, densely populated and highly developed country like The Netherlands, but even more so in a large and rapidly developing nation as Indonesia.

A recent example of this growing cooperation between both nations is the joint effort in developing a Coastal Engineering Manual. We are convinced that these efforts will be beneficial to the fast developing coastal zone in the Indonesian Archipelago.

We therefore recommend the Director General of Research and Development of the Ministry of Public Works of the Republic of Indonesia and the Director General of Rijkswaterstaat of the Ministry of Transport, Public Works and Watermanagement to complete the manual in a joint effort. A manual that can be considered as one of the milestones within the framework of cooperation and as a benchmark for the implementation of coastal policies in the Indonesia Archipelago.

Radinal Moochtar	*Annemarie Jorritsma*
Minister of Public Works of the Republic Indonesia	*Minster of Transport, Public Works and Watermanagement of The Netherlands*

PREFACE

AWAS !

You probably never have seen a book so incomplete and unfinished as this one. The reason is that this is only the beginning of what could become a practical and useful manual for coastal engineering problems in Indonesia. Most handbooks on coastal engineering are meant for coasts in regions with a temperate climate. The situation in Indonesia is different in many respects. Wind conditions differ, since heavy winds due to gales are practically non-existent in Indonesia, also lying outside the tropical cyclone areas. On the other hand, tsunamis occur throughout the archipelago. The sediment supply of rivers to the coast is abundant and the sediment is very fine, giving extensive muddy coasts. Together with the temperature, this creates favourable conditions for mangroves, being typical for many an Indonesian coast. Another typical feature for the area is the abundance of coral, which also deviates from the situation in other climates.

There is still a lot of work to be done before this manual can be completed. Many data and experiences have to come from Indonesia. In this preliminary version, sometimes (very) old data has been used. Probably much of it can or should be replaced by more actual numbers. Not always the same symbol is used for the same parameter, figures have to be redrawn or better copies have to be found. Parts are still lacking, other possibly superfluous or should be rewritten, since some parts come from lecture notes, either from IHE or DUT. Example cases and calculations have to be drawn up. And even more crucial, for some topics like mud coasts, protection by mangroves or coral, much knowledge does not yet exist and should be developed.

All this will have to be discussed with the future Indonesian users. The result so far is what we could produce in the limited amount of time and budget available. Completion will only be possible by means of gotong-royong. Last but not least, a translation into Bahasa Indonesia probably would make this pedoman more effective for use inside the country. Unfortunately we will not be able to offer much help for this task. Maaf.

Delft, June 1996

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CONTENTS

1 Introduction

1.1 Indonesian coasts

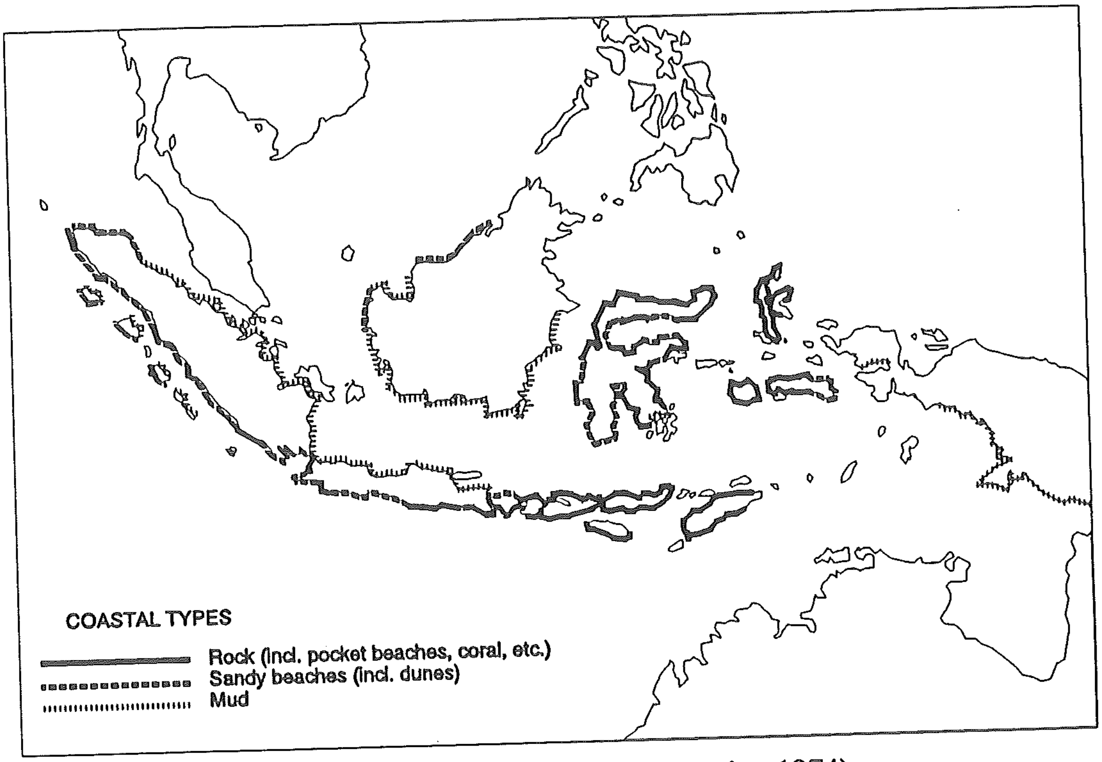

Figure 1.1 Coastlines of Indonesia (adapted from Consentius,1974)

The more than 13,000 islands in the Indonesian archipelago, together have a coastline with a length of more than 60,000 km. Although all of them are tropical island coasts, they differ from sandy beaches and mud flats with mangroves to coral reefs and steep rock cliffs. Figure 1.1 gives a rough and provisional overview of shoreline types in Indonesia (special features like mangroves or coral reefs will be shown separately later on). When designing engineering works, like coastal protections, land reclamations or harbours, different types of coast ask for a different approach. It is therefore necessary to have enough understanding of the behaviour of these different coastal types.

This manual is primarily meant for those dealing with Coastal Engineering. Most handbooks on coastal engineering are based on coasts in temperate climate zones. Many coastal processes show a high degree of similarity all over the world, but in the following will be shown that there are also large differences between coasts in a temperate or tropical climate. Large storms and cyclones, like in temperate or other tropical regions barely occur in Indonesia, making problems like flood disasters not so important. On the other hand, the seismically active region knows tsunamis and volcanic outbursts, changing sometimes the shoreline dramatically. And of course, coastal erosion is a problem, since the processes of sedimentation and erosion are dynamic in nature, while the coastal zone in Indonesia is intensively used, giving tension between natural processes and land use. This tension and the specific nature of coastal processes in Indonesia, justify this manual.

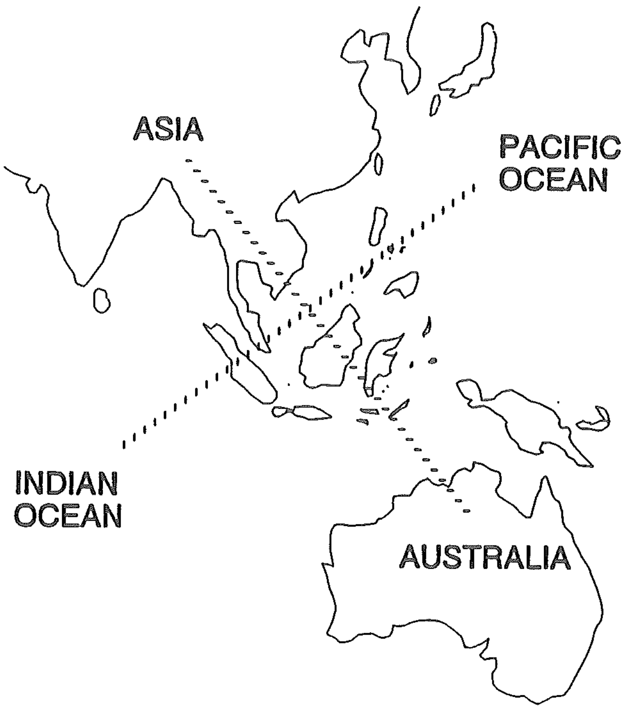

Figure 1.2 The Indonesian archipelago as crossroads of oceans and continents

Coastal forms and processes in Indonesia are highly influenced by the fact that the archipelago is a transition between two oceans: the Indian and Pacific Ocean and two continents: Eurasia and Australia, see Figure 1.2 with again two dominating processes: tectonics and climate.

The tectonic movements of the plates under the oceans have shaped the Indonesian archipelago, including coastal forms, to a great extent and still lead to intense seismic activities in the area. The transition between the two continents is demonstrated from geological findings. The large Sunda islands Sumatera, Java and Kalimantan clearly belong to the Eurasian continent, while Irian and the Aru islands are part of the Australian continent. Sulawesi, however, was "assembled" from Asian, Australian and Pacific parts, see Katili,198?.

The climate is tropical humid (Köppen type Af, except in the lesser Sunda islands, where there is a clearly dry season, Köppen type Aw), leading to large river discharges and sediment yields with fine sediments. These sediments play an important role in the coastal processes. The two continents also strongly influence the climate and accordingly the coastal shaping factors. In the Northern summer (July), the Chinese mainland is relatively warm and Australia cold, leading to relatively low and high air pressures respectively and a monsoon wind from the Southeast. In the Northern winter (January), the situation is the other way around, giving Northwesterly winds. These monsoon winds cause currents between the islands, distributing the sediments and waves, reshaping the coasts.

The two oceans finally, higly determine the pattern of tides in the archipelago. The tidal waves from the two oceans meet each other here and cause, together with the geometry of the islands and continental shelves, a very complicated and interesting pattern of tides. All these phenomena will be dealt with in chapter 3 and 4: Natural System.

1.2 Activities and problems

Being an archipelago-state, about 90% 99%? of all people in Indonesia live in what can be defined as the coastal zone. This leads to an intensive use of the coastal area, which also leads to various and often conflicting interests.

******** To be completed by Indonesian counterparts. ********

1.3 *Problem approach*

Engineering on a coast, whether it is land reclamation, coastal defence or the building of a harbour, means interfering in a natural process. This asks for adequate knowledge of these processes in relation to the purpose of the interference. In general, there should be a balance between knowledge and the choice to be made.

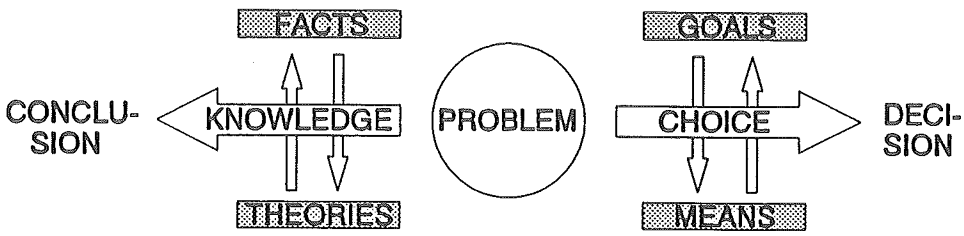

Figure 1.3 Divergent problem approaches

Figure 1.3 shows the two questions to which a problem can lead: ***How does that come?*** to the left and ***What should be done?*** to the right. Answering only the first question is scientifically interesting but does not give any solution. Answering only the second question leads to solutions that often do not work or have unexpected negative consequences like erosion somewhere else. A real solution should contain all elements of insight and purposiveness. The insight should be based on facts (data, measurements) and theories (understanding, proven model concepts). The purposiveness should be based on clear goals and adequate means. In order to reach a balanced decision, a systems approach is recommended, see chapter 2 (The coastal system)

Understanding of the processes is deemed essential in coastal engineering. But, many of these processes are not yet fully understood and a lot of research remains to be done. When quantitative modelling is not yet possible, often there is information on a descriptive level. A qualitative understanding is then much preferable above an advanced, but wrongly applied mathematical model. Therefore, chapter 3 and 4 (Natural System) will deal with coastal processes in a very wide sense, while chapter 8 (Interaction between coastal structures and natural system) treats this subject specifically. These chapters deal mainly with the left hand (knowledge) side of Figure 1.3. Physical theories describe the behaviour of coasts, supported by measurements (facts).

This knowledge is important, since building at or interfering along a sandy or muddy coast, always takes place in a dynamic flow of sediment. The questions where it comes from and where it goes, should always be posed and answered, albeit roughly and qualitatively. When the sediment comes directly from a river, decrease of sediment supply, e.g. by building a barrage upstream, will generally cause coastal erosion. The reverse is also possible, an increase of the sediment supply to Segara Anakan (increased erosion due to denudation?) has silted up the (protected) area between the original coastline and the islands in front of it, giving way to an increase of mangroves. When the sediment flow is along the coast, due to wave or wind driven currents, interrruption will cause accretion and erosion.

Chapter 5 (User function) gives the demands from society in coastal engineering. These form the goals in Figure 1.3. This manual is restricted to the domain of physical sciences and engineering, but often engineers are involved in making plans for coastal areas. The goals should be clear before any project is really being executed, which is sometimes not the case. An engineer might then come up with the right answer to the wrong question. Goals should be set preferably as part of a coastal zone management strategy. A Decision Support System (DSS) can help in grounding a decision, see e.g. van der Weide,1993 and the material in this seminar on Coastal Zone Management.

The chapters 6 through 10 (Infrastructure, Design approach, Interaction, Structural design and Construction and maintenance respectively) can then be seen as the engineering core within the framework of the coastal system. They form the means in Figure 1.3.

Only with a complete set of theories, facts, goals and means the engineer can do his job properly.

1.4 Glossary

******** In the final version of the Pedoman, a comprehensive list of topics, coastal elements and processes should be included. These two figures with definitions along the coastline can be useful as a start.

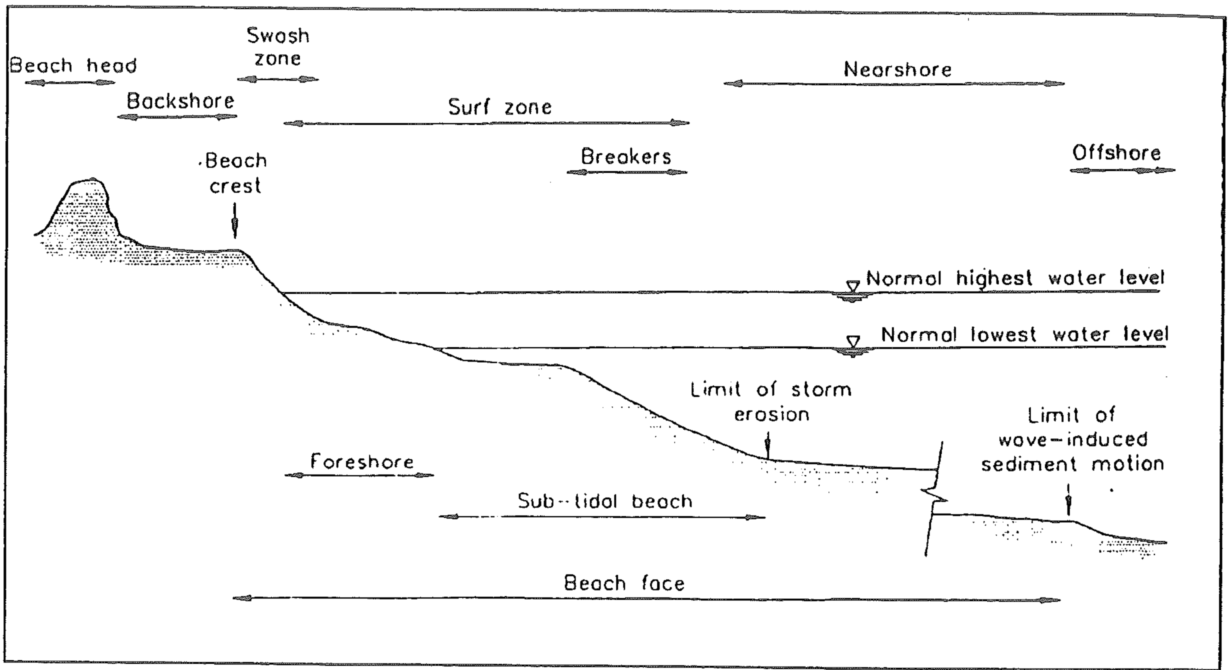

Figure 1.1: Definitions along the coastline [CIRIA, 1996]

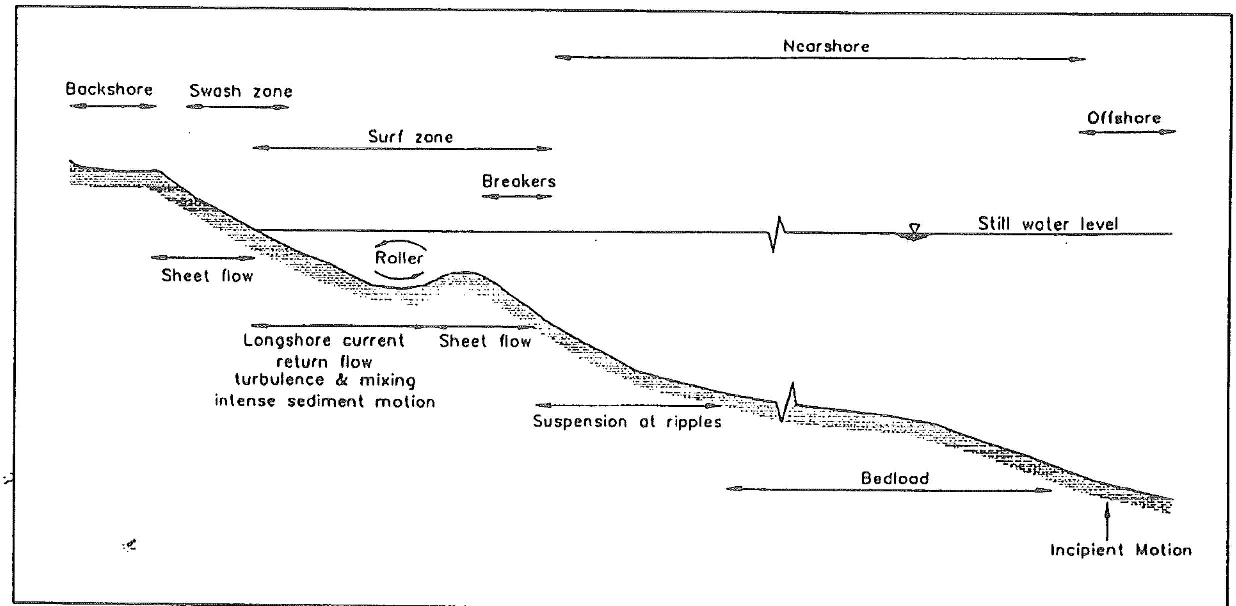

Figure 1.2: Hydrodynamic regions of the beach [CIRIA, 1996]

1.5 *References*

Consentius, W.U., Die küsten des Südöstlichen Asien, Ph. D. Thesis, Technical University Berlin, 1974

Katili, J.A. Review of past and present geotectonic concepts of Eastern Indonesia, Netherlands Journal of Sea Research, 24, 103-129, 1989

Bird, Coastline changes, 1985

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2 Coastal zone management

2.1 Systems approach

2.1.1 General

Although this manual focusses on coastline protection and coastal engineering, it is good to realize that all these activities are part of a much larger framework. The coastal zone is intensively used, certainly in the densily populated areas of Indonesia. At several locations, the Indonesian coasts suffer from erosion, as is the case with many coasts in the world. This erosion is partly due to natural causes, like subsidence or wave attack, and partly due to human activities in the coastal zone and in the watershed of rivers.

In many cases, causes and effects are obvious, like sand mining and coral blasting or civil engineering interventions, such as ports and jetties. Often, however, erosion is the ultimate effect of a chain of interrelated events, the origin of which may be obscure for the superficial observer. Degradation of deltas as a result of river regulation upstream, deterioration of mangrove due to changing hydrological conditions in the coastal lowlands or degradation of coastal reefs due to poor water quality are examples.

Mitigating measures should be tailored to the causes of erosion. Short term local measures will only be effective if the cause of erosion is local. If erosion is due to large scale and long term phenomena, civil engineering interventions should be part of a well planned strategy or even a Coastal Zone Management policy. Insight in the behaviour of coasts is paramount, it is often insufficient to monitor sequential variations without paying due regard to the causes of change. There are several good reasons for this (see also Carter,1988):

- By exploring the root cause it is possible to provide a better basis for understanding, thus allowing more confident prediction
- The results of such an examination are likely to be more universally applicable
- Reliance only upon the indicators of response may understate the overall problem

Coastal problems usually cannot be treated narrowly as mere technical problems. But it is impossible for an individual to cover all topics, ranging from water quality and ecology, economy and sociology to geology and engineering. Many disciplines being of importance, a common methodolgy for all persons involved is necessary, in order to be able, at least, to communicate. Systems analysis has been proved to be an appropriate tool to structure the interaction between the natural resource system and the socio-economic user system as a basis for a multi-disciplinary approach see also van der Weide,1993.

The contents of this manual follow the systems description as outlined in the next section. The manual being limited to Coastal Engineering approach and activities, thus fits in the framework of Coastal Zone Management. The idea is that communication with other disciplines and users of the coast will be easier, avoiding misunderstanding as much as possible.

2.1.2 *Systems analysis*

A system is an abstraction of the real world. Here the following definition is applied:

A system is a schematisation of reality by means of a set of elements and their interactions.

System elements are the basic entities of the system, containing all properties relevant to the problem to be adressed. Interactions are defined as relationships between these elements. They can be abstract, such as the flow of information between groups in society, or concrete, for instance the flow of water and sediment from a river to the coast. Interactions that change the system elements are called processes. The considered system contains only part of all the elements of the real world. Therefore, system boundaries are needed with input and output to describe the interaction with the systems environment.

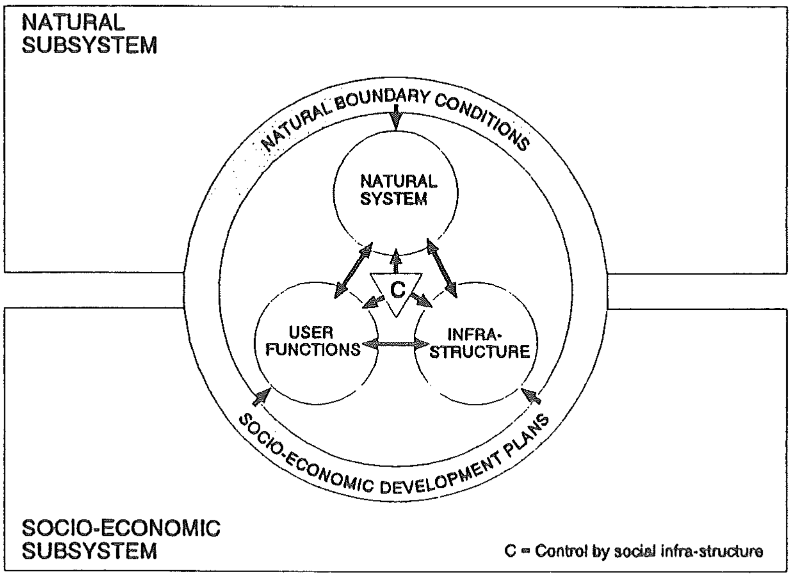

Figure 2.1 Coastal system

Figure 2.1 gives a systems view of the coastal zone (from van der Weide,1993). The Natural Subsystem imposes natural boundary conditions (i.e. everything else but human activities), while development plans come from the Socio-Economic Subsystem (i.e. the form in which the active human driving forces come to work). On a more detailed level, we distinguish three system elements. The *natural system* encompasses atmosphere, lithosphere and hydrosphere with their mutual interactions through A-biotic, Biotic and Chemical processes. The total complex of human activities is split up into two entities. The *user functions* represent the uses which are made of the natural resources. The *infrastructure* represents technical and organisational means to materialise the user functions. Between these elements there are all kinds of interactions. The triangle in the center represents Coastal Zone Management, where information comes together and where strategies are prepared, based on a coherent knowledge. The main task of engineers is to plan and design the required physical infrastructure.

2.1.3 *Stepwise approach*

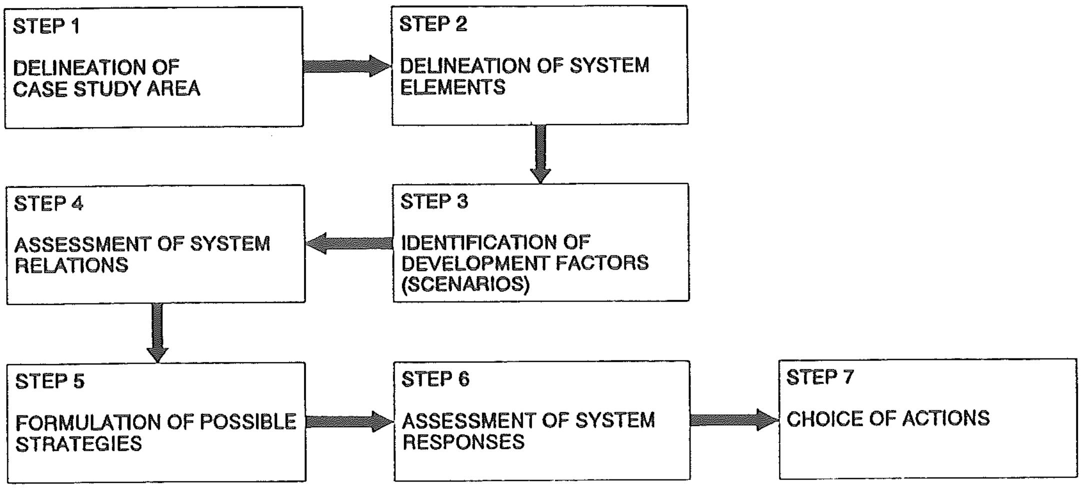

Figure 2.2 Stepwise approach of CZM problem

Now we will try to indicate the steps that have to be taken in the planning process. The steps can be described as follows, see Figure 2.2:

1 **Delineation of case study area**
The limits of the area to be studied are determined, both geographically and socio-economically. This is the outer circle in the systems diagram. The relevant factors from the subsystems are described from available field data and macro-economic data.

2 **Delineation of system elements**
Databases for the elements within the area to be studied are described from available or newly derived material. These are the inner circles in the system diagram.

3 **Identification of development factors (scenarios)**
An inventory is made of relevant processes and plans, both from the natural and the socio-economic subsystem. These are the arrows from the outer circle in the system diagram to the system elements. They can be seen as the agents of change in the system elements. These agents can both be demand driven (from the socio-economic subsystem) or driven by natural processes.

4 **Assessment of system relations**
A model is made of the relations between the various elements of the system. In this model, the effect of changes in a system element on the other elements is described. This can be done in a matrix of possible conflicts between the interests, describing qualitatively the possible effects. These effects are used in the next step to design promising strategies.

5 **Formulation of possible strategies**
With the information gathered in the previous steps, it is now possible to design strategies, that look promising or are advocated by some interest group. This is where the "CZM control center", the triangle in the center of the system diagram, comes into the picture. This can be some administrative institution or conference of involved interest groups. Anyway, it is above the level of a particular interest.

6 **Assessment of system responses**
In the system diagram these are the same arrows as in step 6, but now the effects are quantified for the particular strategies that were developed in step 5.

7 **Choice of actions**
In the final step, in the "control center", a decision is taken on the preferred actions.

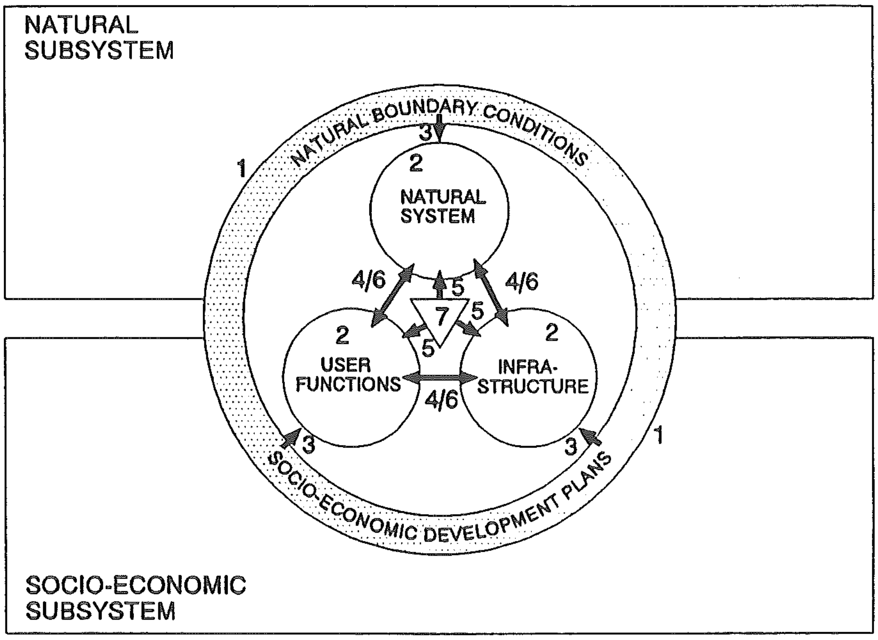

Figure 2.3 Steps related to system diagram

These steps are related to the system diagram in Figure 2.1. Figure 2.3 shows this relation. In the system diagram we have now gone from the outer boundaries to the core. The seven steps together form a policy analysis for a given problem.

2.1.4 Tools

With so many people, with so different background and interests, communication is crucial, but is by no means assured, and much energy has to be put into it. Common tools, common methodologies or even a common language are not self-evident in this field. Therefore, it is necessary to have adequate and common tools to accompany the seven steps in the previously given approach. Tools, in which the outcome of some interference in a system can be studied, are indispensable in CZM nowadays. Engineers and scientists have to make such tools in such a way that others (policy makers, decision makers, interested persons or parties) can do something with the results, see Janssen,1991, vdWeide and Tilman,1993 and World Coast,1993. In that case, the tools together act as a Decision Support System, which can range from a very simple model and presentation to a very sophisticated interrelated system of models with advanced postprocessing and evaluation.

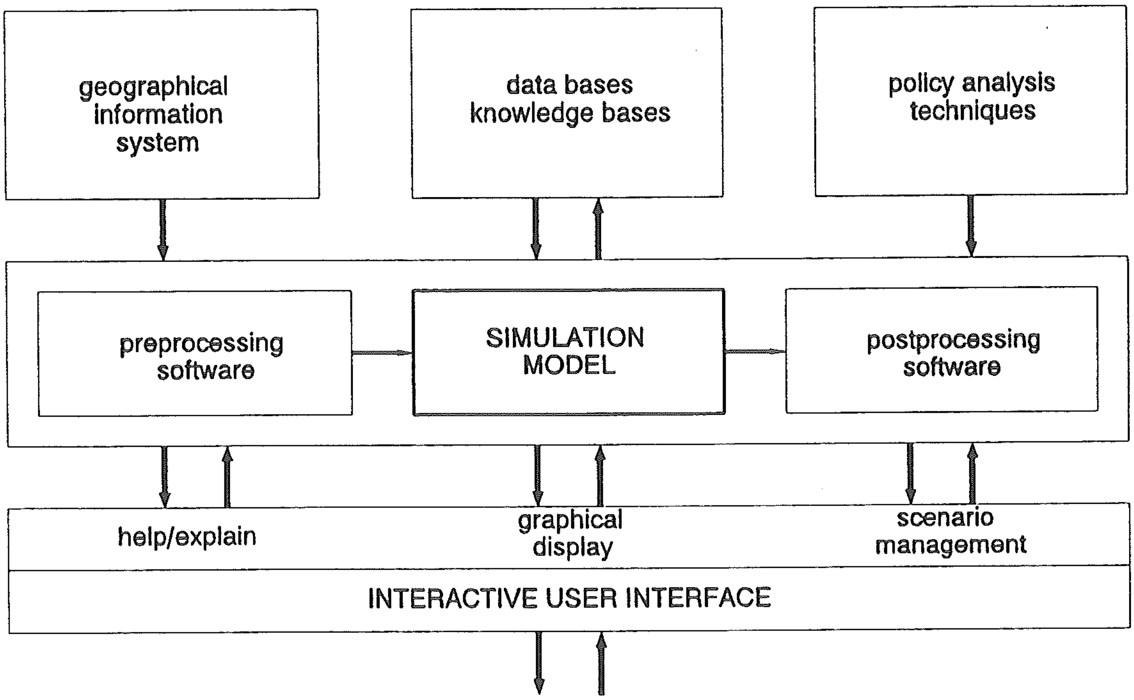

Figure 2.4 A framework for simulation model as part of a Decision Support System

The core of a Decision Support System, see Figure 2.4, is a simulation model, in which the relevant processes are modelled. User-friendly presentation and interface are necessary for the communication with the model. A Geographic Information System and data processing techniques are necessary to support the modelling and the pre/post-processing activities.

In the various steps of the previous section, the emphasis in the DSS lies on different elements. In the first three steps, geographical information systems and data bases play a crucial role. The simulation models, if used at all, will be of a rough nature. In the steps 4 and 6, the role of a simulation model is paramount. These steps are on the knowledge side of Figure 1.3, the simulation models used in these steps therefore also are knowledge oriented (descriptions of how the (sub)systems work). In step 5 and 7, the emphasis will be on choice, so the tools in this part of the DSS-toolbox will come from the policy analysis domain, including techniques like Multi Criterion Analysis, Score Cards etc. Scenario management is important to create strategies. Post-processing in this phase is primarily aimed at presenting data in such a way, that choices can be made. So, also in the supporting tools, one can distinguish the paths of knowledge and choice.

In order to describe the complex interaction between the resource system and the socio-economic system in the knowledge oriented simulation model, the following classes of models can be identified:

- **economic models**, which are based upon a description of the socio-economic system in terms of production and consumption and related flow of money, energy and resources;
- **resource models**, which focus on the production function of the natural system in terms of the availability of the required resources;
- **emission and deposition models**, which concentrate on the regulatory function of the natural system in terms of its physical, chemical and biological processes.

Until recently, only physical processes could be modelled. In recent years, however, models are being developed to describe the biologic and economic aspects of systems. Social systems are highly unpredictable and therefore difficult, or impossible, to model.

It should be realized, that all models are a schematization of the real world. Depending on the sophistication of the system description, first or higher approximation of the real world system can be obtained. Model validation is therefore required to check the conformity between model behaviour and behaviour of the real world. Often, such a validation has to be followed by a model calibration, to establish most appropriate values for model parameters for the case being studied. For practical purposes, only validated and calibrated models should be used.

The order of approximation to be selected is a compromise between required accuracy, technological possibilities and related costs. As the outcome of this evaluation differs from one project to another, a large variety of modelling concepts is being used in coastal sciences. Tools are used to serve a particular purpose; they are not an end in itself.

In general, the following rules of thumb apply:

- tools should be as simple as possible;
- if a more sophisticated tool is applied, the merits of the additional level of sophistication should be beyond doubt;
- tools should only be selected if their feasibility can be assured.

These rules hold for simulation models in the knowledge domain, as well as for policy analysis models in the choice domain.

2.2 Coastline management

2.2.1 What is coastline management ?

The terms coastal zone management, coastal resource management, integrated resources management and coastal area planning and management are often used interchangeably in the international literature. There are two components to this definition -- planning an management. The first component, integrated planning, is a process designed to interrelate and jointly guide the activities of two or more sectors in planning and development. The goal of integrated planning is the preparation of a comprehensive plan which specifies the means to effectively balance environmental protection, public use and economic development to achieve the optimum benefit for all concerned. The integration of activities usually involves coordination between data gathering and analysis, planning and implementation.
This final point is the essence of the management side of the definition. Coastal management is the process of implementing a plan designed to resolve conflicts among a variety of coastal users, to determine the most appropriate use of coastal resources, and to allocate uses and resources among legitimate stakeholders. Management is the actual control exerted over activities and resources. Public participation plays a key role in both planning and management. [Hildebrand, 1989]

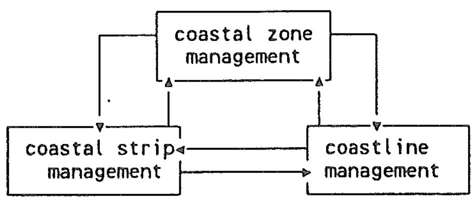

Figure 2.5: relation between CZM, Coastal strip management and CLM

Sometimes the term "Coastal Zone Management" is used for what is in fact "Coastline Management". In Figure 2.5 The relation is given between Coastal Zone Management, Coastal Strip Management and Coastline Management.
Coastal Zone Management is the integrated management of the Coastal Zone, i.e. the area where the influence of the sea is obvious. It mainly focusses on physical planning and resources allocation. Coastal Strip Management is the administrative management of a relatively narrow strip of land on both sides of the waterline. Regulations regarding set-back lines are typically subject of Coastal Strip Managament. Coastline Management is the management of the waterline, preventing erosion, if necessary, or allow erosion in such a way that it does not causes difficulties.
So "Coastline Management" is only a part of CZM; in fact Coastline Management has to be the result of a good CZM-policy for one sector. Coastline Management is managing a coastline in such a way that the CZM-policy can be executed. Thus: maintaining the coastline at those points where it is necessary, but also allowing a (controlled) retreat of the coastline in those places where maintaining the coastline position is not absolutely necessary.
The technical means to maintain a coastline vary considerably. Both soft measures (artificial beach nourishment) or hard measures (beach walls and revetments) can be considered. However, this is technical detailing of coastline management, and not coastal zone management.

2.2.2 *Alternatives in coastline management*

For the management of the coastline one has to distinguish clearly between problems due to flooding (inundation) and due to coastal erosion. Both problems have to be dealt with separately; they both need to be solved.

Inundation and erosion options

Inundation (Flooding) can be caused both by river floods as well as by storms at sea. For the Indonesian condition storms at sea are not so releveant. Only surges due to Tsunamis may be important in some parts of the archipel. Although river floods are not really the subject of this book, they also may be important in coastal areas. Therefor they often have to be included an a Coastline Management Policy.

Alternatives for a coastline management policy in any case must meet requirements concerning safety against inundation to a certain, *predefined* level. Additional demands concerning protection of other functions in the coastal zone landward of the waterline can be made. In principle one can opt for the following alternatives:

a. inundation is allowed with a rather high frequency, provision is made for the timely evacuation of inhabitants and for the survival of real estate during the flood.
b. inundation is principally not allowed; consequently no provision is made for evacuation, etc. However, much effort is then put into flood defence structures.

In the first case, the inundation frequency usually adopted for a coastal zone is in the order of once a year to once in 20 years. In the second case, the inundation frequency is once in 100 years or less. An inundation frequency between 1/20 per year and 1/100 per year should be avoided at all costs. In that range the flood occurs too seldom to keep the people aware that they have to maintain evacuation schemes, etc., but when a flood does occur the losses are too high to ignore them.

In the Netherlands option b has been selected. The allowable inundation frequency varies from 1/1250 per year for river flood planes to 1/10000 per year for intensively urbanised areas below mean sea level. In the United States (on the barrier islands, but also on may places in the floodplain of the Mississippi-river) and Denmark option a has been selected. In Denmark the inundation frequency is in the order of 1/20 per year. In the United States the frequency is not really determined, but is in the order of 1/10 per year.

Apart from the flood risk, one has to account for the loss of land due to coastal erosion. This problem can be approached in four different ways:

1. *Retreat*: the coastline is allowed to erode, provision has to be made to mitigate the problems due to this erosion.
2. *Selective preservation*: the coastline is allowed to retreat, except at those places where major interest in the coastal zone may be lost.
3. *Preservation*: the entire coastline will be maintained at a given location (for example the position of the coastline in 1995 will be maintained).
4. *Expansion seaward*: at locations of concentrated erosion, artificial defences extending into the sea will be built, bringing coastal recession to a standstill.

Some comments on these options

Retreat option

If nothing is done, the coastline will erode. This is not acceptable in locations where the the coast has a very important function, eg. in case of an ubanized waterfront.

If erosion continues, there is the possibility that villages in the coastal strip have to be relocated to a more inland location. In the past this has happened often to smaller villages. Also damage will be caused to recreational areas, natural reserves and agricultural areas. The consequence of this option is that a good set of set-back lines is required.

Selective erosion control option
The second alternative is to control the erosion in a selective way. Only at those locations where the eroding zone hosts very important functions, will action be taken. This can be done with beach nourishment, but other solutions are also possible (like the construction of a sea-dike). After some years artificial headlands will be formed along the coast (the coast between the headlands continues to erode). The costs to defend these headlands will increase in due course. Because there are many functions in the coastal strip, some choices have to be made. What has to be protected, what is "important" ? In this alternative for example the following choices can be made:

- all villages in the coastal strip will be protected;
- natural reserves with an (internationally recognized) high value will be protected;
- agricultural area with expensive irrigation structures (artificial irrigation and drainage) will be protected;
- investments for recreation will be protected (hotels, etc.).

The details of such an alternative have to be worked out on a regional level.

Full erosion control option
The coastline of a given year will be maintained. Erosion will be compensated fully. This can be done by beach nourishment, by groynes or by headlands. A small strip has to be available for natural fluctuations of the beach. Nourishments can be performed on the beach, but also just in front of the beach, on the inshore zone. There will be no loss of land.

Seaward expansion
This alternative is a more active one than the other alternatives. In this alternative the coastal strip is strengthened by making more beach in front of it. This can be done by the construction of very long groins and other constructions in the sea. The main purpose of this alternative is the creation of extra land. Usually this option will only be selected in areas where land has a very high value. This can be an economic value (like area for urban and industrial expansion (cf.the new port area of Rotterdam, the new airport of Hong Kong)), but it can also be an ecological value (e.g. the creation of new mangrove forests). It is a more offensive policy than the other ones.

2.2.3 Method of analysis

To allow a good political discussion on the subject, and to inform decision makers on all aspects of the alternatives for coastal protection a policy analysis approach has to be followed. For the four alternatives previously mentioned the effects on coastal defence, on nature, on recreation, etc. have to be computed in terms of extent and cost. Several options are possible:

Mitigation Policy Responses
This means that in case of (expected) erosion the functions, present in the coastal zone will have to be relocated to other places. New villages have to be constructed, or the

inhabitants are compensated in a financial way, new natural reserves have to be created, etc. With other words, the negative effects of the selected options are more-or-less compensated by positive action elsewhere.

Managing Development
Perhaps the most difficult approach politically, restricting beachfront development is also the most effective. Risk from coastal erosion is greatly diminished when development densities are reduced, beach front structures are set back. Areas most vulnerable to erosion such as highly mobile spits and very low mudflats are left undeveloped.
Purchasing property in eroding areas and holding it as open space is the most direct way to control development; however, it is also expensive. Although property acquisition can alleviate concerns that restricting development constitutes an unlawful taking of private property, it raises other issues such as whether public purchases of highly eroding land are sound investments.

Comprehensive Policies for Erosion Control Structures
Erosion control structures have the ironic effect of accelerating erosion, either in front of the development the structure is designed to protect, or downdrift. Groin fields, for example, interrupt the longshore transport of sand that replenishes beaches naturally, building up the beach on the updrift side of the groins and leaving steadily eroding beaches downdrift. Jetties designed to stabilize harbour mouths and inlets can affect entire regions. And although seawalls and bulkheads may stabilize an eroding shoreline for a time, normal and storm wave action eventually strip away the beach in front of the structure and scour out its base, causing the wall or bulkhead to fail.

Sound beach management requires that both national, provincial and local governments limit or prohibit erosion control structures, particularly vertical structures such as seawalls and bulkheads. Policy options range from requiring that erosion control structures be removed entirely to restrictions on control structures' repair and replacement.

Beach Renourishment
In areas where development is particularly dense, or to protect an important natural or man-made feature, beach renourishment may be viable. In a typical project, dredges or pumps move sand of a suitable type and size from a sand bar, an accreted area, or an upland source to the beach where it is graded to approximate a natural slope. Unfortunately, renourishment is expensive and sand has to be available. Further, there are no guarantees the new sand will stay in place, on the contrary, beach nourishments are designed to be eroded away.

2.2.4 Constructional aspects

Set-back lines
A set-back line is a line landward of the coastline. Between the set-back line and the coast no activities are allowed. There may be set-back lines for various activities. For example the set-back line for houses can be 50 m behind the coastline, while the set-back line for parking lots is perhaps only 5 m behind the coastline.
Of course it is necessary to define what is the "coastline". However, it is not very relevant how the coastline is defined, the only important thing is that the coastline is clearly and unambiguously defined, so that everyone knows what is the coastline. This

should also be clear in the case where the coastline is moving.
It is very inadvisable to define a set-back line only in meters. In any case the erosion speed of the coast has to be included. Therefore a definition such as "It is not allowed to build houses in a zone of width 50 m width behind the coastline" is not useful. In the case where there is a yearly erosion of 1 m per year, this might be acceptable, but when the erosion is 10 m per year, or when the erosion is 0.1 m per year, this is not an acceptable width.
Therefore one should always define a set-back line as a function of the coastal retreat. For example: "It is not allowed in a zone which is threatened by the sea in the coming 50 years". So in fact in this case the width of the set-back zone is 50 times the yearly erosion.

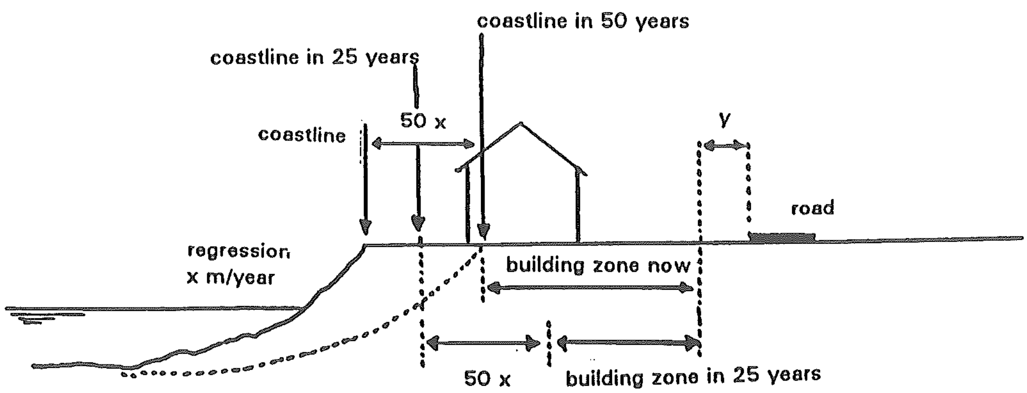

Figure 2.6: Moving set-back line

The consequence is that one has no problems with houses in the coming 50 years. However, what will happen between now and 50 years from now ? Of course the set-back line should move landward with the movement of the coastline. See figure.
The value of 50 years can for example be based upon the life-time of a house. The idea is that after 50 years the house has to be rebuilt anyway. So when all houses are built 50 erosion-years away from the coastline, it is never necessary to remove a house because of coastal erosion.
However, a practical planning problem will arise. Usually there is a street somewhat landward of the houses. Local regulations often forbid to build in a zone y meters from the street. So in fact the only place to build a house is in the "building zone" as indicated in the figure. This building zone is decreasing every year, because also the 50-years set-back line moves landward with a speed of x m per year. And at a certain moment the building zone becomes so narrow that no building is possible any more, simply because a house does not fit into the building zone any more. Such a lot then becomes unsuitable for building.

For the landowners this is a considerable economic loss, and they will certainly put pressure on the permitting authority to allow the construction of houses somewhat seaward of the set-back line. The temptation to do so is sometimes very great, especially when the landowner is quite influential. However, such a permit will cause managerial problems in the future (after a few decades). The civil servant who issues the permit now will probably not be in service any more, and therefore no longer accountable for his mistake. This aspect is usually the weak point in set-back strategies, especially in developing countries.

Another aspect is that the set-back line for houses is A years, because the life-time of a single house is L_H years (for example $L_H=50$). But the lifetime of a street of houses is considerably longer. So for streets a set-back line with a life-time of at least 2 L_H has to be selected. For a subdivision of a town, the lifetime is even more. Therefore the key elements of a subdivision (shopping centre, cultural and religious buildings, etc) should be located further away (e.g. 5 L_H).
In general one may use the following table:

Lifetime of a building	(50 years)
Lifetime of a street	(100 years)
Lifetime of a subdivision	(250 years)
Lifetime of a town	(1000 years)

summary:
- set-back lines move slowly landward
- various types of set-back lines:
  - for roads, foodstalls, etc
  - for building new houses
  - (for removal of houses)
  - for important buildings

Some authority is needed for the control of:
- illegal building in set-back zone
- removal of sand from beaches, foreshore, river-mouth
- removal of essential vegetation
- disposal of waste and wastewater

In general, in case of coastal erosion one may follow the guidelines of the "textbox" hereunder.

1 How much do you intend to pay for living at an eroding coastline ?
2 Is that amount enough to combat erosion in sustainable way ?
3a If yes: Combat erosion
3b If not: Is that amount enough to combat erosion temporarily ?
4a If yes: You have the possibility to give this problem to your children.
4b If no: make a retreat-policy:
- observe erosion carefully
- define good set-back lines
- make good regulations that everyone holds the set-back lines
- make people aware of the problem

What to do in case of coastal erosion ?

In the United States in 1987 the National Research Council (NRC) reviewed erosion management strategies and advised on how best to implement these strategies. They recommended to delineate coastlines subject to erosion to include:
- Imminent erosion hazards, or areas likely to be directly affected by erosion within 10 years;
- Intermediate erosion hazards, areas likely to be affected within 30 years; and
- Long-term hazards, areas likely to be affected within 60 years.

The NRC suggested that these zones (referred to as E-10, E-30, and E-60, respectively), be determined initially using historical shoreline change maps.

The NRC recommended that only readily moveable structures be permitted seaward of the E-60 line; most development should be confined landward of the E-30 line. No structures over 5,000 square feet should be allowed seaward of the E-60 line. No costruction should be allowed for the E-10 zone.

Artificial Beach Nourishment

It is not the intention to go into details of the technical aspects of artificial beach nourishment in this chapter. Only the managerial aspects will be discussed.
The technique of artificial beach nourishment is based on the idea that sand is brought towards the coastline from elsewhere, with the intention that it may erode at the beach. In this way it protects the original coastline. So: the sand is brought with the intention that it will be washed away by nature.
This is quite difficult to make clear to politicians and to the public. They usually think that after a nourishment the wide beach will stay in position. Everyone is very happy with the beautiful wide beach, but after the first stormy weather a large part of the new beach will disappear under water (in fact the artificial profile is adapted to a more natural one). The public does not see this sand under the water and so in their perception the nourishment is a failure.
Also one has to repeat the nourishment after a couple of years. That is no surprise for the technical managers of the coast, but the funding politicians prefer a solution "for ever".

Another managerial problem with artificial beach nourishment is that the work has to be done at more-or-less regular intervals. So after a number of years money needs to be made available to execute the renourishment. The interval is not constant as it depends on the number of storms during that interval. So, sometimes there can be an interval of 7 years between two nourishments, sometimes the interval is only 3 years. This requires a certain flexibility in the budgeting system, which is very often not possible in public finance.

Therfore, although artificial beach nourishment is often a very attractive way of coastline management, it is wise from a political point of view to sometimes opt for another solution.

Revetments and related hard structures

The public, and consequently also the politicians often demand a "hard and solid" solution. Very often they want a beachwall or a revetment, because they have the idea that in that way they can stop the erosion. When the erosion is caused by a gradient in the longshore sediment transport capacity (and this is usually the case), beach walls have no effect at all on the loss of sand due to this gradient. Their only effect is that the erosion does not take place at the dry part of the beach, but in front of the revetment, under water.
So in fact the shore in front of the revetment will become deeper and deeper, and eventually the wall will fall into the sea. The only way to prevent this, is by placing a bottom protection in front of the revetment or wall. Sometimes it is not necessary to place this revetment immediately after construction of the revetment, and its placement can be postponed for a number of years. This can be attractive from a financial point of view.

The advantage of a beach wall structure in coastal zone management is that it gives a clear border between land and water. This border line becomes quite static, which eases planning and land use. All land, landward of the wall, can be used for any function and (provided the wall is designed, constructed and maintained in a correct way), the land can be used "for ever".
Unfortunately, from an ecological point of view such a static separation between the terrestrial and the marine ecosystem is not desirable at all. From an environmental point of view a gently gradient between land and water is much more preferable.

From a management point of view, one should take care of the maintenance of the wall. In some countries the organisation of maintenance might be a problem, however with some effort this problem can usually be solved. More serious is that one has to monitor the shore in front of the wall. That means regular soundings, interpretation of the soundings and in case of unacceptable erosion, expansion of the bottom protection. This can be done by placing mattresses or dumping rip-rap in front of the wall.
The organisation of monitoring and maintenance of bottom protection systems is, in most countries of the world, a serious problem. These works cannot be seen (they are always under water), and if one neglects the maintenance for a limited time, no disaster occurs immediately. So often the maintenance is postponed because of budgetary reasons. But when this continues for too long, the damage becomes so considerable that even a minor storm may completely destroy the structure.

An other important drawback of revetment and beach wall structures is that in the medium and long run the beach in front of the wall will disappear. This will create considerable problems, especially when the beach has a touristical function. Often the only solution in such a situation is to execute massive beach nourishment.

Along the Belgian coastline such a situation exists. Nearly all the way along the coastline of Belgium condominiums have been built on the coastline. In front of the buildings there is a road, separated from the beach by a beach wall. At the locations where the coast is eroding, this causes a lowering of the beach, resulting in a fully submerged beach during high water. Because the Belgian beach has a very important touristical and recreational function, this cannot be tolerated, and the Belgian government has to spend considerable amounts of money on beach nourishment in order to maintain the beaches.

2.3 Design of coastal works

2.3.1 The traditional design method

During his education the average design engineer has learned a lot of Facts and Techniques. However, he was never trained in Asking Questions. Many engineers are much more familiar with Analysis. During their previous engineering education the engineer mainly dealt with problems that are essentially mono-disciplinary. The problems to be solved were selected to illustrate and reinforce the principles taught. If the engineer was able to construct the appropriate model, he can solve the problem. Most of the input data and properties were given, and there usually was a correct answer to the problem. Even during his professional carrier after graduation he still follow the same approach to solve real world problems. The engineer expects clear formulation of the problem, sufficient data and information available, and he treats the problem on hand essentially as a single disciplinary problem. A fundamental characteristic of this type of engineer is that in most cases he Think in Solutions, without realizing what the Real Problem is. The following example will illustrate this.

Example 2. 1:

> *"It is hot in the room, I am thirsty! t am going to the canteen and have a glass of water." This is a fundamental characteristic of our way of thinking. In most cases we think "In Solutions", without asking what the reason is of being thirsty? Maybe the situation in the room? What makes the situation in the room me thirsty? Is it the temperature, humidity, or something else? Taking a glass of water will not solve the "Real Problem': It is only a remedy against the symptom (= thirsty) of the problem (= dry situation in the room). After taking a glass of water and returning to the room I will have the same problem after some time. The problem has not been solved! We keep circling in circles, because we think only in solutions and did not analyze the problem. If we would like to solve the real problem, we have to analyze the situation in the room. If it is a dry environment, we need to control the humidity in the room. If it is the heat, maybe the room needs isolation or temperature control. In this way we do real problem solving and not symptom combatment only.*

Except in case of a complete new problem, there are always a number of existing examples of solutions for a more or less similar problem on hand. One may collect existing solutions and compare them with the problem set, which is in fact an Analysis of Existing Solutions in order to find out to what extent they meet the requirements or needs of the problem to be solved. Ultimately one arrives at an "Adjusted Design" in which all useful elements of the existing solutions are included. The analysis of existing solutions is part of "Experience". In some cases so called "Standard Designs" or "Typical Designs" of structures are available based on experience elsewhere. When applying these typical design the engineer rarely have a way of knowing the correct answer. Hopefully the design works, but is it the best or most efficient design for the problem on hand? Only time will tell.

The method outlined above is a typical Traditional Design Method. The disadvantage of the traditional design method is that it assumes that the knowledge and experience are readily available to the design engineer. This is not always the case, particularly when dealing with a complete new problem or when the design engineer does not have the

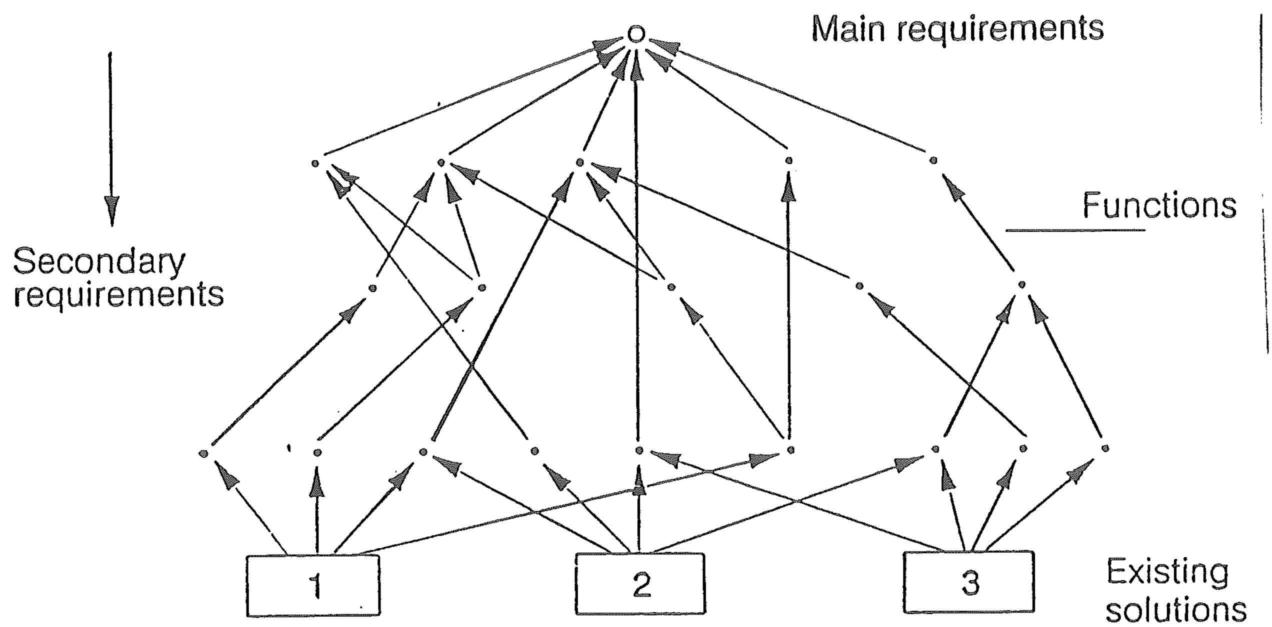

Figure 2.7: The traditional design method

knowledge or experience to solve the specific problem. Figure 2.7 illustrates the traditional design method in a schematic way.
Real world problems are rarely that neat and the Real Problem that the design engineer is expected to solve may not be readily apparent. He may need to draw from many disciplines, including non-engineering disciplines, to arrive at a solution. The data available may be very fragmentary at best, and the scope of the project may be so large that no individual can follow it at all. Furthermore, usually the design must proceed under severe constraints, e.g. time, money, environment, social, etc. Hereafter, three types of design methods will be discussed briefly. The first two are typical traditional design methods, while the third is a combination of these two.

2.3.2 The black box method

Most people think of a traditional design engineer as an artist or magician who uses some computations, judgement based on experience, and especially genius thought flashes to arrive at successful solutions. The essential part of the design process takes place within the brain of the designer. Although many solutions designed in this way performed quite satisfactorily, the designer is not always able to explain the reason of his decisions. The final result of design according to this method depends on the information of the problem received and information gathered from other examples or experience, and the constraints imposed. However, by removing a number of constraints the design engineer can be stimulated to be more creative in his search for new solutions.

2.3.3 The glass box method

While the black box method is mainly based on intuition and feeling, the glass box method is more based on comprehension and reasoning. Essentially this method considers design as a rational process, although many designers using this method are still not able to explain all their decisions. In many cases he uses design standards or guidelines, and standard computer packages for his computations. One may call these type of design engineers as "Human computers". He reacts to the information supplied to him and after following a predefined set of actions and computation steps, he ultimately arrives at the "best" of a number of possible solutions. The main characteristic of this method is that **the objectives, the variables and the criteria are defined prior to the design process.** This method **is mainly suitable to solve routine** design problems, which **can be solved** by standard procedures or computer programmes. Using symbols, the results of the design process can be assessed by others. In this way alternative solutions can be explained and developed. Also this method requires experiences with other similar problems solved before. This method is more transparent for others and thus easier to communicate than the black box method.

2.3.4 The strategy control method

In many situations, especially where experience is missing, both methods fail to arrive at proper solutions. In these cases the need for new methods where the positive aspects of the two methods are combined, emerges. In the strategy control method the design process is divided into two parts, i.e. searching for the solution, and control & evaluation of the design strategy. The main characteristic of this method is that it establishes a relation between the solutions and the objectives. The final result depends on the ability to control the design strategy and on the design problem. Each problem requires its own design strategy. This method is a more fundamental approach to solve complete new or complex and multi-disciplinary problems, where no existing solutions or experiences are available. An example of this method is the Fundamental Design Method (FDM) developed by Matchett (see chapter 7).

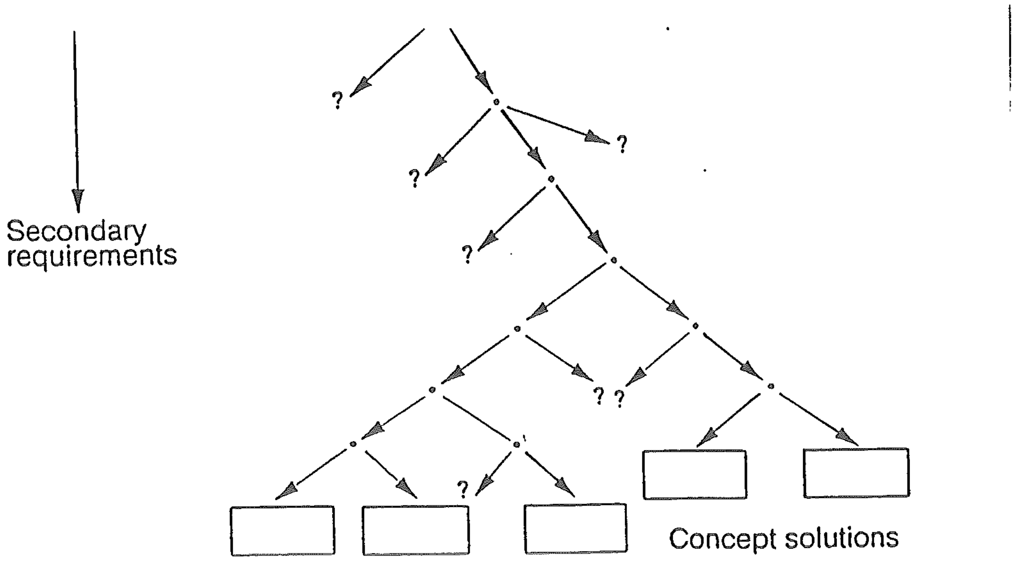

Figure 2.8: The fundamental design method

While the emphasis of the traditional design methods is placed on the analysis of existing structures to arrive at a syntheses (arrange existing things in a new way), The Fundamental Design Method places its emphasis on the analysis of the problem self and its requirements to arrive at a solution. See Figure 2.8. In practice however, both methods are used together. Although many modern design methods exist, for the design of coastal engineering works, the FDM approach is very suitable and the concept will generally be followed throughout this manual.

2.3.5 Discussion

Before starting with the design we have to analyze the problem. A diagnosis is needed. What is really the problem"

> *"Before starting the design, it has to be clear what is the problem, and what is the cause of the problem. Solving some of the syptoms, is often not a good solution."*

Why do we need a design method?

> *"To conduct our design correctly and efficiently according to established scientific principles to arrive at an optimal solution."*

What is correct design?

> *"A Correct Design is always the <u>Optimal</u> fulfilment of <u>All Real Needs</u> under a <u>Certain Condition</u>."*

The definition above contains the following elements:

- Less than optimal is less better; optimal is a function of time, cost and quality.
- All real needs should be considered in order to avoid imbalance of the design, e.g. cheap structure but high risk of failure.
- Consider only real needs; to apply prefab elements should follow from the needs and not because the contractor has over-capacity in production.
- A design which performs well in a dry climate is not automatically a good design for a humid climate.

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3 Natural system (Descriptive)

3.1 Geomorphology

3.1.1 General

Coastal processes are characterized by a very wide variation in time and length scales. Typical time scales in wind waves are of the order of seconds, of hours in tidal waves, days in banjir river outflow, weeks and months in monsoons, (many) years in sedimentation till millions of years in tectonic processes. Typical length scales are microns for a single sand grain or silt particle, meters for the width of a beach, kilometers for beach lengths, tens of kilometers for delta coasts, hundreds of kilometers for shelves and thousands for tectonic plates.

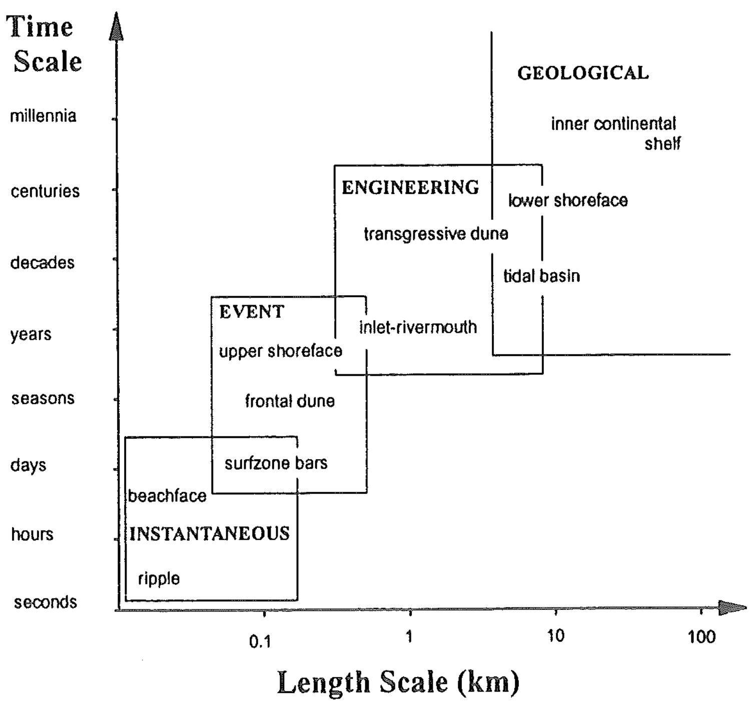


Figure 3.1 Time and length scales in coastal engineering (Carter and Woodroffe,1994)

Figure 3.1 (from Carter and Woodroffe,1994) shows some of these scales in relation to each other. Sediment transport due to flow or waves can be seen as *INSTANTANEOUS* adaptation to fluctuating boundary conditions, while an *EVENT* (a storm, a bandjir, a volcanic eruption but also seasonal variations) can bring about changes long after the event has happened. *ENGINEERING* indicates a time scale that spans many fluctuations in boundary conditions, while *GEOLOGICAL* denotes changes in trends of boundary conditions, like sea level rise or tectonic movements.

Most coastal morphodynamic studies have been done on the first two scales, which is usually shorter than the lifecycle of most engineering works. Furthermore, in a geologically dynamic environment as the Indonesian archipelago, understanding coastal processes without some notion of the geological processes, is virtually impossible. Therefore, we will start with the description of the most relevant large scale processes.

Inman and Nordstrom,1971 discern three scales in coastal development:

> **First-order features (tectonics):** associated with the tectonics of the earths crust with typical length scales of 1000 km longshore, 100 km crossshore (incl. continental shelf) and 10 km vertical.
> **Second-order features (sediment budget):** associated with erosion of mountains and deposition in deltas and on continental shelves, modifying the first-order features. Typical length scales compared with the first-order features are 100 km, 10 km and 1 km respectively.
> **Third-order features (shoreline dynamics):** associated with nearshore transporting and depositional processes depending on e.g. wave action and sediment grain dimensions. Typical length scales are now 1-10 km, 100m-1 km and 10 m respectively.

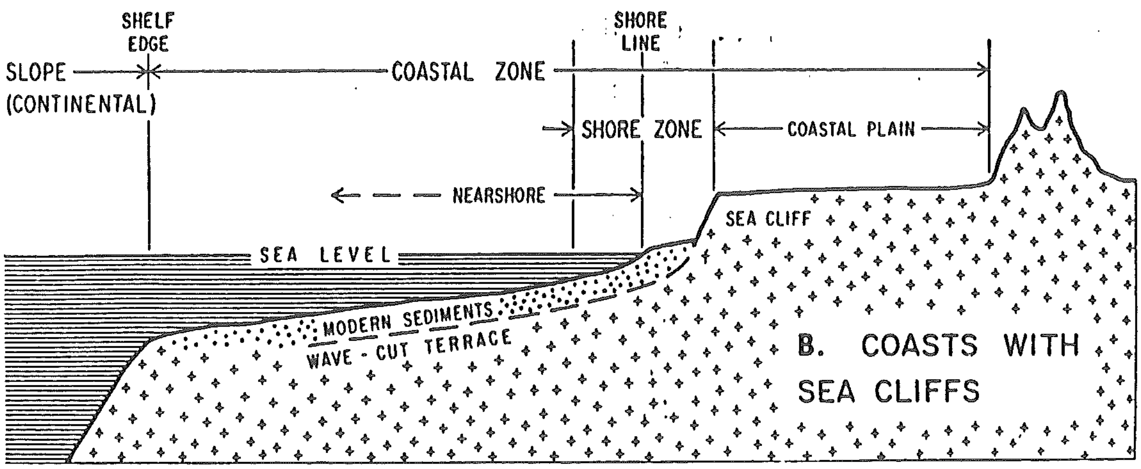


Figure 3.2 Definition sketch for coastal zone nomenclature (from: Inman and Nordstrom,1971)

Together with these features, Inman and Nordstrom discern two zones, see Figure 3.2: a "coastal zone" based on the first- and second-order features, and a "shore zone" based on third-order features. These definitions serve a physiographical purpose and the coastal zone as defined here is not necessarily the same as usual in Coastal Zone Management (CZM). But the same division as between CZM and Coastal Engineering, can be felt here. This manual focusses on the coastline and hence on the third-order features, but talking about these features only makes sense against the background of the first- and second order features, specially in the Indonesian archipelago. Therefore, these feature orders will be described in the following sections, using the names in brackets.

3.1.2 Tectonics

The tectonically highly active region has been the subject of many geological studies for many years. The first Snellius expedition in 1929-1930 has revealed much knowledge of the geological history of the area, see e.g. Umbgrove,1947 and van Bemmelen,1949. The Snellius-II expedition gave new momentum to the research on the genesis of the archipelago. For an overview of the evolution of ideas about regional geology, see Katili,1989.

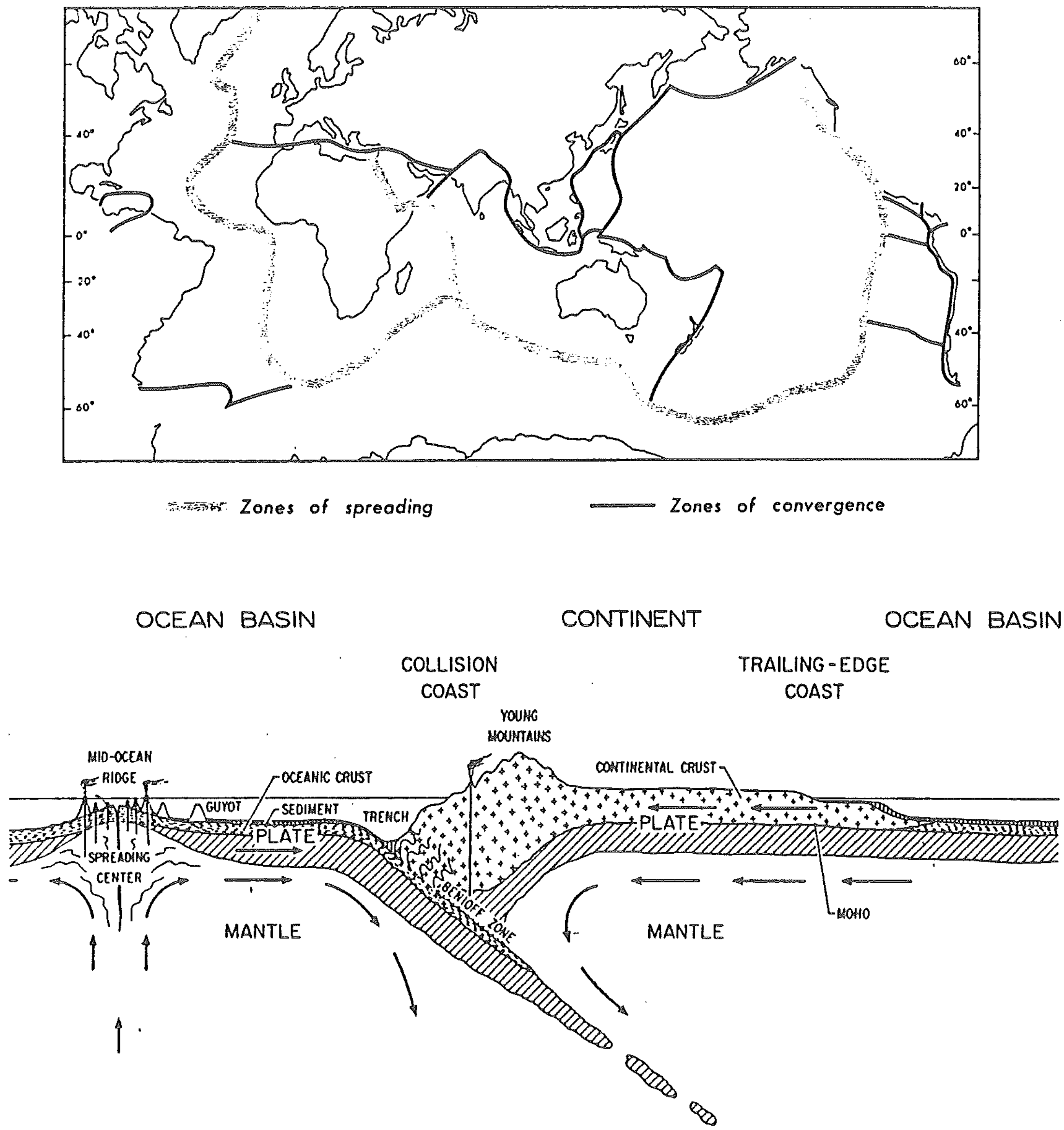


Figure 3.3 Diverging and converging zones of oceanic plates and the formation of collision and trailing-edge coasts (from Inman and Nordstrom,1971)

The generally accepted theory that describes best the origin and nature of the first order features of the Indonesian coasts is the concept of plate tectonics, see Figure 3.3 from Inman and Nordstrom,1971. The earth's crust under the oceans is quite different from continental plates. Basalt erupts from the earth's mantle through deep fractures in the crust, the so-called mid-ocean ridges, and diverges from there in two directions. This basalt ($\rho \approx 3000$ kg/m^3) is heavier than the granite ($\rho \approx 2700$ kg/m^3) of the continental plates, which tend to ride on the oceanic plates.

Where two oceanic plates converge, one is pushed down under the other, see Figure 3.3, a process that comes with mountain building. When a continental plate lies atop the oceanic plate, the edge is more or less crushed and a **continental collision or leading edge coast** is formed with a deep trench in front without a continental shelf. The coast on the other side of the continental plate is in a completely different situation and here the second-order feature of sedimentation can create a continental shelf. This type of coast is named a **trailing edge coast**. When neither of the converging oceanic plates carries a continent at its leading edge, a so-called **island arc collision coast** emerges. Behind an island arc a **marginal sea coast** can exist, which is, for our purpose, not very different from a trailing edge coast. A somewhat more detailed division is given in Inman and Nordstrom,1971, but for the Indonesian archipelago these are the most important ones.

Looking at the map of SE-Asia, these first-order features can clearly be seen. The West coast of Sumatera and the South coast of Java are clearly collision coasts, while the coasts along the Sunda shelf can be seen as trailing edge or marginal sea coasts. The area between the Eurasian and Australian continental plates is an example of an island arc (the Banda arc) with very deep seas in between. **Note:** Sometimes all Indonesian coasts are classified as collision coasts, probably from the idea that they all are island collision coasts. This is not true for the Java sea coasts of Sumatera, Java and Kalimantan. The Sunda shelf is clearly part of the Eurasian continent, hence they are no island arc collision coasts, but should be seen as trailing edge coasts!

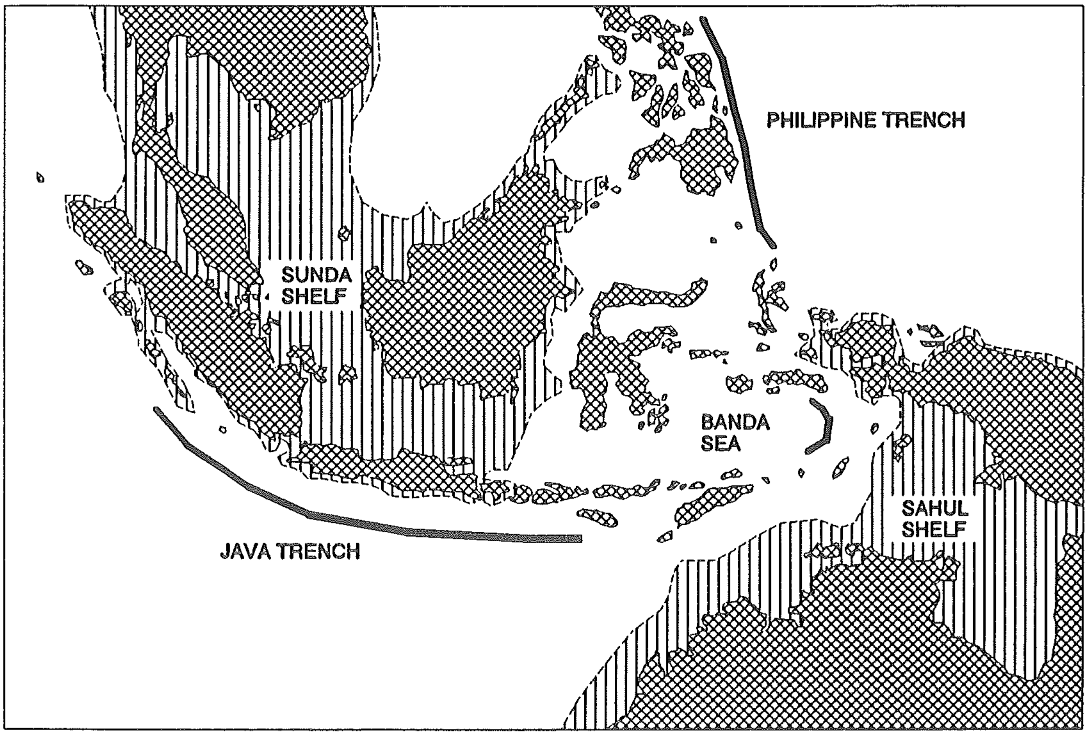


Figure 3.4 Bathymetry of the Indonesian archipelago and adjacent areas

3.1.3 Sediment budget

The second-order features deal with the erosion of the mountains formed by the first-order features and are just as important and specific for the region. The tropical wet climate makes the sediment yield of the rivers much larger than in temperate climate regions and much more important than the direct abrasion by wave action of igneous or sedimentary rocks (which is worldwide already no more than 5 - 10 % of the sediment on beaches, see e.g. Komar,1976.

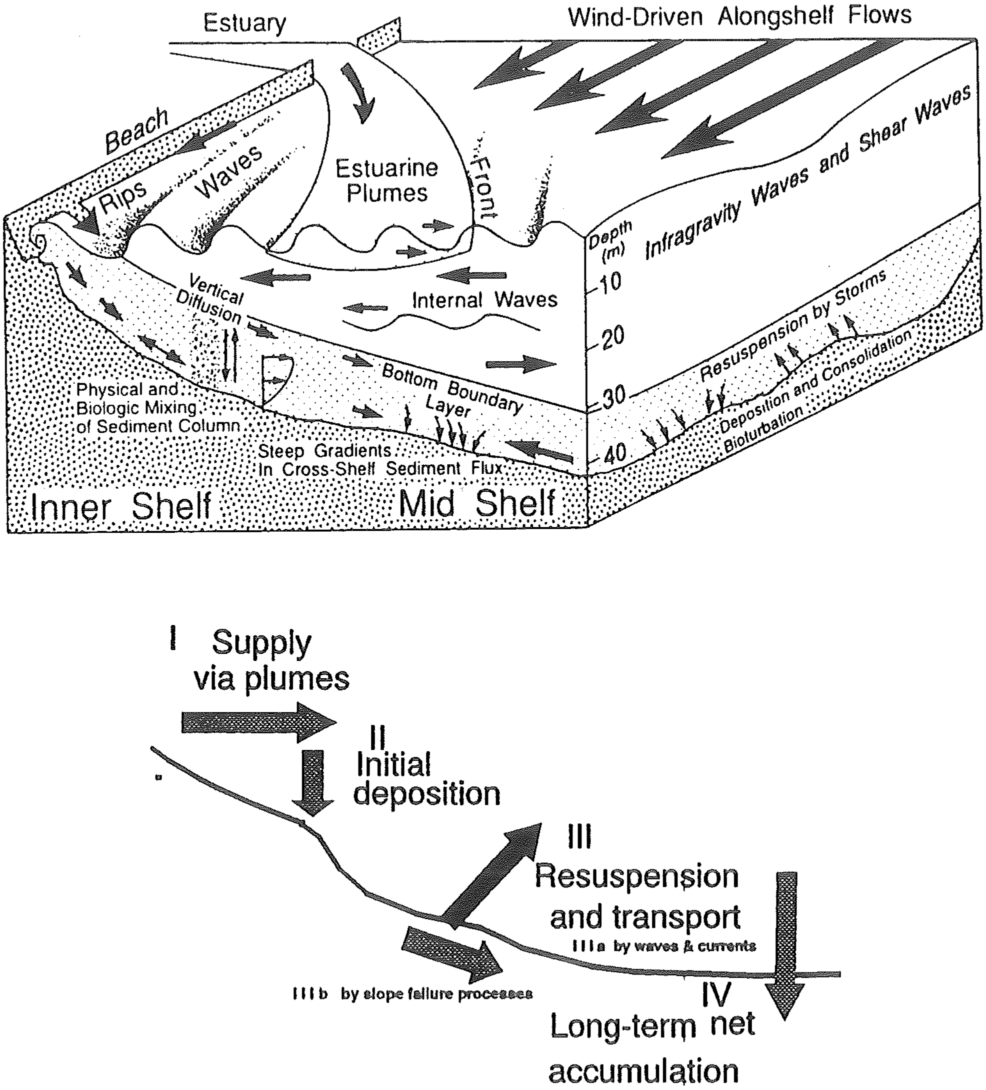


Figure 3.5 Conceptual diagram illustrating physical transport processes and stages in the dispersal of river sediments on the continental shelf (from Wright,1995)

Figure 3.5 shows the idea of the budget of river sediment on a shelf. The river sediment enters the sea usually in a plume with lower density than seawater (stage I). It deposits (stage II) contemporaneously with stage I. Following, or in some cases concurrent with, their initial deposition, river-derived sediments may be resuspended and further transported by forces such as waves and currents (stage III). Where the environment is energetic throughout the year and bed stresses exceed the threshold for particle movement, stages II and III will occur simultaneously. (This is probably the case in the Java Sea, when NW-monsoon wind and rainy period occur simultaneously in December through March). Stage II and III can be repeated several times. Stage IV is the "final" accumulation of sediment in a sink. This can be the ocean bottom, but also an estuary that is filling up slowly. The sediment supply and size is, of course, also important in these stages. Here, the tropics deviate considerably from temperate regions on earth.

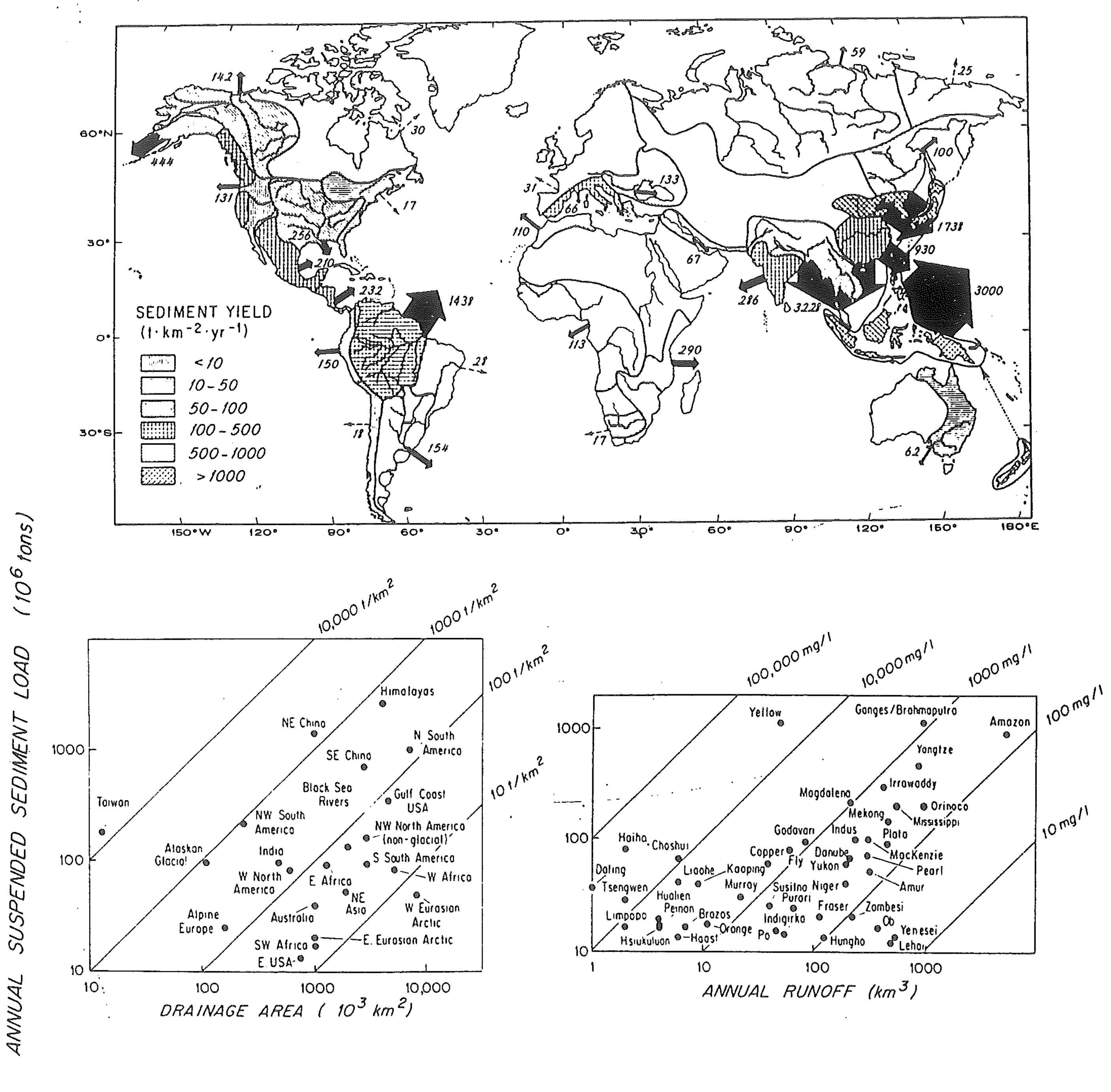


Figure 3.6 Sediment yield and related parameters from various drainage basins of the world. (from: Milliman and Meade, 1983)

Figure 3.6 shows the delivery of sediment by rivers to the oceans (from Milliman and Meade,1983). It is estimated that 50 - 70 % of the total sediment discharge from the continents to the oceans is delivered in SE-Asia! This is again caused by tectonics and climate. The Himalaya as the highest mountain range in the world is abrased and feeds large river systems, while the tropical climat causes intense weathering. The rivers in Indonesia are estimated to have a sediment yield of more than 1000 tons sediment per km^2 drainage area per year, which is a factor 10 to 100 more than rivers in temperate and geologically less active regions.

These sediments are transported to the coasts and from there, via the continental shelves, finally to the oceans. The Sunda shelf, see Figure 3.4, is not formed by sedimentation, but is a part of the Eurasian continent and was a "peneplain" between the mountains of Sumatera, Borneo and Java in the Pleistocene era when the sea level was more than 100 m lower (the Java sea is about 50 m deep). It is therefore part of the first-order features, but once it is there, it functions as an important reception basin for the sediment run-off from the islands and, hence, as an important factor in coastal forms and processes.

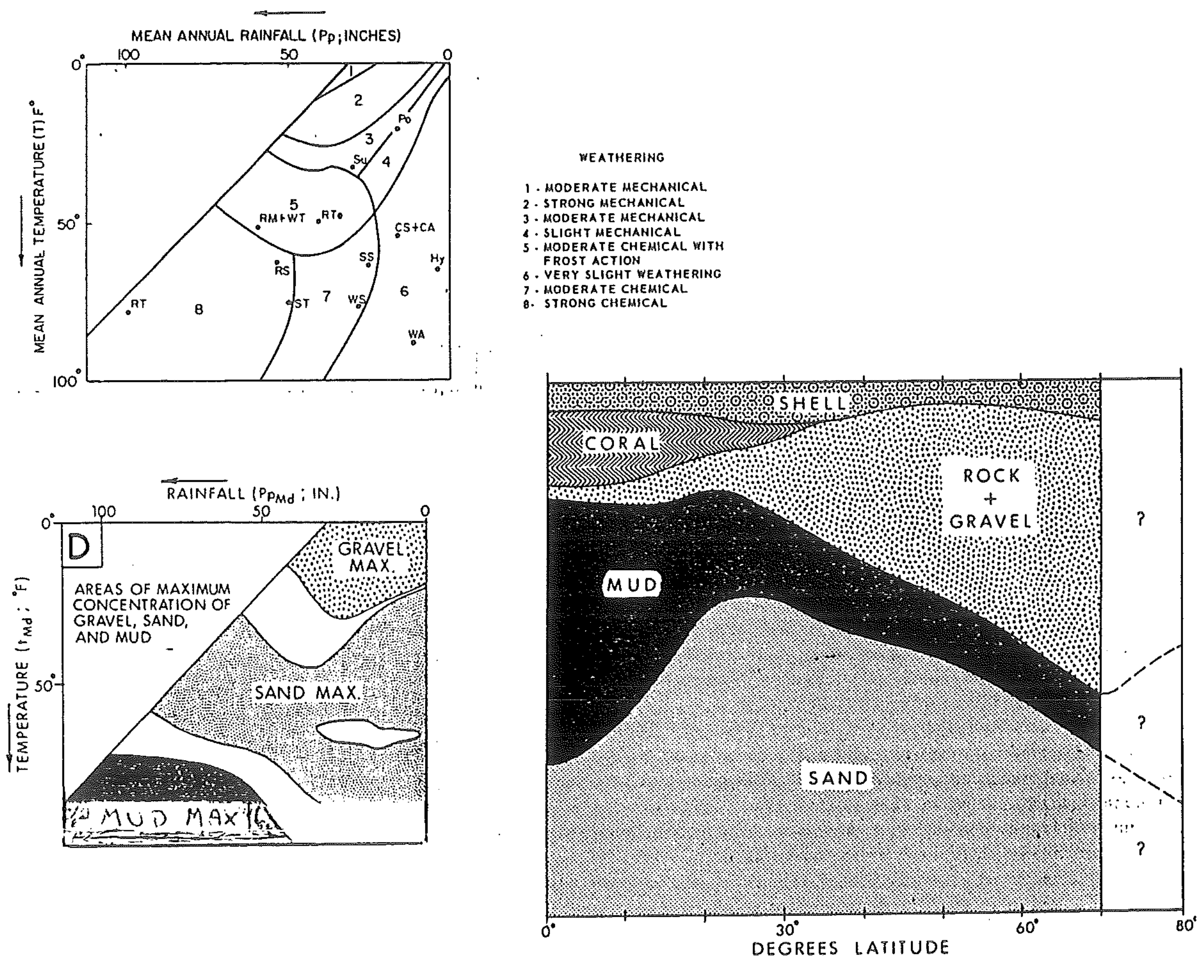


Figure 3.7 Relation between temperature, rainfall, weathering and sediment types and resulting zonation of inner shelf bottom sediments as a function of latitude (from Hayes,1967)

Another typical factor for the region is the abundance of very fine sediments, like silt and clay particles. This is primarily caused by the climate. Figure 3.7 gives an impression of the weathering as a function of climate (rainfall and temperature) and the types of sediments found on the inner continental shelves (from Hayes,1967). This leads to the lower part of Figure 3.7, from which can be seen that around the equator, on an average, the upper layer of the inner shelf bottom consists for about 30 % of sand, for 50 % of mud, 15 % of coral and 5 % of shells. Rock and gravel, which are abundantly present at higher latitudes, are virtually absent in the tropics.

In enclosed and protected seas, it can be expected that fine sediments may be even more important than the average in Figure 3.7. In the framework of the Snellius-II expedition, biological research was done at the North coast of Java, Bali and Lombok. In that study, also data on the sediment properties was collected (de Wilde et al,1989). Figure 3.8 shows the results. At the North coast of Java, the silt contents is more than 95 %. Where velocities are relatively high, like in the strait between Bali and Java, the silt content is very low.

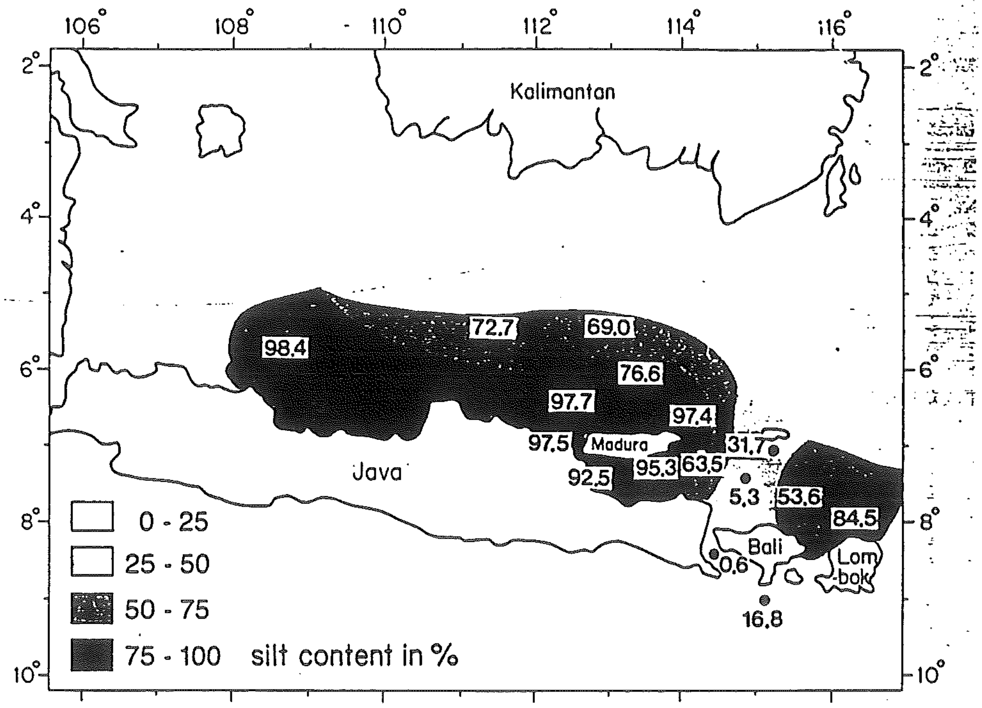


Figure 3.8 Distribution of silt (particles $< 50\mu m$) content of the upper bottom layer (from de Wilde et al.,1989)

The same can be found in Jakarta Bay, see Figure 3.9. Now the sand content is indicated instead of the silt content, so a low figure means muddy. In large parts of the bay, the sand content is very low, comparable with what was found in East Java. Outside the bay (Northwest of the +++++ line) the sand content reaches values of more than 50%. The influence of the Citarum delta (NE part of the bay) is clearly visible: very low sand contents in front of the delta and high contents very near the delta, because there the heavy particles settle first, see also section 3.2.2.

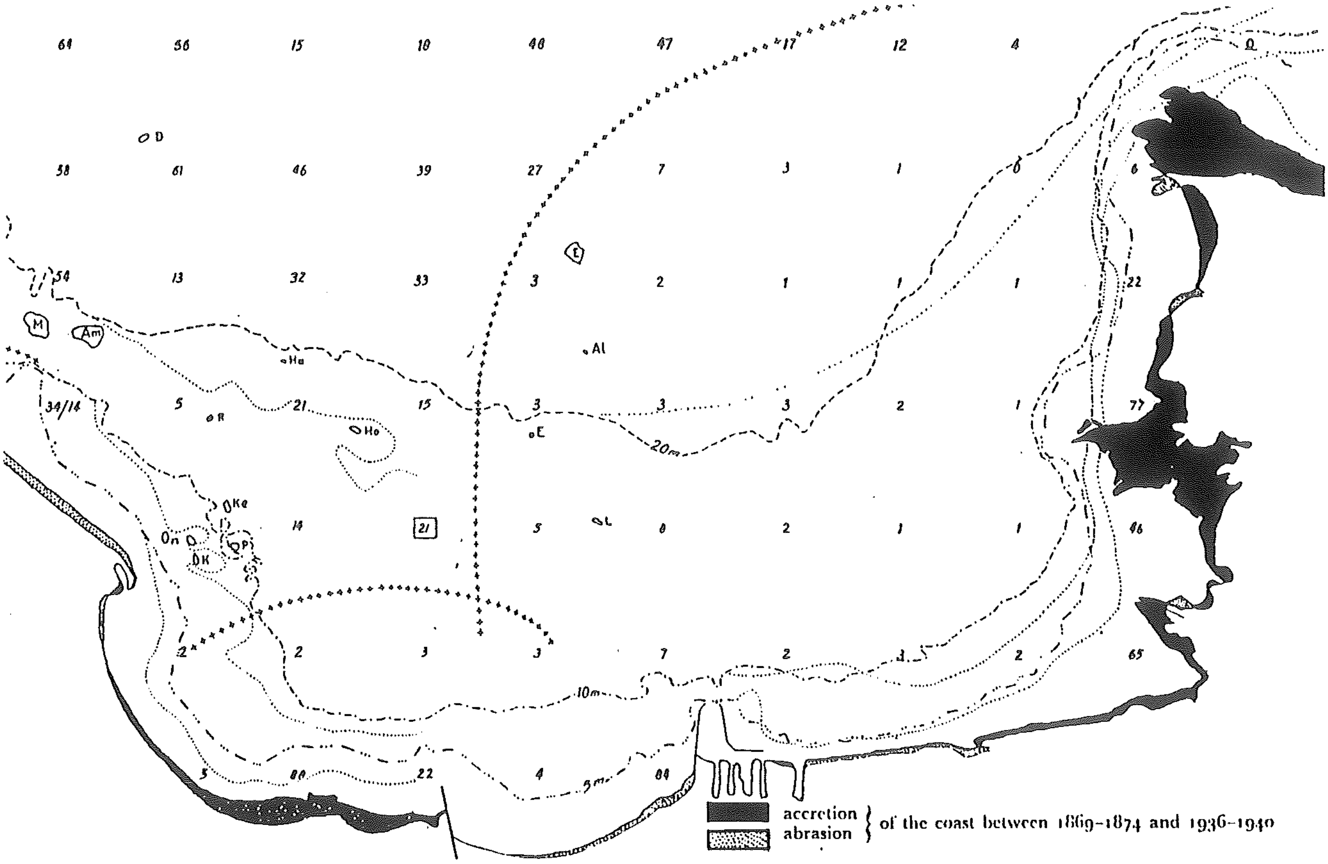


Figure 3.9 Sand content in Jakarta Bay. Numbers indicate percentage sand. (from Verstappen, 1953)

Around the Pulau Seribu, North of Jakarta, there are sandy patches between the coral islands where the velocity is relatively high, see van Bemmelen, 1949. Figure 3.10 shows some information of sediments in the Java Sea. It appears that farther offshore sand and gravel can be found, probably from the pleistocene when the Java Sea was still a plain between the mountains of what are now the islands of Sumatera, Java and Kalimantan, with a relatively high ridge in the center (where now the Karimunjawa islands are situated.

From all available sediment data, the conclusion seems justified that the very fine river sediments are deposited and transported in a relatively narrow belt along the coast.

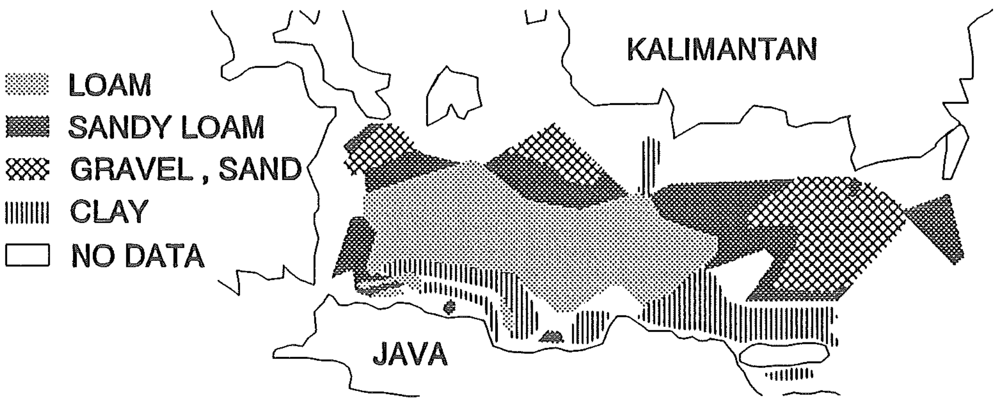


Figure 3.10 Sediments of the bottom of the Java Sea (from KNAG, 1922)

3.1.4 Shoreline dynamics

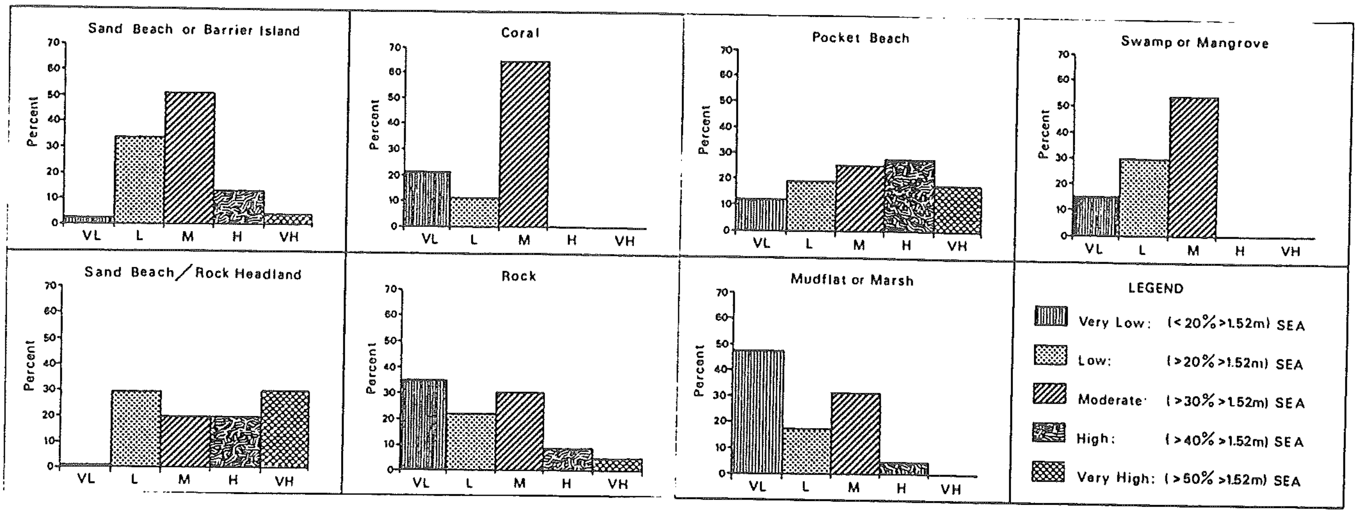
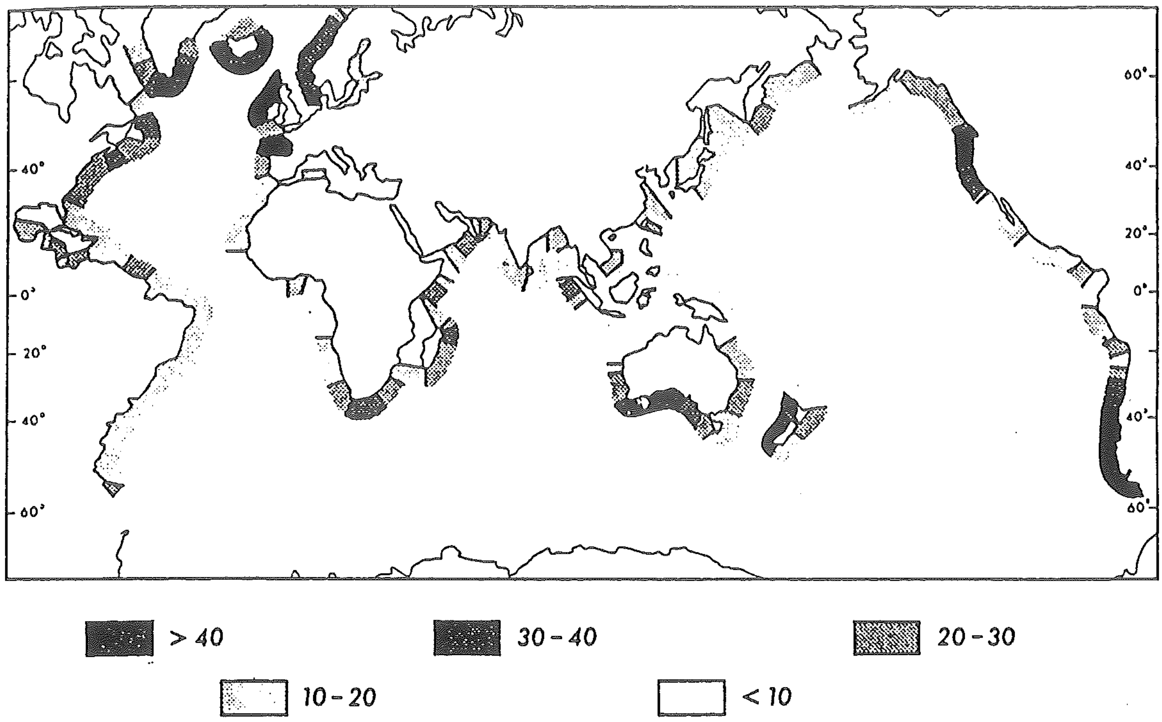


Figure 3.11 Wave climate and shoreline type of the Americas (from Dolan et al.,1975)

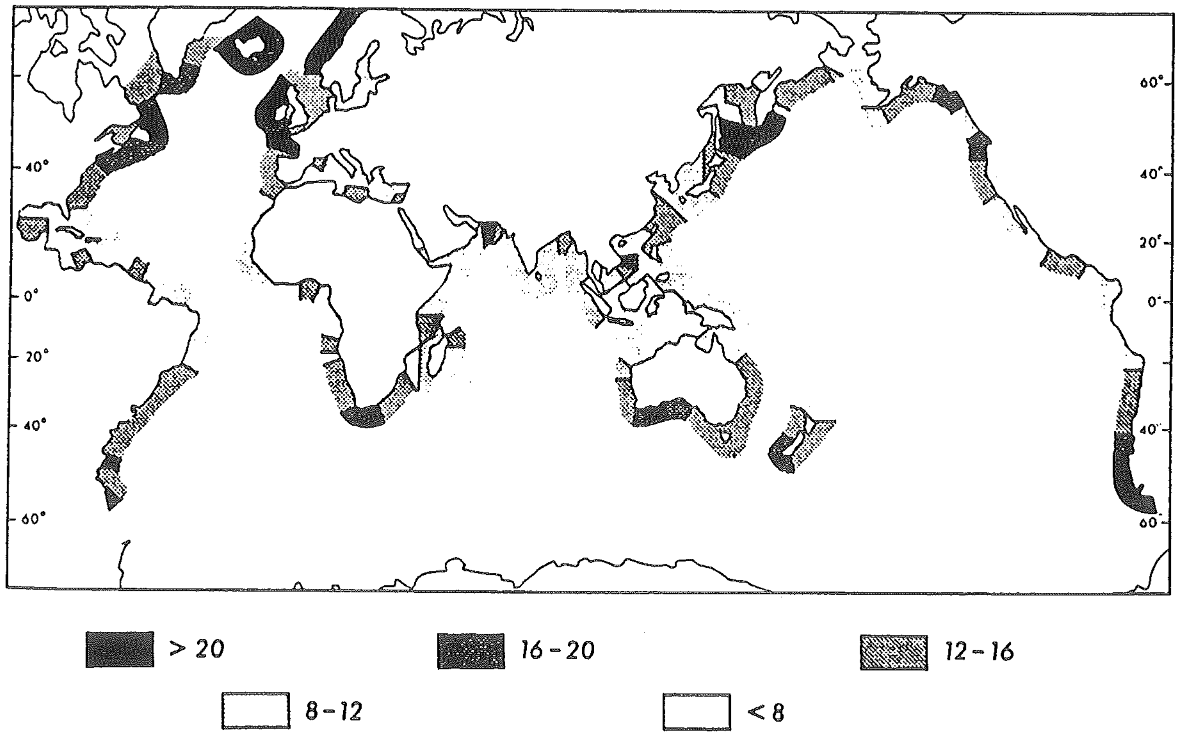
The sediments that are delivered by the rivers to the sea, also make up largely the sediments of the coastline in the Indonesian archipelago, but it is good to bare in mind from the previous sections that this sediment does not "belong to the coast", but it is only on its way from the river to the ocean. This very basic awareness already indicates the dynamic character of shorelines. The dynamics is further intensified by the action of tidal currents and waves. Especially the latter determines the type of sediment on the shoreline, see Figure 3.11. This figure comes from the American continent. Mangrove (mud) coasts, abundantely present in Indonesia, can not exist in higher wave climates. In America the same is found for coral reefs, which is not in line with what is found in Indonesia. Intense wave action is often found at the ocean side of coral reefs in South East Asia and the Pacific.
******** In the final version of the Pedoman a similar figure as Figure 3.11 should be included for the Indonesian archipelago ********

Figure 3.12 gives some rough figures of the wave climate in SE-Asia. Although very rough, it can be explained, together with the sediment supply to the Sunda shelf as described in the previous section, that the East coast of Sumatera, the North coast of Java and the South coast of Kalimantan are mainly muddy. The difference between the West coast (mud) and the East coast (sand) of the Malaysian peninsula also comes from the difference in monsoon winds and fetch. The coasts that border the Indian Ocean, have hardly any continental shelf (collision coast, see section 3.1.2), receive much less sediment and experience higher waves (specially visible on the West coast of Sumatera). There, sandy beaches and rock coasts with pocket beaches can be expected.

Note: In many manuals on shoreline protection, much attention is given to beach and dune erosion due to storms. This may be useful for coasts in the temperate regions, but for Indonesia, this will be less relevant, due to the differences in sediment size and wave climate.



Percentage frequency of occurrence in at least two quarters of the year of waves 8 ft high or higher. (Data compiled by Meisburger, 1962.)



Greatest height in feet reached by waves occurring with a frequency of 3% or greater in at least two quarters of the year. (From data compiled by Meisburger, 1962.)

Figure 3.12 Wave heights and frequencies on coasts in SE-Asia (from Davies,1985)

3.2 *Coastal forms and processes*

3.2.1 *General*

Coastal forms and processes are mentioned together deliberately, because it is dangerous to look at a coastal form as to a slide instead of as to a movie. These forms and processes belong to the second- and third-order features of section 3.1.1 (and even first-order when tectonics still play a role at present), but no clear distinction will be made between these orders from now on.

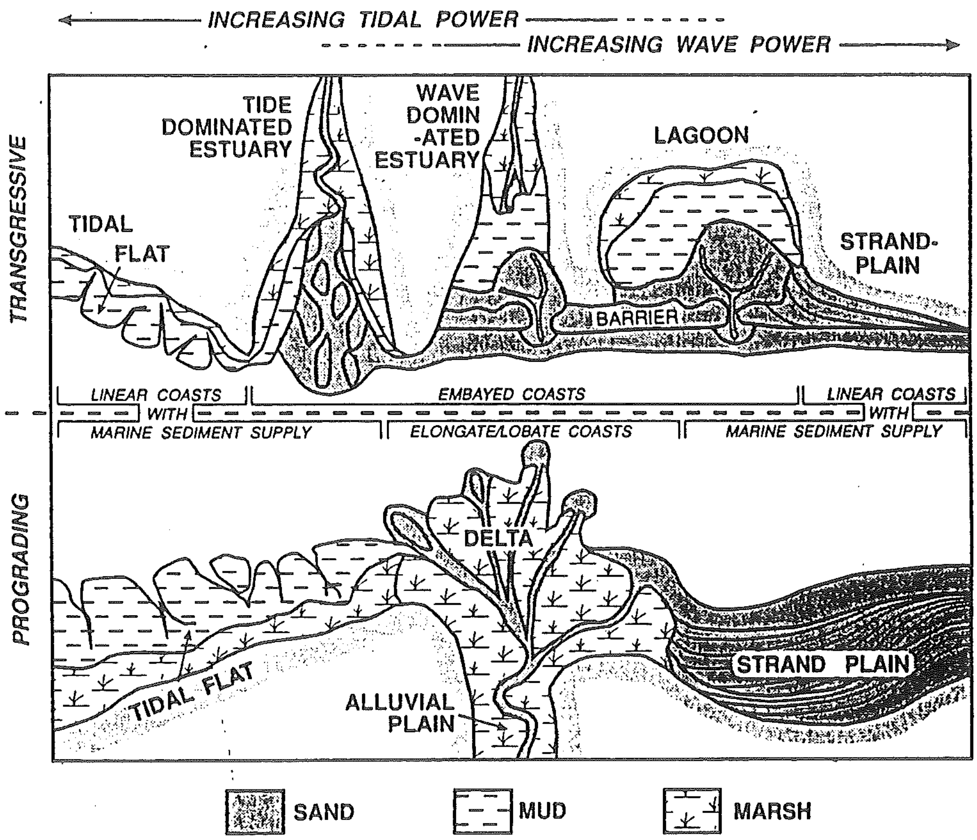


Figure 3.13 Coastal forms for prograding and transgressive coasts (from Boyd et al.,1992)

Figure 3.13 (from Boyd et al.,1992) shows possible types of shorelines depending on four factors: 1. river sediment delivery, 2. (wind) wave power, 3 tidal power and 4. relative sealevel change. In all cases there is a net supply of sediment to the coast, net erosion is not considered here but can of course occur, e.g. when a river is dammed off and river sediment supply comes to a halt while there is no source of marine sediment supply.
The lower half of the figure represents prograding or regressive situations when the landside is on the winning hand, either because of a relatively falling sea level or because of an excess sediment supply over a sea level rise. The upper half represents a relative

sea level rise exceeding the effect of sediment deposition. The change in sea level is relative, meaning that subsidence of the land with a constant sea level has the same effect.

In the prograding case, deposition of river sediment leads to delta building. When wave power and tidal power are low, the sediment of the river will build a long narrow ("elongate" or "birdfoot") delta. Strong waves with longshore currents tend to stretch the deltacoast parallel to the general orientation of the shoreline, while strong tidal action usually creates patterns perpendicular to the shoreline, see also section 3.2.2. Outside the influence of the river, a strand plain develops when wave action dominates and tidal flats when tidal power prevails. The figure also shows that a strand plain does not contain mud, while tidal flats do. This is again in line with Figure 3.11, as far as wave power is concerned.

In the transgressive case, an estuary is the equivalent of a delta in the prograding case, but now, the sediment supply is not enough to keep pace with the relative sea level rise. The sediment is now no longer merely fluvial, but has also a marine source, since the flood tide or waves bring in sediment from the sea. A lagoon has only a marine sediment source, no river is flowing in.

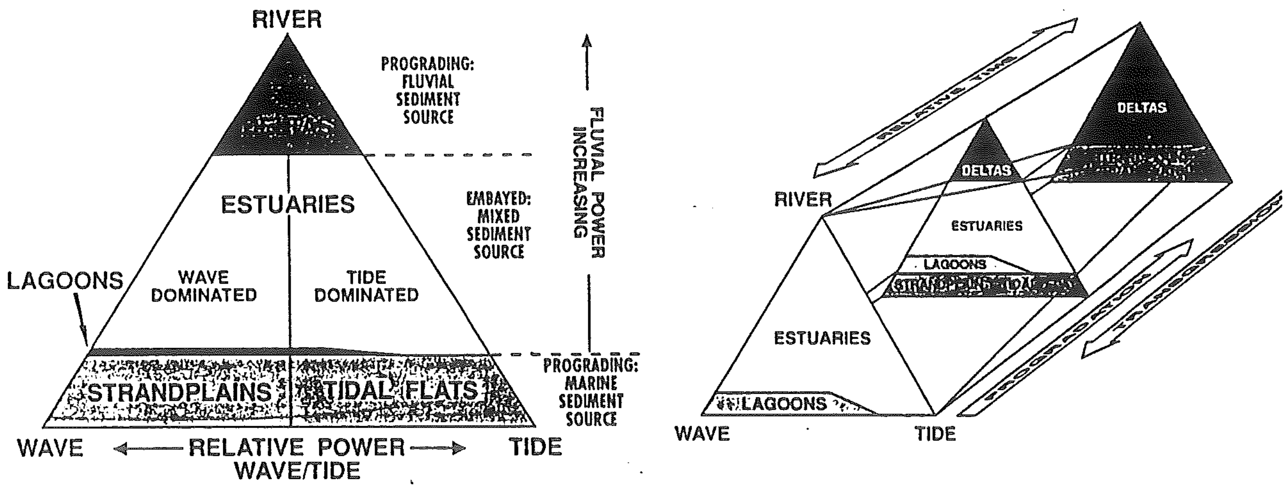


Figure 3.14 Classification of coastal forms as a function of physical features and evolution due to relative sea level and sediment supply (from Boyd et al.,1992)

Based on the mentioned processes, Figure 3.14 gives a classification for all these types of coasts. The ternary diagram on the left presents the fluvial power on the vertical axis and the coastal powers on the horizontal axis, wave power to the left and tidal power to the right. The top of the triangle represents deltas and the bottom strand plains and tidal flats where river influence is negligible, with estuaries in between. Lagoons form, in this diagram, the end member of the estuary spectrum. The right hand side of the figure gives a possible idea about the evolution in time, relative to the change in sea level and sediment supply. With a rising sea level, all deltas change into estuaries and vice versa. Strand plains and tidal flats vanish and become shelf when the sea level rises.

3.2.2 Deltas and estuaries

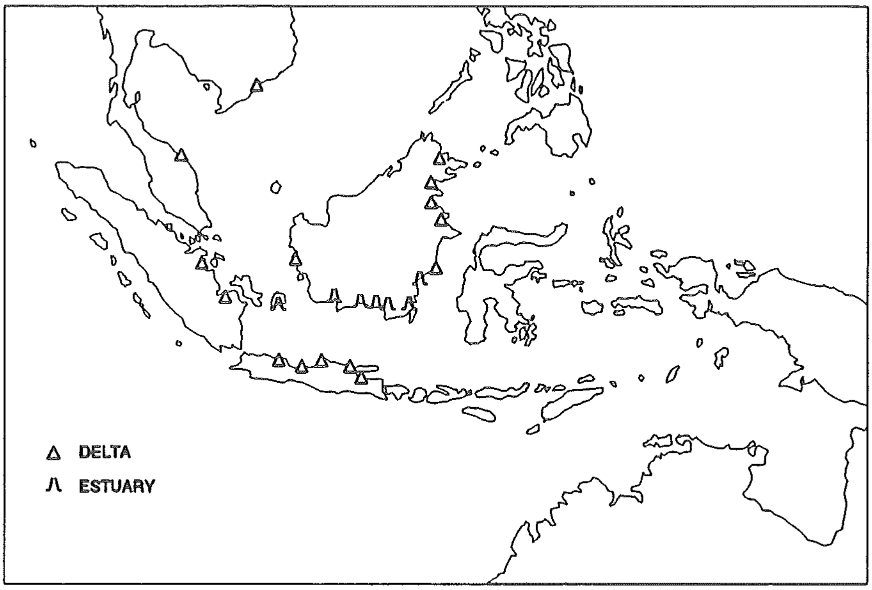


Figure 3.15 Deltas and estuaries in the Indonesian archipelago (from Consentius,1974)

The Indonesian coasts contain several deltas and estuaries, see Figure 3.15. Most rivers show deltas, except those on the south coast of Kalimantan which are mainly estuaries, because of the postglacial sea level rise (Consentius,1974) with which the sediment supply did not completely keep pace. In general, a river mouth is the place where the river delivers its discharge and sediment to the marine environment in which it is, temporarily, deposited on its way to the ocean. On that way, it can be resuspended by waves or currents, reworked by organisms and plays a role in coastal forms and processes.

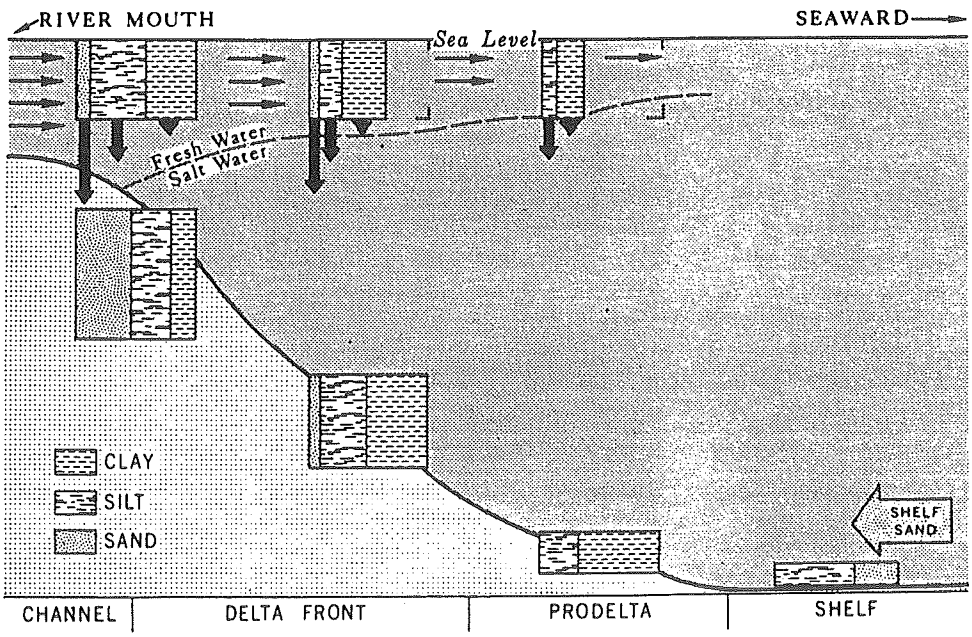


Figure 3.16 General picture of sediment deposition near river mouth (from Coleman,1976)

The general picture of a river mouth is given in Figure 3.16. The density of fresh water proper being 1000 kg/m^3, with typical values of sediment concentration between 1 and 10 kg/m^3, see Figure 3.6, and values for seawater varying between 1020 and 1030 kg/m^3, the river water outflow is usually a buoyant plume. Such a plume spreads over the seawater and the sediment settles, first the heavier sand particles, farther away the finest particles (upper part of Figure 3.16). The composition of the bottom varies accordingly (lower part). Farther away from the river mouth, parallel to the coast, the sand can form ridges along the coast, even in a muddy environment, see section 3.2.5.

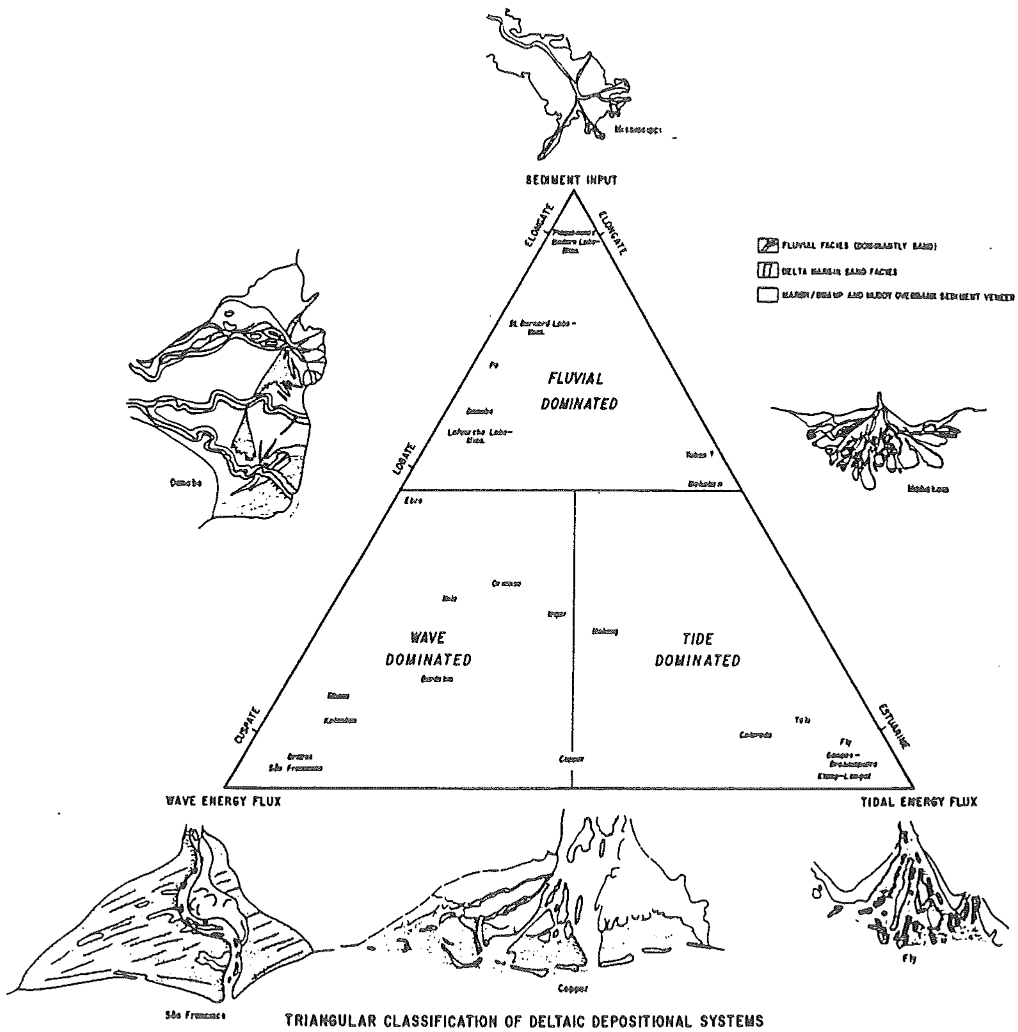
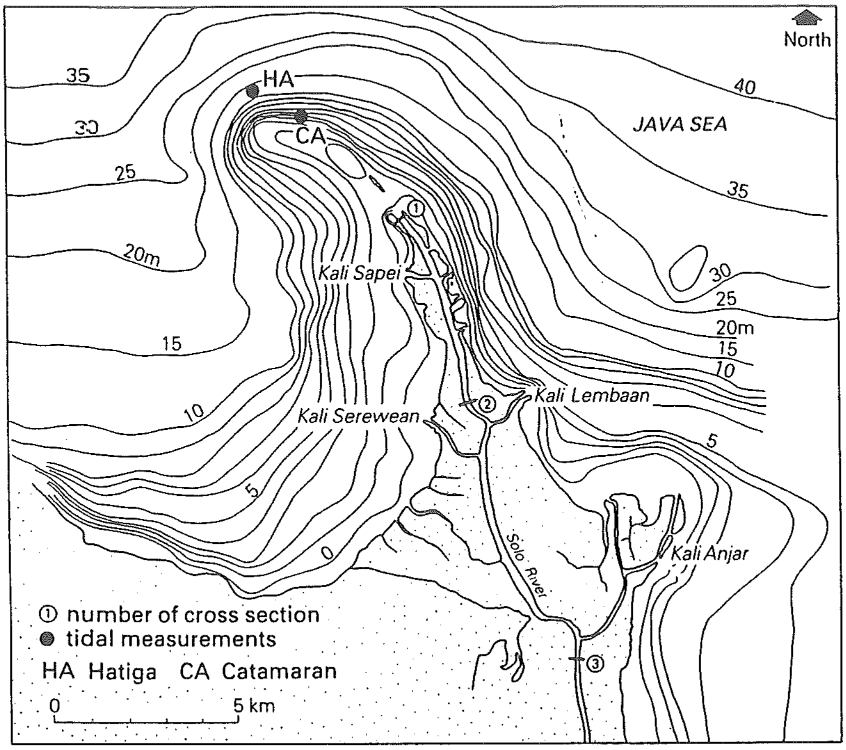


Figure 3.17 Delta classification (from Reineck/Singh,1983)

The classification of deltas is formed by the upper triangle of the ternary coastal processes diagram (Figure 3.17 resp. Figure 3.14). The top end of the triangle is formed by river dominated deltas, showing a very elongate plan view. Wave action tends to stretch the delta parallel to the coast, while tides cause patterns perpendicular to the coast. Due to the strong tidal influence, the difference with estuaries becomes less distinct, so these deltas are even named estuarine deltas. In between the corners of the triangle, all intermediate forms can be found.



The Solo delta,

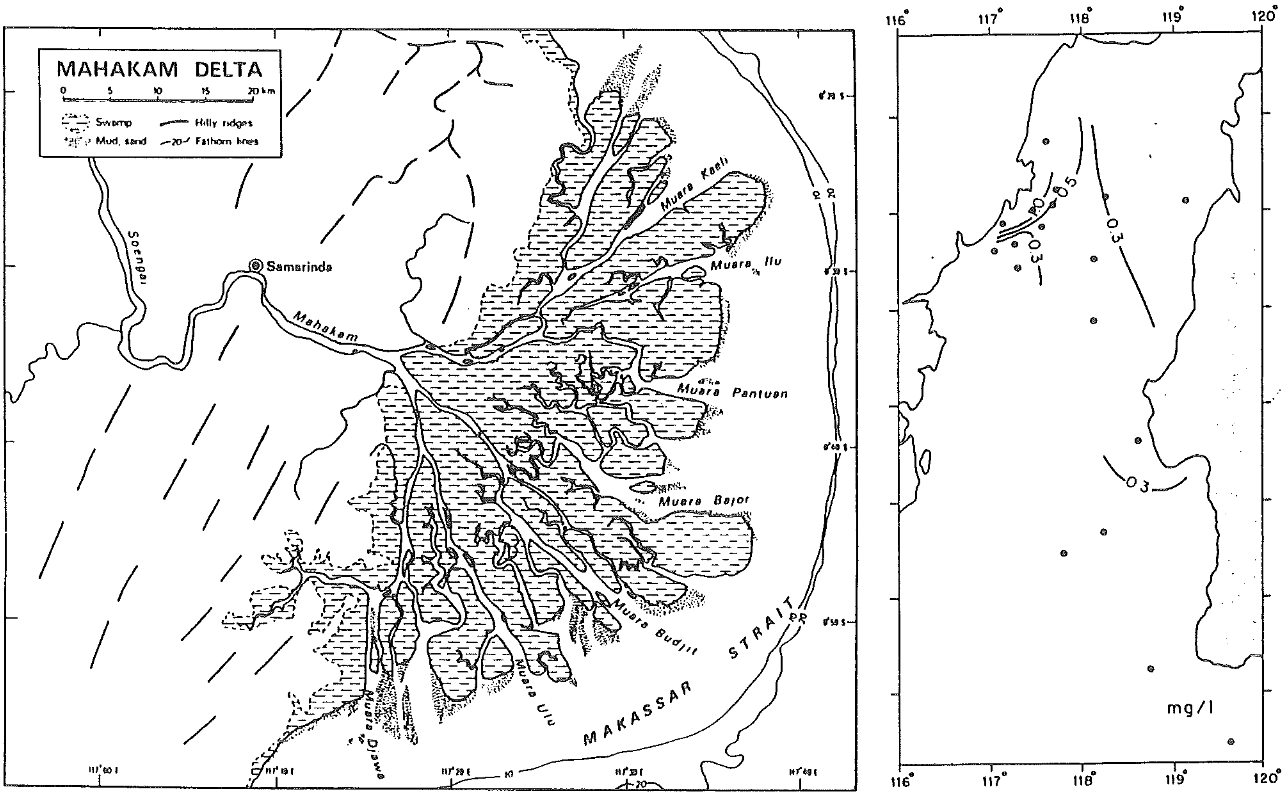


Figure 3.18 Deltas of Solo and Mahakam river

The Indonesian archipelago is characterized by relatively low wave power, so no examples from the left-hand side of the diagram can be expected. Where there is also little tidal power, the elongate river type can be expected, which is indeed the case for the rivers on the North coast of Java, like the delta of the Solo river, see Figure 3.18A. The Mahakam river is a fine example of a mixed fluvial-tidal delta-type. Figure 3.18B shows the Mahakam delta together with the sediment spreading far away, showing that the very fine particles are indeed transported a long distance from the river by tidal and monsoon currents.

3.2.3 *Sandy beaches*

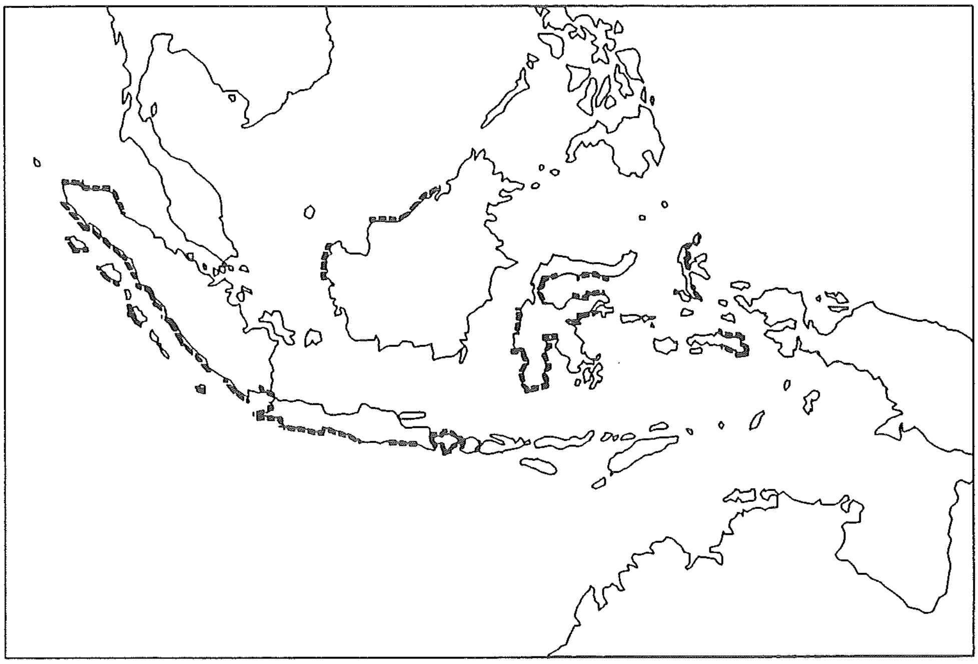


Figure 3.19 Sandy coasts in Indonesia (provisional)

Figure 3.19 gives an overview of sandy beaches in the Indonesian archipelago. The sand on beaches in Indonesia will be mainly of fluvial (terrestrial) origine. Calcareous sands from coral debris will be present near coral reefs, while on beaches of the ocean coasts, some abrasive material from rocks will form the beach material. Finally, reworking of the pleistocene shelf bottom, is a possible source for beach sand. The description of beaches and beach processes is usually done in two directions: perpendicular and parallel to the shoreline (cross-shore and longshore respectively), although together they form one process.

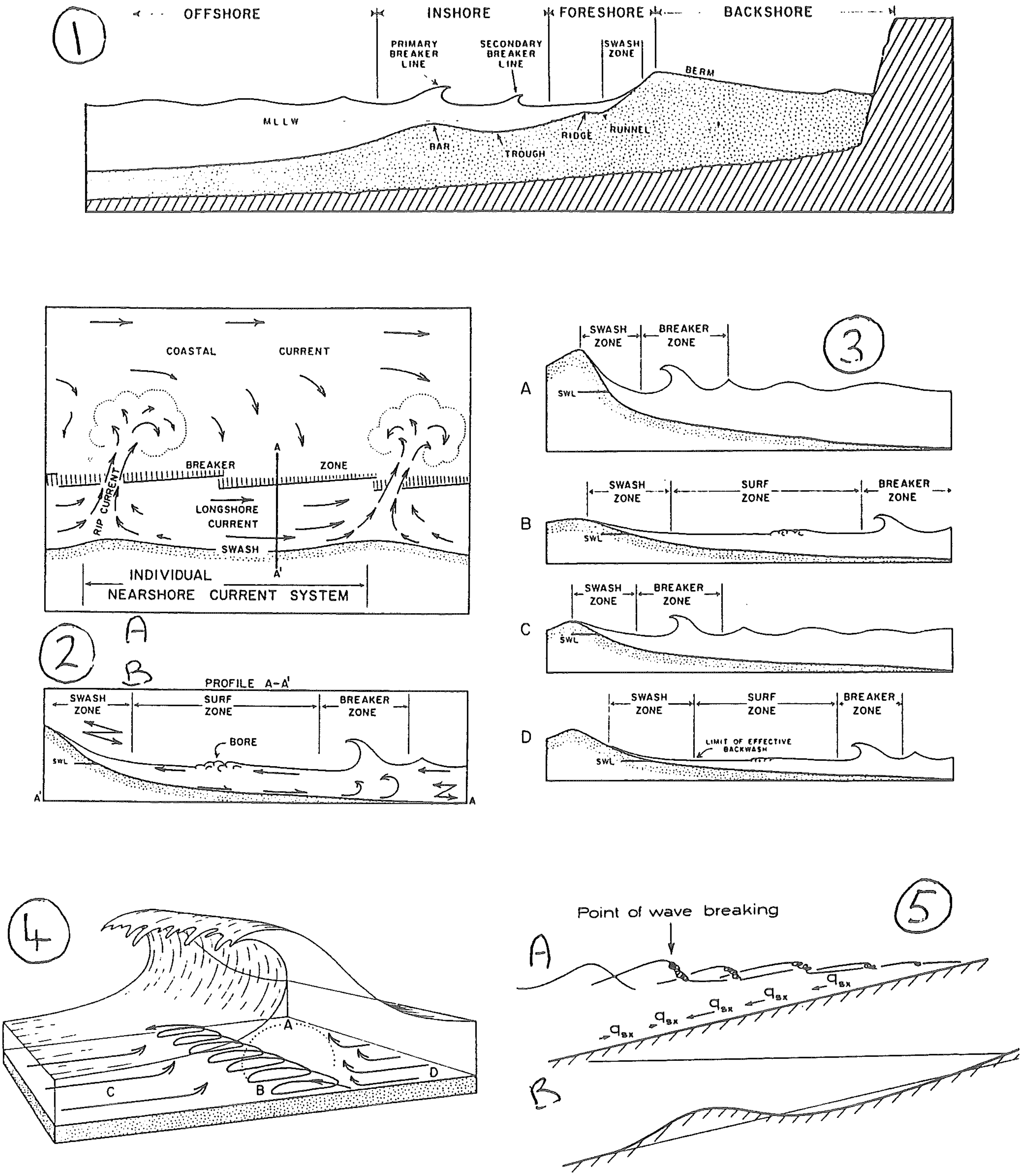


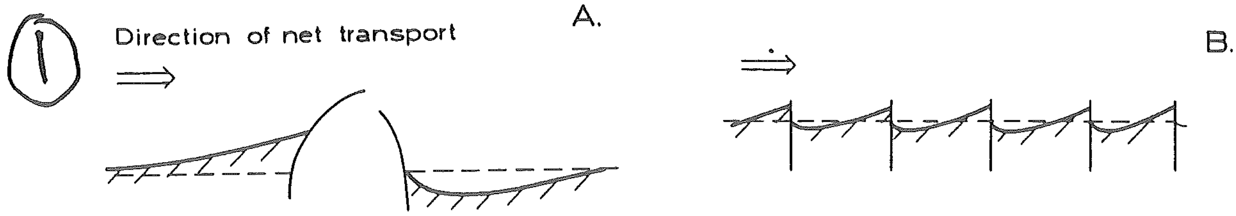
Figure 3.20 Cross-shore definitions and processes of sandy beaches

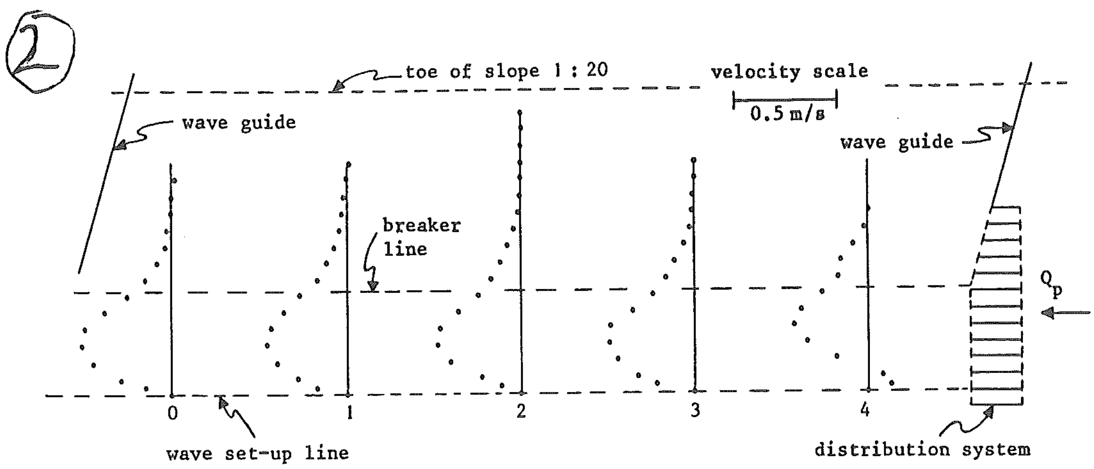
Figure 3.20 shows some cross-shore features of a sandy beach. Part 1 gives topographic features of the beach environment. It should be noted that ridges and runnels are exposed during low tide whereas bars and troughs are continually submerged. The boundary between the foreshore and inshore areas is arbitrarily placed at the mean lower low-water line. Terminology of nearshore current systems is presented in part 2. 2B shows the three principal dynamic zones of the beach environment (SWL = still water line): the swash zone, where the wave run-up and the backwash on the beach occur (the beach is alternately dry and wet due to the waves), the breaker zone where the waves become unstable and break and the area in between, the surf zone, where broken waves travel on as bores (comparable with traveling hydraulic jumps). Part 2A gives the link with the longshore features. In the surf zone, there is a longshore current due to radiation stress parallel to the coast, while the coastal current is due to tides or to wind. Radiation stress perpendicular to the coast causes wave set-up, which, together with the longshore current, discharges through rip currents back to the sea at more or less regular intervals along the shore. Radiation stress and set-up will be dealt with in more detail in the next chapter.

Several investigators have pointed out that the cross-shore beach profile roughly has a parabolic shape (Bruun,1954, Dean,1981), which can be derived from the assumption of equal energy dissipation per m^2. In chapter 4 this will be treated in more detail. Existence and width of a surf zone is governed by beach slope and tidal phase, see part 3 of Figure 3.20. Steep-sloped beaches seldom possess a surf zone as relatively deep water allows waves to break close to shore and thus the swash zone meets the breaker zone (3A). Gentle beaches possess a surf zone under almost all conditions as waves must break at some distance from the seaward edge of the swash zone (3B). Moderately sloping beaches commonly lack a surf zone during high tide (3C), but exhibit one during all other phases of the tidal cycle (3D). The net flow in the surf zone is shoreward at the surface and seaward at the bottom, see part 2B and 4C, while offshore from the breakerzone, the net flow at the bottom is shoreward, see 4D, due to the assymetrical orbital motion in the waves. The result is a net sediment flow directed to the breaker, causing a bar and a trough, see part 5A and 5B, although also other mechanisms will play a role in bar formation.

Longshore currents also cause sediment transport, usually indicated as littoral drift, which becomes manifest through coastal erosion or accretion around structures, see Figure 3.21, part 1. 1A represents erosion and accretion around a harbour (net littoral drift is from left to right) and 1B a field of groynes, which has been built to stabilize an eroding coast. Longshore current is wave driven by means of radiation stress and can be seen as a "river" along the coast. Figure 3.21 part 2 gives measurements of longshore currents in a laboratory situation (from Visser,1991).

Where the littoral drift passes a headland or comes in a bay or lagoon, a so-called spit can be formed, see Figure 3.21 part 3 (from Davies,1985). This figure also shows that there is a continuum from lagoons to estuaries and deltas. See also Figure 3.14.





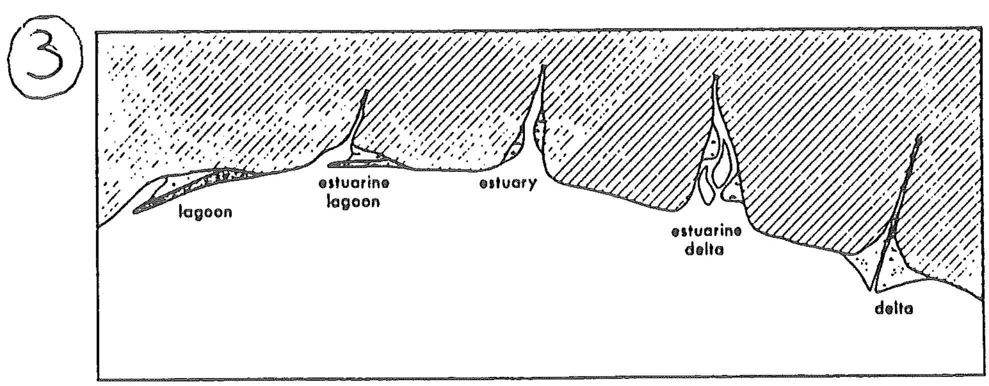


Figure 3.21 Longshore coastal features and transport

3.2.4 Coral reefs

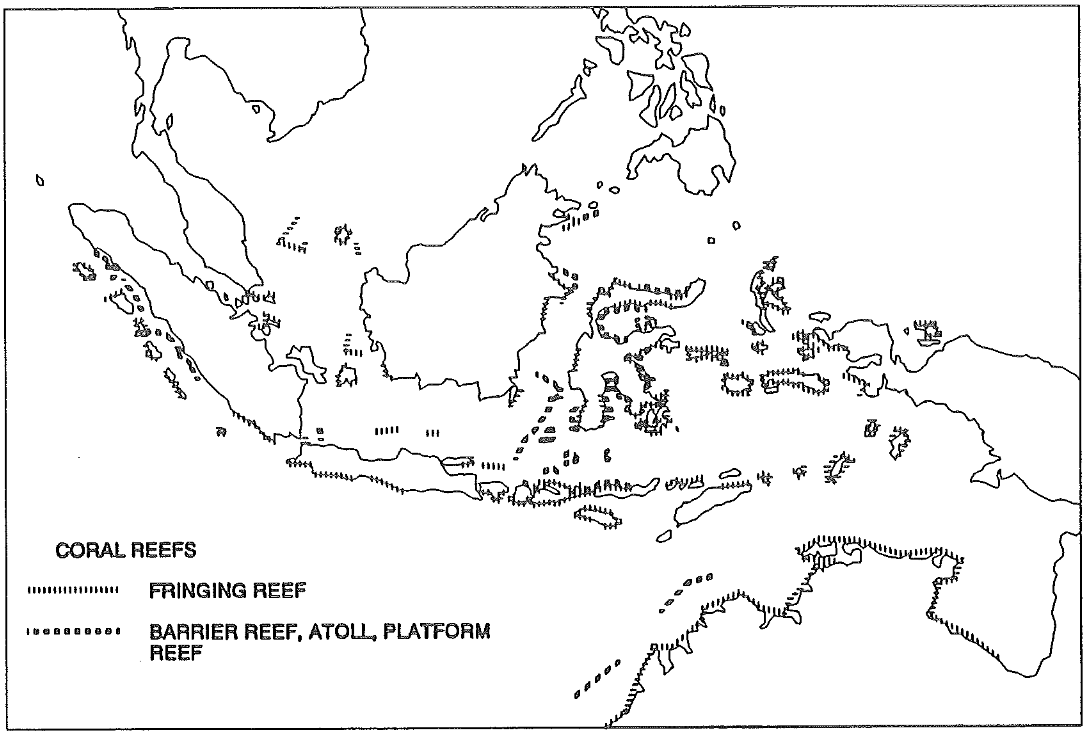


Figure 3.22 Coral reefs in the Indonesian archipelago (from Consentius,1974 and Bird,1983)

Figure 3.22 gives a (again provisional) overview of coral reefs in Indonesia. Coral reefs are skeletal structures and are built, from calcium carbonate out of seawater, by coral polyps. Closely associated are calcareous algae, growing in and around coral structures. The algae draw nutrients from the coral and turn the carbon dioxide from the polyps respiration again into oxygen by photosynthesis. The coral is essential for reef building, but forms only a small part of the reef material. Algae and all kinds of shelly organisms and sedimentary fragments of shell and coral, fill in the skeletal coral structure to form a calcareous mass, which eventually becomes a massive reef limestone. Even after the coral is dead, the reef can remain intact. Growth of reefs starts where free floating planktonic coral larvae and algae find favourable conditions. In very clear water coral can grow up to a depth of about 50 m. On top of the coral, the algae can grow even higher, building up a slightly higher reef crest. For more detail see e.g. Bird,1983 and Fairbridge,1968. Factors influencing reef development are (Bird,1983):

- Watertemperature: must be higher than 20° C
- Visibility: sunlight, hence clear water, is essential for algal photosynthesisthe
- Salinity: not too fresh and not too salt, between 27 and 38 ppt
- Circulation: supply of nutrients and oxygen (specially at night when photosynthesis does not work) is essential for healthy reefs
- Level: corals cannot survive prolonged exposure to the atmosphere, they die off from about a foot above low water spring level

From these demands it can be concluded that reefs will usually not be found near large river outflows, because of salinity and visibility (turbidity from sediment load), which is also the reason why most coral reefs are being found near islands and much less along continents, where large rivers debouche. The Lesser Sunda islands, where rainfall is much less than in the rest of Indonesia, have rather ideal conditions for coral growth.

Where the slopes of reefs are exposed to strong wave action, there is ample supply of oxygen and nutrients, but parts of the skeleton are being broken off. On the leeward side, in sheltered water, coral growth is more varied and coral debris from the windward side piles up, as sand and gravel to form islands much larger than the reef proper.

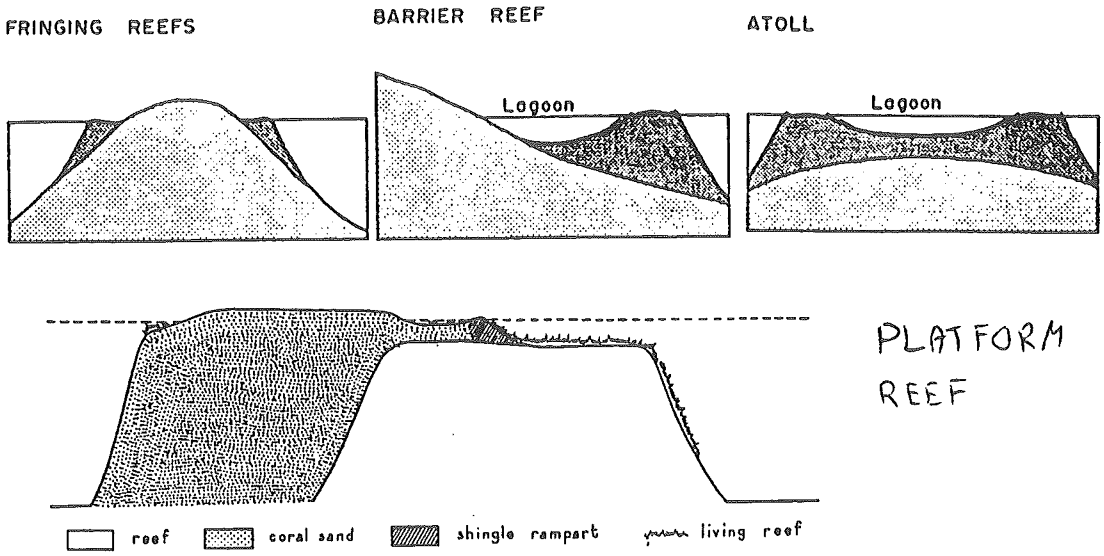


Figure 3.23 Reef landform types (from Bird,1983 and Verstappen,1953)

Figure 3.23 shows various types of reefs. The simplest reef forms are fringing reefs, built upwards and outwards in the shallow seas that border continental or island shores. Offshore reefs include barrier reefs, atolls and platform reefs. These forms probably have to do with sea level rise. Starting in shallow water, the corals can grow on, on their own debris, even when the original bottom has become very deep. The nucleus of the islands of Pulau Seribu are platform reefs, with relatively large amounts of calcareous sand and gravel at the leeward side of the reef.

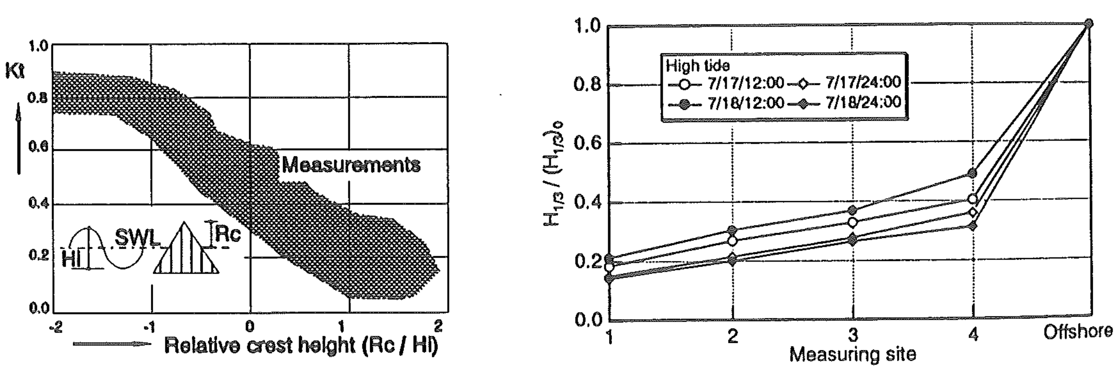


Figure 3.24 Wave reduction of (single) reef structure (from van der Meer and d'Angremond,1991) and measured reduction on coral reef (from Sulaiman et al., 1994)

From a coastal engineering point of view, coral reefs are important, since they form a natural protection, either as a "revetment" (fringing reefs) or as an offshore breakwater (barrier reefs) in front of sandy beaches, a situation that often occurs along Indonesian coral coasts, like on the Lesser Sunda islands. The wave reduction of a single, sharp crested reef is given in Figure 3.24A. On an average the wave reduction is only about 50 % when the waterlevel is around the reef's crest. But a barrier reef can be seen as a series of single reefs or as a very broad crested reef, giving very effective wave damping. Figure 3.24B shows measured wave reduction on a coral reef in Bali, which lies around 80%. Another effect of a reef will be a rather intense wave refraction, making waves approach a beach behind the reef more perpendicular, thus reducing littoral drift and possibly erosion.

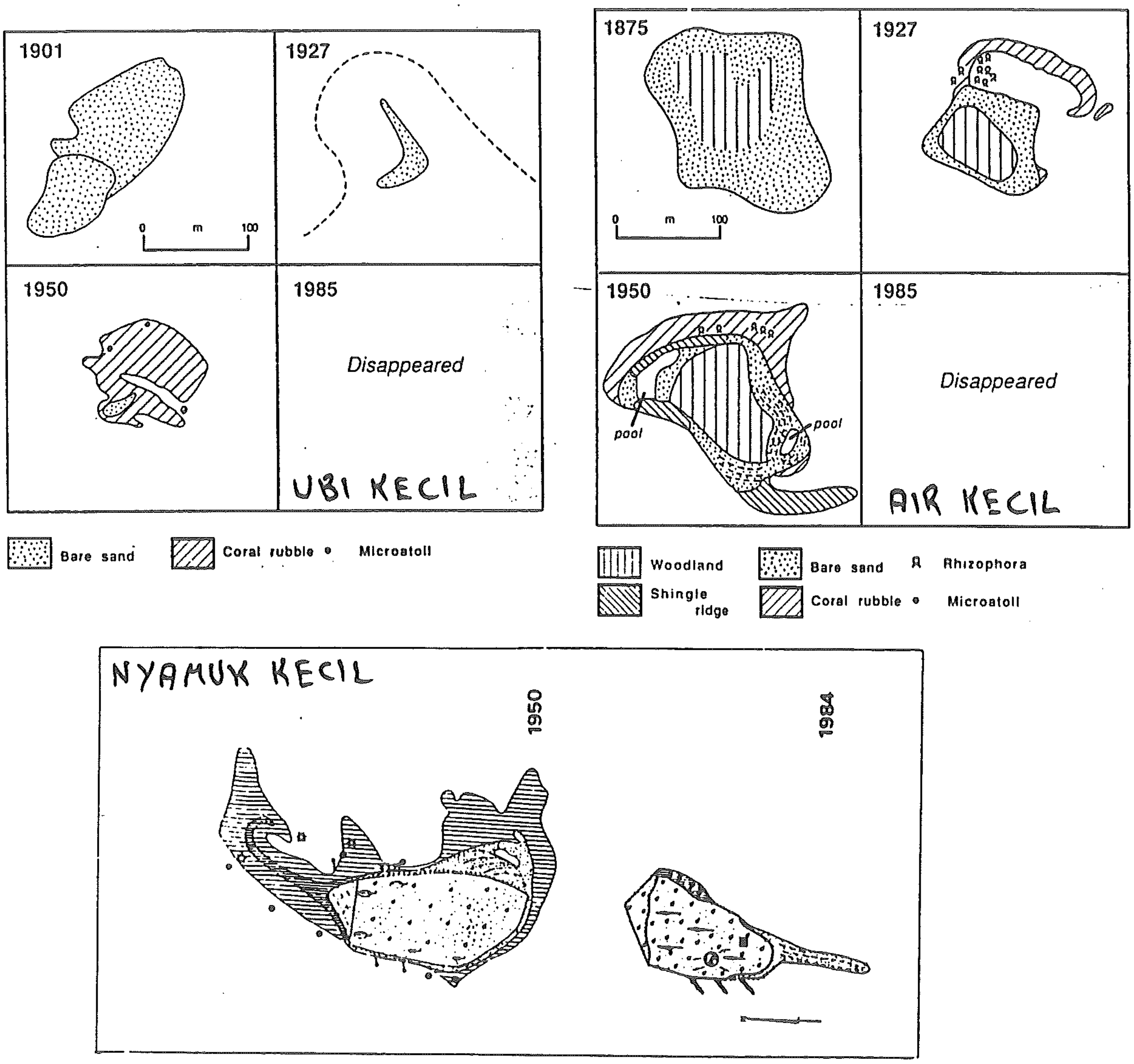


Figure 3.25 Vanished platform reefs and connected islands of the Pulau Seribu

Pollution (including turbidity) and coral and sand mining have deteriorated some of the Pulau Seribu, some of them are even completely vanished, see e.g. Soekarno,1989 and Verstappen,1988 (see Figure 3.25). Along the coast of Bali, severe beach erosion has occurred at places where coral reefs have been taken away for cement production.

3.2.5 *Mud coasts*

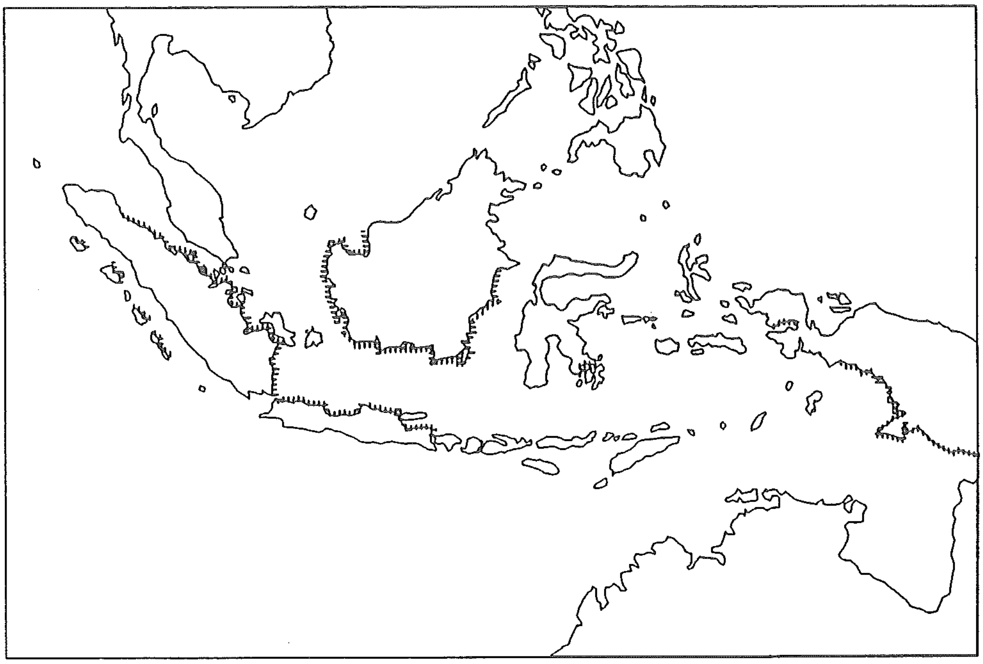


Figure 3.26 Mud coasts in Indonesia (provisional)

Figure 3.26 shows the Indonesian coasts that consist mainly of mud. In the introducing sections on coastlines (3.1.4 and 3.2.1) it was already mentioned that mud coasts are related to relatively low wave energy, but tidal power can be high. Together with the superfluous supply of fine sediments from the rivers, this makes the East coast of Sumatera, the North coast of Java, the South coast of Kalimantan and the South coast of Irian Jaya locations where mud coasts can florish. The behaviour of mud on coasts is very different from sand. The much smaller size of the particles not only gives lower settling velocities and lower critical velocities, but also leads to other properties as cohesion and consolidation and fluid mud layers, where the difference between water and soil is not always very clear.

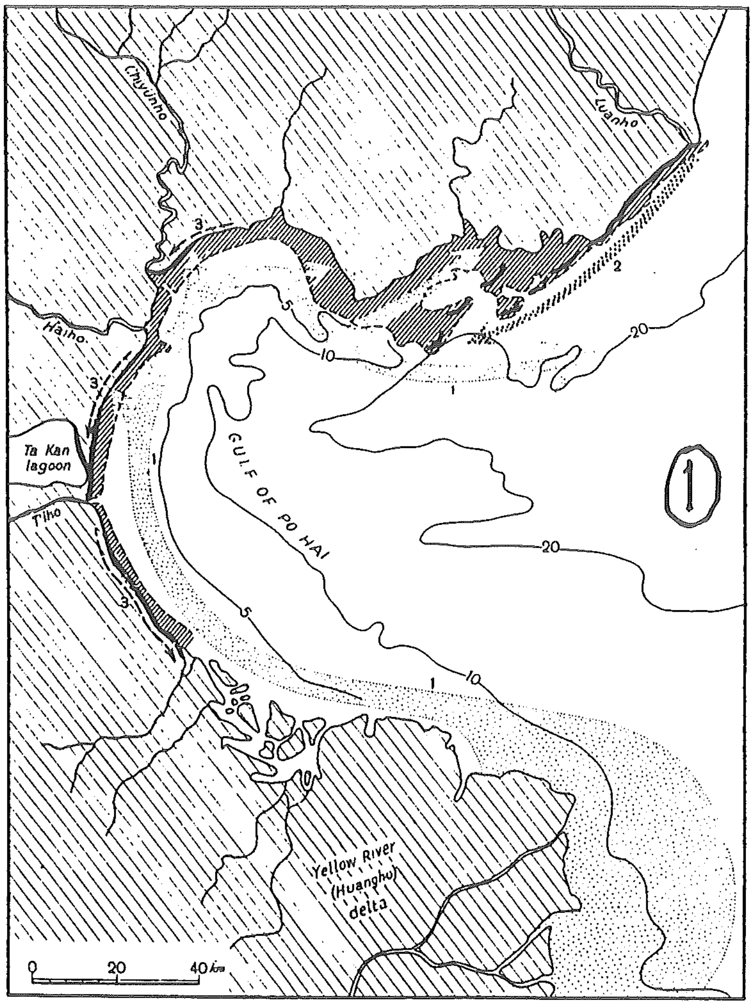
Mud coasts are usually prograding coasts. Van Bemmelen,1949 reports advances on the East coast of Sumatera of more than 30m/year. It is believed that in the Sriwijaya era (approximately 700 - 1400), Palembang and Jambi, now more than 100 km from the coast, were true seaports. Also Java's North coast generally progrades, see Verstappen,1953. This does not mean that coastal erosion never poses a problem on these coasts. Locally and periodically, considerable shifts in the coastline are possible, due to the dynamic character of the conditions. Transgression and progradation alternate sometimes, leading to so-called cheniers, to be discussed later on in this section.

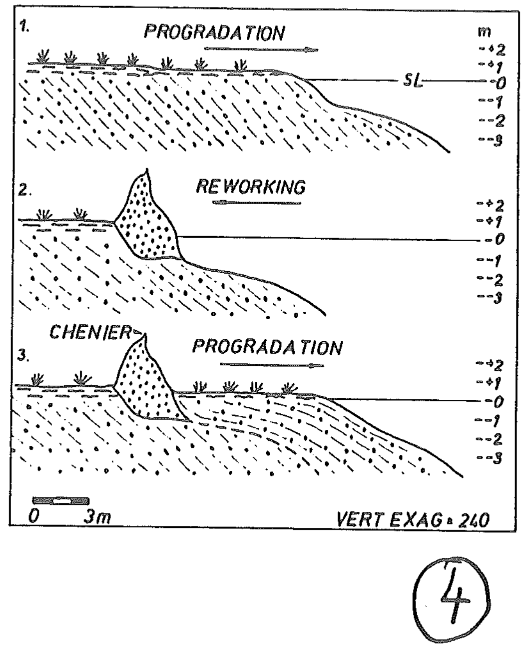
Also human activities can cause problems along mud coasts. Pumping of fresh water from the coastal area will lead to subsidence of the soil when it consists mainly of clay (sand is hardly sensitive to this type of subsidence). This subsidence will subsequently lead to inundation or erosion of the coast.

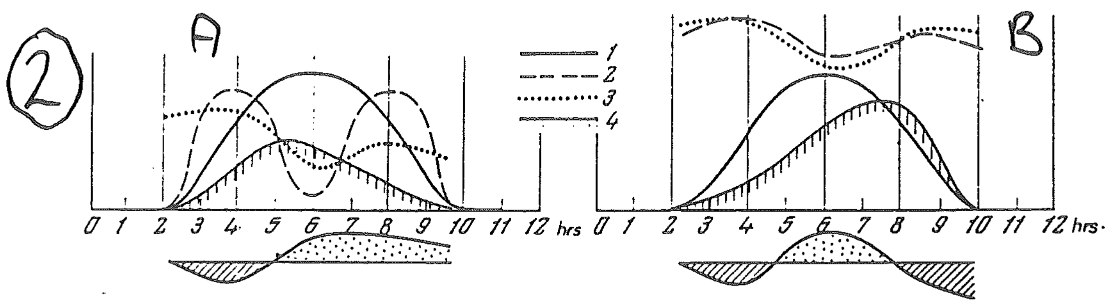
Mud is very mobile and the transport differs completely from the transport of sand. It can be transported by the water in suspension or as a fluid mud layers. On sloping bottoms, fluid mud layers can even move independently due to gravity. This is e.g. the case in the mouth of the Yellow River, which contains so much sediment that the plume of the river outflow has a negative buoyancy and the river sediment moves as an avalanche into the sea. Long distance transport, however, is always in the form of suspension. Although waves can contribute to the stirring up of mud, the transport is dominated by tidal or wind driven currents. Figure 3.27, part 1, shows the transportation of mud and sand in the Gulf of Po Hai in the Yellow Sea (from Zenkovich,1967). The sand from the Luanho river in the North moves, wave driven, in a Southwesterly direction, while the mud flow from the Yellow River is transported Northerly by tidal currents. The mud is trapped along the coast, specially by the sand spits and ridges from the Luanho. Mud can be transported quite long distances. E.g. the mud along the North coast of South America (Guyana and Surinam) comes from the Amazon river, which debouches more than 1000 km away.

As said before, the mud coming from rivers is distributed along the coast by tidal currents. Zenkovich,1967 ascribes the progradation of mud on tidal flats to three factors. On these flats the rise of the tide is usually shorter than the fall, with a corresponding difference in current speed between flow and ebb. Therefore, the flood tide normally carries more material on to the flats than can be carried away. Secondly, the velocity needed to put silt particles into suspension is considerably higher than the velocity at which material is transported without settling. Therefore, only part of the material brought by the flood and deposited may be carried back again by the ebb. Thirdly, on the higher parts of the flats the flood current will cover the whole area of the flat, while the ebb will be concentrated in channels, due to percolation and seepage from the surface. Figure 3.27, part 2A, gives a sketch of the progradation of a tidal mud flat under normal circumstances (calm weather).

Consider a point somewhere at mid level on a tidal flat, which is covered for several hours during a tidal cycle. The waterlevel rises and falls and the velocity does so twice in that period, mainly coupled to the slope of the waterlevel in the time curve. In the figure, not the velocity but the "mobility" of the "bottom water" is used. This also includes (small) wave action. The water coming up on the flat already contains sediment, slightly increasing because of erosion on the flat slope. When the rise of the waterlevel halts and the mobility decreases, particles will settle, giving accumulation of sediment. The total content of suspended matter increases for some hours as a result both of bottom erosion and of the increase in the amount of water bringing suspenden matter from the sea. With the ebb, the phenomena are repeated in reverse, but the amount of suspended material is slightly less, due to the reasons given above, leading to an asymmetrical curve for the total amount of sediment in a water column, with a peak to the left of the high water mark and the surface is slightly raised after the water has receded.







1. Curve of the variation of level and alteration of depth above a given point.
2. Curve of the mobility of bottom waters (currents and waves).
3. Content of suspended matter in bottom layer.
4. Total amount of suspended matter in a water column of varying height.

The rate of erosion (hatched shading) or accumulation (dots) is shown

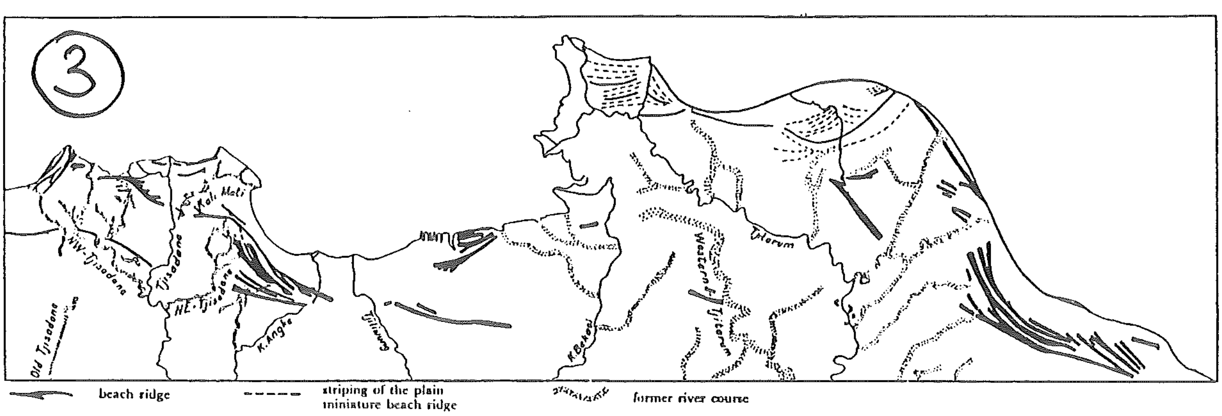


Figure 3.27 Mud coast features

During periods with more wave activity, the situation is completely different, see part 2B of Figure 3.27. Water reaching the mud flat is already completely saturated with suspended material and remains almost saturated also during ebb. Deposition will hardly take place and the curve of the total amount of sediment now has a peak to the right of the high water mark and the bottom is lowered. Mud flats therefore, cannot be treated as stable, except in almost completely closed areas of water. They only form and continue to exist when they receive new material constantly. The same mass of mud is repeatedly stirred up and carried from place to place, thus leading to temporary erosion and accretion in different sectors.

Another periodicity, related to the balance of power between river sediment supply and wave action can cause a pattern of mud flats or marshes alternated with sand ridges. These are clearly visible e.g. on Java's North coast, see Figure 3.27, part 3 (from Verstappen,1953) and probably also explain why sand beaches can exist occasionally at some locations along a mud coast. These ridges are probably "cheniers". The word chenier comes from the French word for oak and was used for the sand ridges in the Mississipi delta area where oak trees were growing. In tropical areas these ridges are often occupied by coconut trees which live on lenses of fresh water, concentrated in the relatively high ridges, while salt loving trees, like mangroves can grow on the marsh seaward of the ridges. Figure 3.27, part 4 (from Reineck and Singh,1980) shows the development of a chenier coast. Their dimensions (in the USA) are about 150-200 m broad, 3 m high and up to 50 km long. According to Reineck and Singh,1980 they owe their origin to a variable sediment supply. When the nearby rivers, bringing fine-grained sediment, reduce their sediment supply, waves rework the existing sediment of the previously prograding marsh and a beach with a sandy ridge is formed (possibly with sand from the sea bottom, transported to the shore due to the asymmetrical orbital motion outside the breaker zone, see the section on sandy beaches). As soon as the supply of fine sediment increases again, a muddy marshy zone is build up in front of the sandy ridge.

There is still a lot to be learned, however, on this process. According to Verstappen,1953, the sand on these ridges also has a fluviatile origin, since they are broadest at river mouths, becoming narrower with increasing distance. Sorting of sand and mud outflow from rivers could be related to the differences in fall velocity and longshore transport mechanism. Mud will settle and will be transported over a much wider belt than sand. Augustinus,1982 reports on the chenier coast of Surinam where sand supply for a chenier can be washed out of a muddy sediment deposition or comes from river sediment supply. Further studies will have to clarify the situation in Indonesia.

******** This will be an important topic in the final version of the Pedoman. Chenier type coasts, or any other form of alternating sand and mud beaches, are complicated and hence, their behaviour is difficult to understand and hence, engineering on this type of coast will need more knowledge and insight of the phenomena. ********

3.2.6 Mangroves

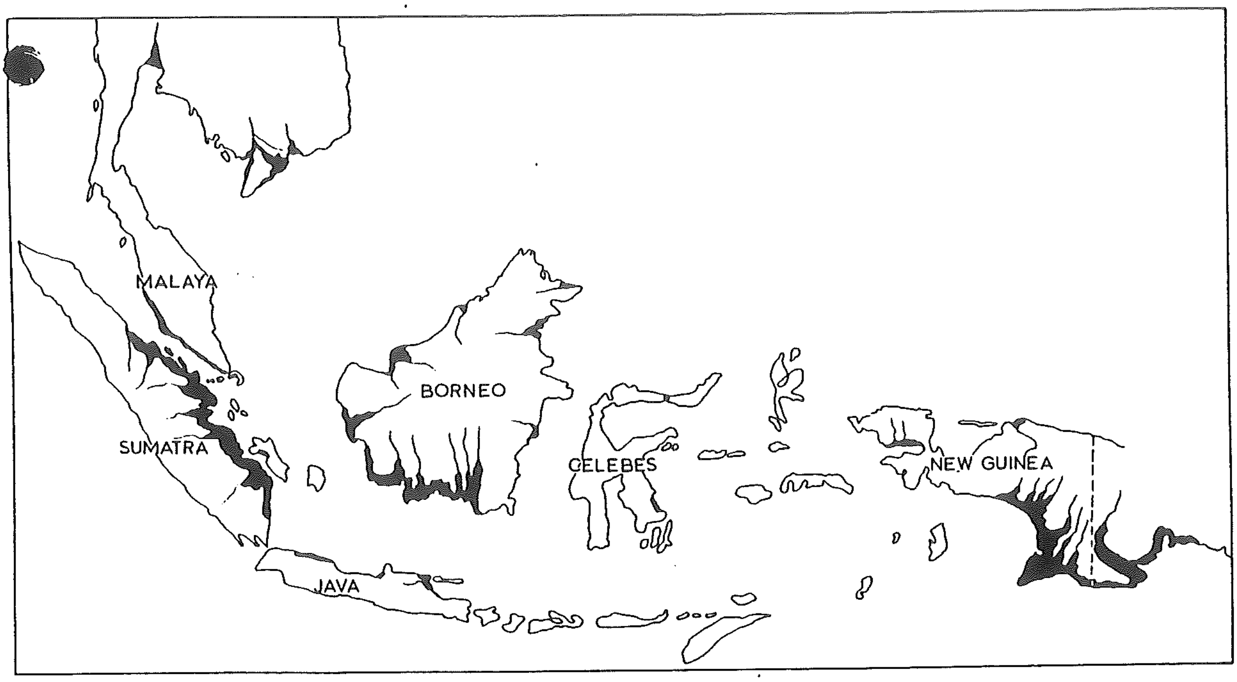


Figure 3.28 Mangrove forests in the Indonesian archipelago

Although mangrove trees are can grow on many soiltypes, they are most abundant along mud coasts, see Figure 3.28. Mangrove forests are the natural vegetation of many tropical coasts and tidal inlets and form a higly productive ecosystem, a maternity and nursery for many marine species. Living in very dynamic circumstances, mangrove trees are real miracles in surviving. They can cope with salt water where most other plants cannot. Seedlings have little opportunity to settle, so mangroves are viviparous, giving birth to an almost complete tree in a capsule (the propagule) that can travel with the tide and can turn into an upright standing young tree within a few days.

There is an increasing awareness of the importance and vulnerability of mangrove forests. These forests also play a role as a natural coastal protection and at places where they are removed for whatever reason, erosion and/or artificial protection is the price to be paid. Mangroves can only exist on coasts with a moderate wave climate. This is mainly caused by the fact that the seedlings cannot settle in highly dynamic conditions. Once the trees are grown up, however, they can even stand an occasional cyclone.

One of the most striking visible features of mangroves is the root system. Because of the anaerobic conditions in the mud soil on which mangroves usually live, they improve their gas exchange with the atmosphere by means of aerial roots. Of the many mangrove species in the world, the two most important are Avicennia and Rhizophora. Some authors report on Rhizophora mangrove forests with a seaward Avicennia fringe, others the other way around. These species have completely different aerial roots. Rhizopora works with "prop"-roots or "stilt"-roots, while Avicennnia works with "snorkel"-type pneumatophores, emerging vertically from the bottom, see Figure 3.29. These roots play an important role in wave damping, probably even more than the trunks of the trees.

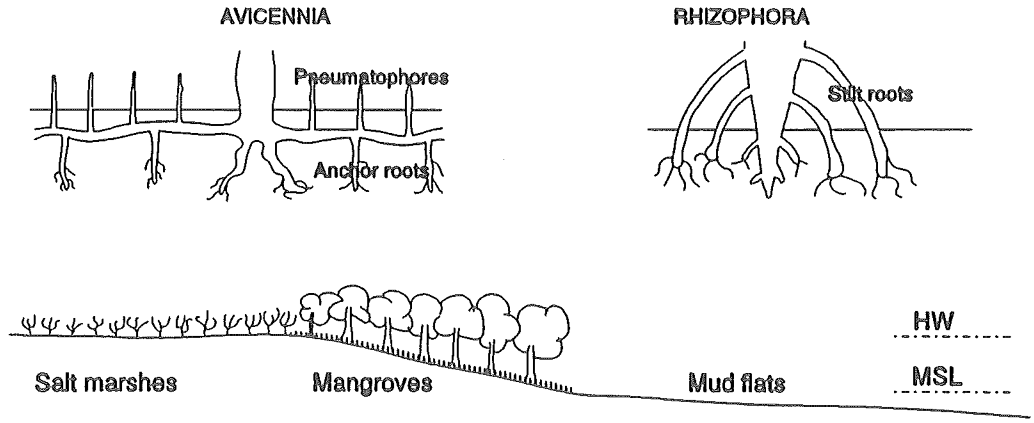


Figure 3.29 Mangrove roots and typical cross-section of mangal

Because of this breathing system, mangrove trees need fresh air regularly and for this reason they can live only in the upper tidal zone, approximately above Mean Sea Level up to High Water (Spring). They can stand storm surges, but after extremely long periods of flooding, drowning of mangroves has been reported or even suffocating when large quantities of sediment have covered the root systems during a storm. Below MSL, the seedlings cannot settle and at higher levels, the mangroves cannot compete with other plant species. The coastline in front of mangroves often consist of mud flats, possibly covered with some pioneer seagrass vegetation. Behind the mangroves, between HWS and the level of occasional flooding by the ocean, salt marshes can be found with halophytic (salt-loving) herbs and grasses, before the "normal" vegetation starts, see Figure 3.29. The width of these tidal forests is mainly determined by the tidal range. The vegetation itself probably determines the slope of the bottom to a high degree.

Figure 3.30 gives the computational results for the wave transmission for a mangrove belt of 100 m wide, from Schiereck and Booij,1995. Wave damping is quite effective at small waterdepths, but with high waterlevels, most wave energy is transmitted (the possible influence of branches and leaves has not been taken into account). This would mean that mangroves can have a significant influence on sedimentation during a large part of the tidal cycle, but are not very effective as breakwater. The latter seems in line with the findings of e.g. Sauer,1962 and Jennings and Coventry,1973 who report severe damage of vegetation behind mangroves, while the mangroves themselves were not or hardly damaged.

Among biologists and ecologists there has been much discussion whether mangroves are able to act as "land-builders". E.g. Davis,1940 states that the seedlings can settle on the mud flats in front of the mangroves, giving a raise of the bottom level by sedimentation and organic waste material. Others, like Scholl,1968, say that mangroves simply follow the changes in coastal morphology. Once the bottom level is high enough, mangroves will settle. Bird,1972 says that mangroves need a certain bottom level to settle, but after that they are able to raise the bottom level. This could be caused by the wave damping, creating an environment in which sediment can more easily settle.

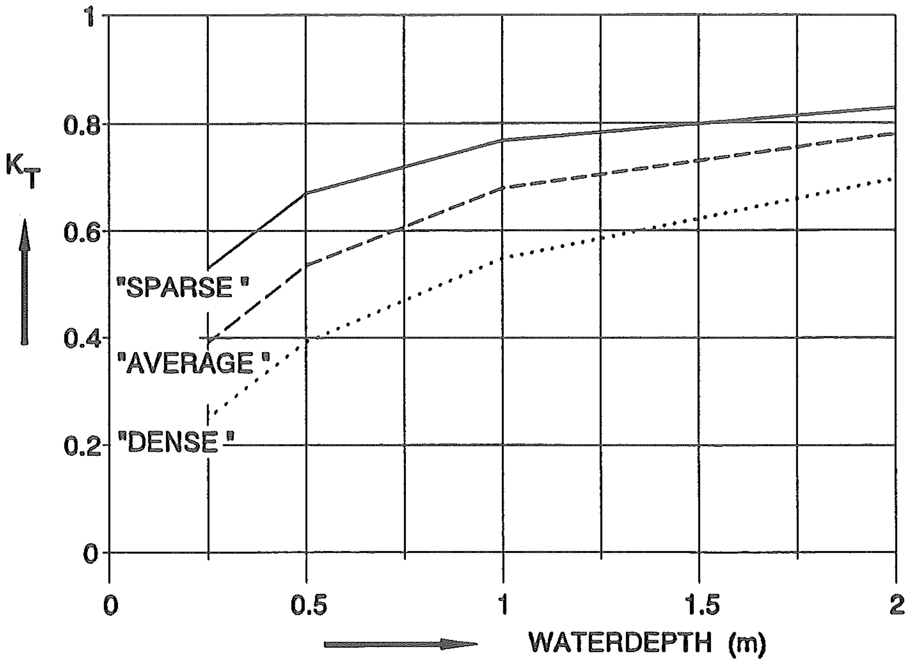


Figure 3.30 Wave transmission through 3 possible mangrove forests (100 m wide)

An assumption often made with sandy beaches is the idea of equal energy dissipation per unit area as proposed by Bruun,1954. The idea is that, regardless of the process of sand movement perpendicular to the coast line, the breaking waves will "allow" a certain depth such that the energy dissipation is equal at every location. This leads to a parabolic curve for the beach profile. Outside the breakerzone things are more complicated, since there the process is no longer dominated by the violence of the breaking, but by a more subtle combination of velocity, boundary layer and fall velocity of the sand. On a mud coast, however, the sediment is so fine that it is not impossible that the assumption of equal energy dissipation is also valid outside the breaker zone. The sediment is getting into suspension quite easily and the wave energy dissipation again determines the waterdepth. (On unconsolidated mud coasts a completely different mechanism can be present. The waves can attenuate into the muddy bottom and wave energy is dissipated by internal work in the mud layer. This has not been taken into account here.)

From wave energy calculations, it appears that the assumption of equal energy dissipation leads for bare mud flats to slopes of 1:1000 to 1:2000 and for mangrove belts to slopes of 1:100 to 1:200, see Schiereck and Booij,1995. This is in line with what was found on Australian mud flats and mangals. Taking these figures, this would mean that cutting mangroves would lead to a much flatter slope, hence to considerable coastal erosion. Again, additional research will be necessary to ascertain these figures for the Indonesian situation.

3.2.7 *Rocky coasts*

******** These pages are a reprint from King,1966. Depending on the needs in Indonesia, a tailored text should be drafted ********

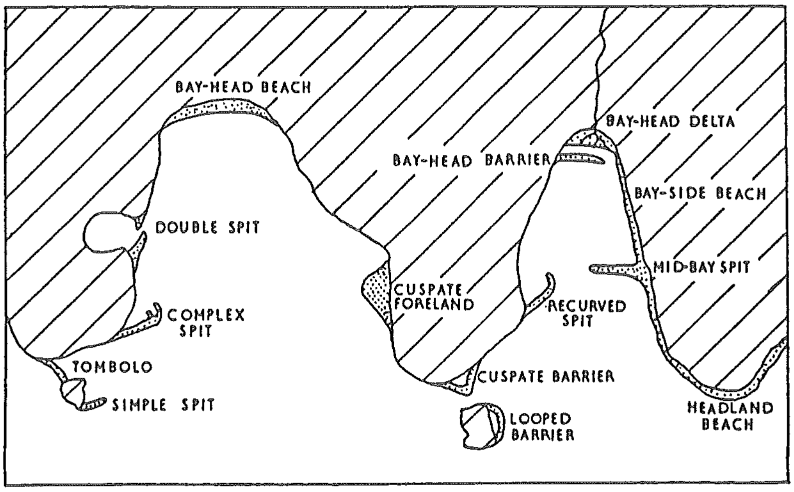


FIG. 12–6. Diagrammatic map to show the different types of depositional features on a youthful steep indented coast.

The features formed during the early stage of modification of a steep indented shoreline are very varied in character. The main types of features are illustrated diagrammatially in fig. 12–6 and may be listed as follows:

Bay-head beach
Bay-head delta
Bay-head barrier
Bay-side beach
Mid-bay spit
Headland beach
Spit—simple sand
    shingle
    mixed sand and shingle
  recurved
  complex
  two direction—double spit
Looped barrier
Tombolo
Cuspate barrier
Cuspate foreland

(a) The initial form and modifying processes

The initial form is theoretically one which has not been altered at all by marine agencies where sea-level comes to rest against an area of diverse relief with fairly steep slopes. This form is well shown in the north-east part of the South Island of New Zealand in the Marlborough Sound area as shown by Cotton (1954).[1] This is a recently submerged mountainous area, which does not even show cliffing on the headlands.

The most important modifying process on such a coast is the wave action. The effect of the outline of the coast on the pattern of wave attack is also significant. Wave attack will be very uneven in intensity on an indented coast; refraction will cause concentration of energy of the longer waves on the headlands and dissipation in the bays. The oblique approach of waves, particularly the shorter ones, along an intricate coastline, is also very important as it affects the direction of movement of beach material, which will probably be towards the bays. Under these conditions the direction of movement of beach material will not necessarily be in a uniform direction along great stretches of coastline but will be directed away from the headlands, which will, in any case, hamper the free movement of material alongshore.

(b) Modification produced

i. BY EROSION. The absence of beaches allows the waves to attack the coast, their attack is most effective where the energy of the sea is concentrated on the headlands. In general this has the effect of straightening the coastline, which is the ultimate aim of the marine forces. In detail, however, the initial result may be the reverse because if the rocks are of uneven resistance to marine erosion the softer ones will be worn away more quickly and the resultant coast will be more irregular than at first. This initial increase in the detailed complexity of the coastal outline produces a 'crenulate' coastline. It is well illustrated on the coasts of south-west Wales and parts of the south-west peninsula of Devon and Cornwall and is illustrated in fig. 12–1.

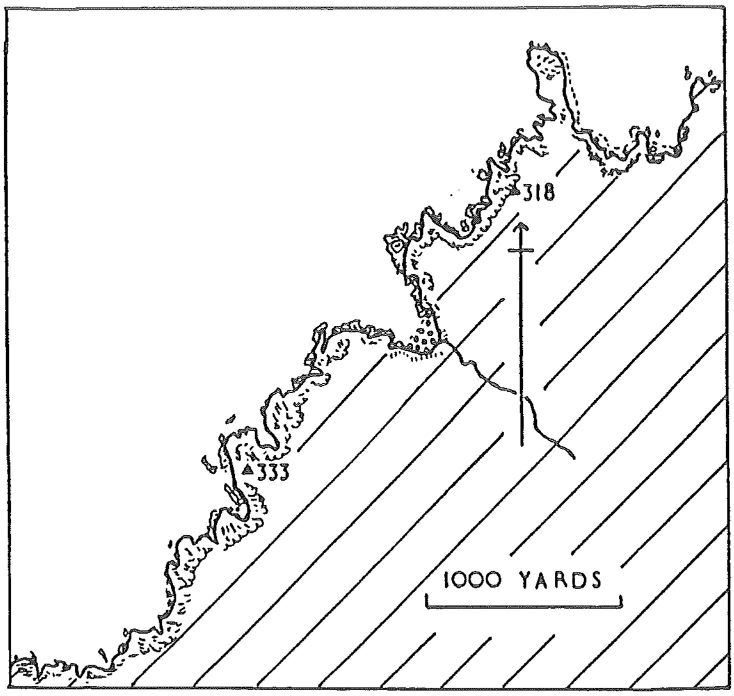


FIG. 12–1. Map of the crenulate coast of Cornwall. (Crown Copyright reserved.)

Coastal irregularity, due to differences of rock resistance, is very clearly shown on the part of the coast of Dorset where the strike of the rocks trends parallel to the coast. The resistant Portland and Purbeck rocks have been narrowly breached in Lulworth Cove while a wide bay has been scooped out in the soft Wealden clay inland of the limestone. Stair Hole just to the west of the cove illustrates an earlier stage in such a development where the hard rock has only just been breached by a tunnel, while the sea is starting to remove the softer rock from behind, in which work it is assisted by much slumping and mudflow activity in the clays.[3] The various stages including the more open Worbarrow Bay to the east are shown in fig. 12–2.

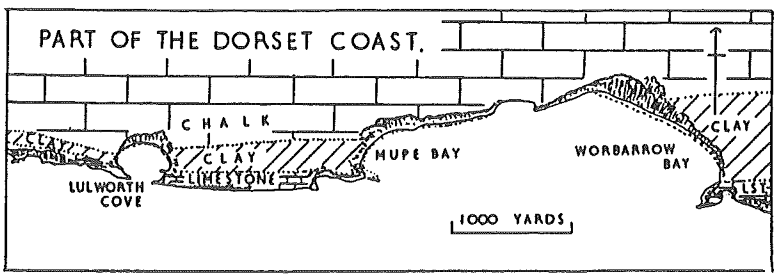


FIG. 12–2. Map of part of the coast of Dorset to show Lulworth Cove in relation to the rock type. (Crown Copyright reserved.)

ii. BY DEPOSITION. During the early stage of modification of a steep indented coast the features formed by deposition are many and varied; their form depending mainly on the type of material available and on the relationship between the coastal outline and the incidence of wave forces. In considering the form of the depositional features there are two tendencies to be taken into account. Firstly, the movement of material along the shore tends to prolong the direction of the solid coastline at points where there is an abrupt change in direction of the rocky shore. Secondly, there is the tendency for wave built structures to turn to face the direction from which the dominant waves are approaching.

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4 Natural system (Quantified)

4.1 General

4.1.1 Climate

Two climatic factors, wind and rainfall, will be treated here briefly, as far as relevant for coastal processes. More details can be found in climatological atlasses or other geographical sources. Wind causes waves and currents in the seas, thus influencing coastal processes and rainfall causes river discharges which form the main source of sediment for the coasts in Indonesia.

Indonesia extends from 6° N latitude to 11° S latitude, so a typical equatorial climate can be expected. Moreover, none of the islands presents a mass of land large enough to modify to any great extent the climate found along the equator in the great oceans, the characteristics of which are constant high temperature with little variation throughout the year and high rainfall. Between 5° N and 5° S latitude there is little evidence of a wet and a dry season, but at the most some evidence of a 'wet' and 'wetter' season. Outside this area, Java and especially the Lesser Sundas, seasonal changes begin to be apparent.
In Figure 4.1 A through D an overview is given of wind and rain in the archipelago.

WIND

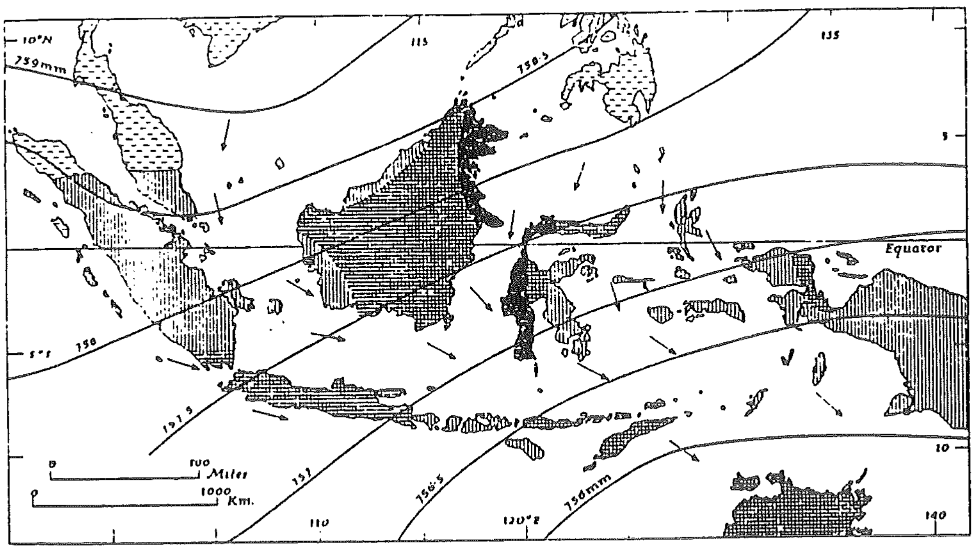
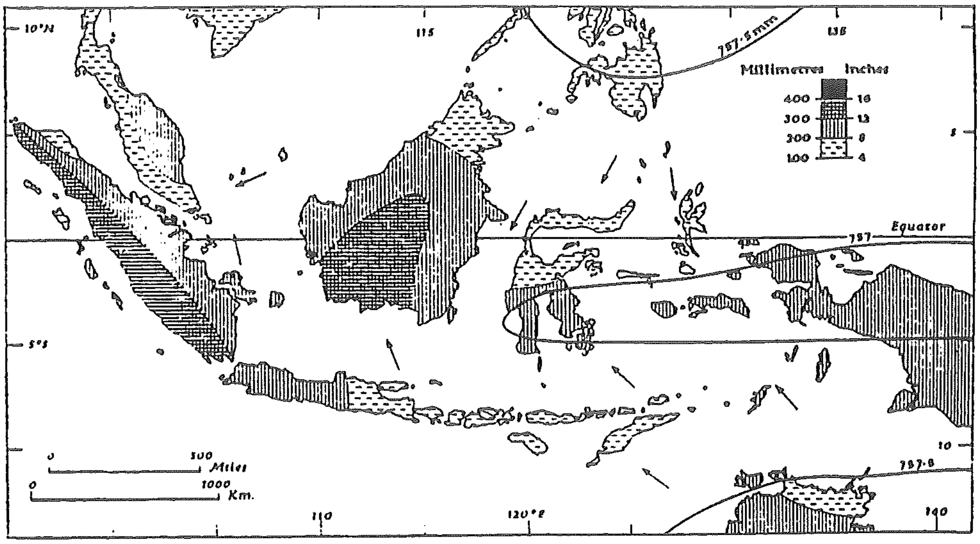


Fig. 118. Pressure, winds and rain. January



Pressure, winds and rain. April

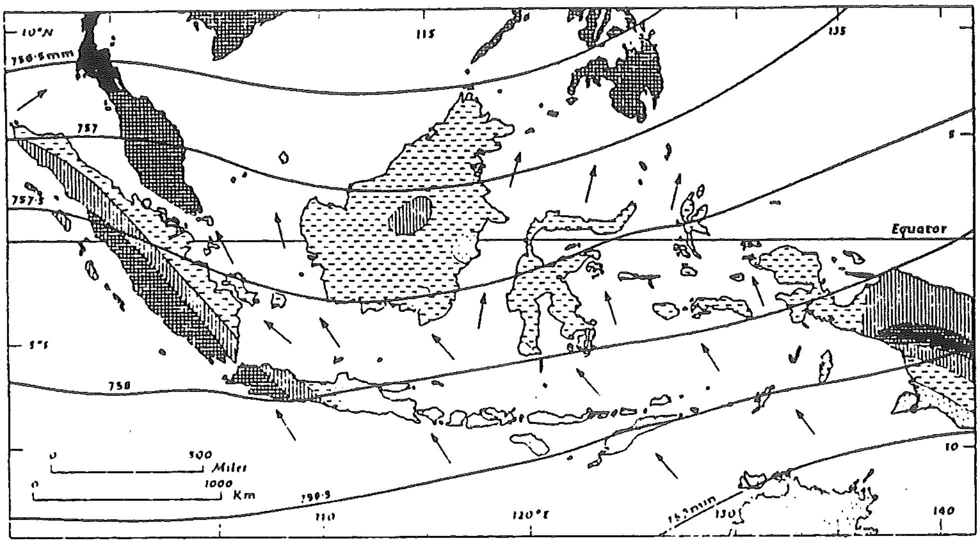
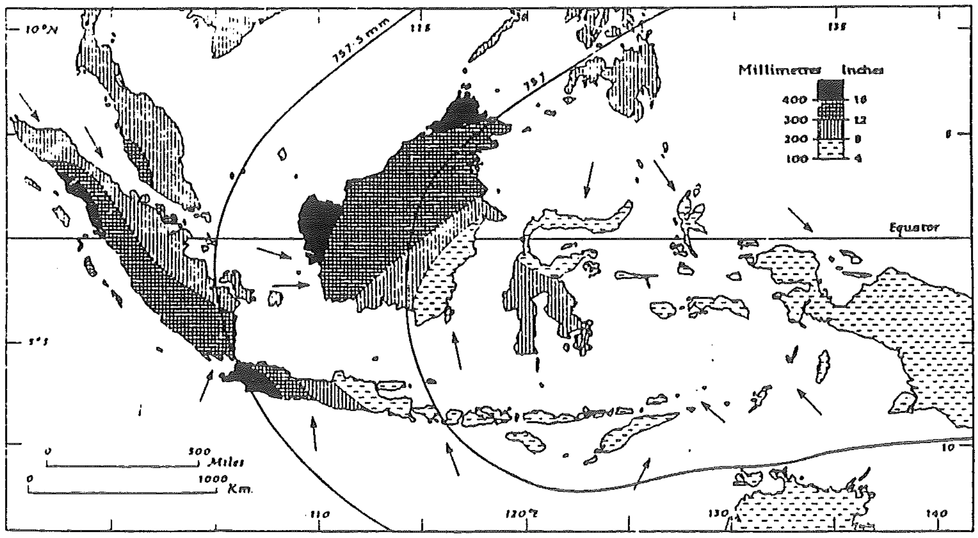


Fig. 120. Pressure, winds and rain. July
Source: A. Austin Miller, *Climatology*, p. 83, fig. 27 (London, 1931).



Pressure, winds and rain. November

Figure 4.1 Pressure, winds and rain in January, April, July and November.

Indonesia lies between the two monsoon centres of Asia and Australia. From December to March the air flows from Asia and Australia and from May to September in almost exactly the opposite direction, see Figure 4.1 A (January) and C (July). In January, the wind direction is northerly, or north-easterly, to the north of the equator, and westerly, or north-westerly, to the south of the equator. As the wind direction in Java is westerly, the season is referred to there as the west monsoon and the name is frequently applied elsewehere in the archipelago, even north of the equator where the wind has no westerly component. In July, winds are south-easterly to the south of the equator, and southerly, or south-westerly to the north of the equator. This is the season of the so-called 'east monsoon', though the wind is nowhere due east.

During the change-over from one direction to the other, in April-May and again in October-November, the winds are light and variable and their direction is such that they closely approximate to the normal system of north-east and south-east trade winds, see Figure 4.1 B and D. Table 4.1 (from Naval Int,1949) gives for four stations in Indonesia the wind directions per month (old Dutch names are used, Discovery Oostbank is in the Java Sea, SE of Bilitung, de Bril is S of Ujung Pandang.

Table 4.1 Percentage frequency of wind direction

Discovery Oostbank (1911–1918)

	N	NE	E	SE	S	SW	W	NW	Calm
J.	3	0	0	0	0	3	40	54	0
F.	3	0	0	0	2	3	25	67	0
M.	6	2	2	4	3	10	37	33	3
A.	4	8	6	26	11	10	17	13	6
M.	4	3	10	49	9	11	9	5	0
J.	1	4	10	69	9	3	3	1	0
J.	1	1	9	79	3	2	0	0	4
A.	1	1	11	82	3	0	1	1	0
S.	1	1	8	79	6	4	1	0	0
O.	4	2	4	54	16	11	6	3	0
N.	4	7	5	21	10	16	20	17	0
D.	3	1	1	1	3	9	44	37	0

De Bril (1911–18)

	N	NE	E	SE	S	SW	W	NW	Calm
J.	7	0	0	0	0	1	65	26	0
F.	9	0	2	0	1	3	57	27	2
M.	10	1	10	4	5	4	49	17	0
A.	3	3	61	3	3	1	6	3	6
M.	6	1	60	22	5	0	4	1	0
J.	0	1	71	24	2	0	0	0	2
J.	0	1	60	31	2	0	0	0	5
A.	0	0	63	36	1	0	0	0	0
S.	0	0	59	34	5	0	0	0	2
O.	2	0	45	40	9	0	3	0	1
N.	5	2	32	23	13	2	21	2	2
D.	5	1	6	0	3	3	63	10	9

Batavia (1891–1925)

	N	NE	E	SE	S	SW	W	NW	Calm
J.	13	5	1	4	7	8	18	18	27
F.	13	5	2	4	6	8	14	18	30
M.	12	7	4	7	9	8	13	11	28
A.	10	14	10	11	11	6	5	4	29
M.	9	16	13	13	9	4	4	2	30
J.	9	17	15	14	9	2	2	2	30
J.	10	18	16	15	10	2	1	2	26
A.	12	19	14	15	11	2	1	2	23
S.	17	16	10	13	12	3	2	2	24
O.	17	14	8	10	14	5	3	3	26
N.	15	8	5	8	15	9	8	6	26
D	11	5	3	5	10	11	17	12	26

Christmas Island (1910–19)

	N	NE	E	SE	S	SW	W	NW	Calm
J.	2	0	2	11	7	11	18	8	41
F.	2	3	6	10	2	8	20	9	40
M.	1	3	17	13	3	5	13	8	37
A.	0	3	31	15	1	2	4	1	44
M.	1	4	32	21	2	1	1	0	38
J.	2	4	40	21	0	0	1	0	31
J.	2	5	37	31	0	0	0	0	24
A.	0	5	38	36	0	0	0	0	20
S.	1	4	32	45	0	0	0	0	18
O.	2	1	30	42	0	2	1	1	20
N.	4	5	16	36	1	1	3	4	31
D.	3	1	6	21	6	11	8	6	38

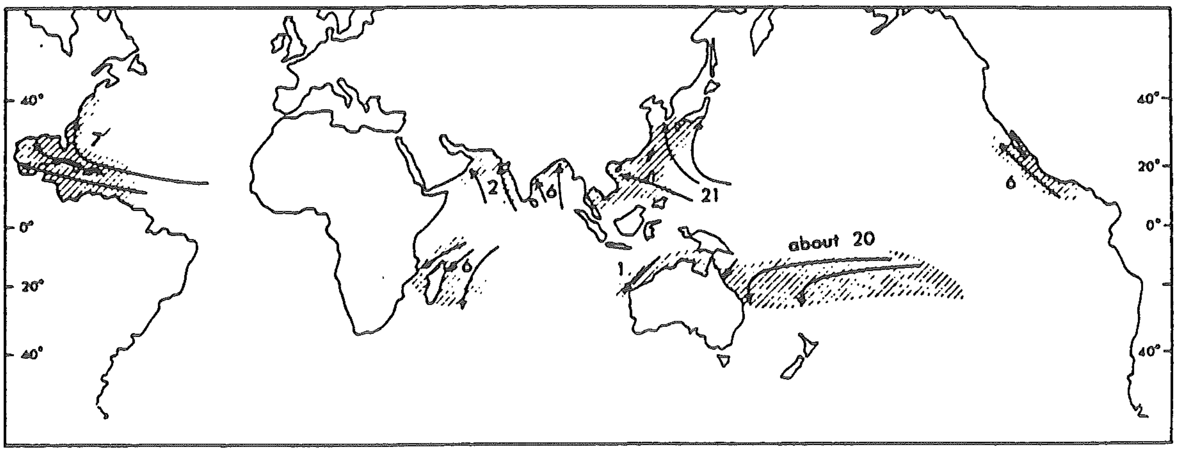


Figure 4.2 Coasts subject to tropical cyclones with annual frequencies (Davies,1985)

The mean wind velocities in exposed stations (lighthouses) at the height of the monsoons are from 6 to 8 m/s (Beaufort 4). In general the wind velocities in Indonesia are not high and gales are rare. Figure 4.2 shows that Indonesia is outside the region of severe tropical cyclones (due to the absence of Coriolis force at the equator). Small isolated squalls occur with wind velocities reaching 20 m/s (Beaufort 8) for short periods, certain areas being especially susceptible, e.g. the Malacca straits where the storms are known as Sumatras. They occur only at night, most frequently in the months June through August.
Another indication of the relatively mild wind climate is indicated e.g by the high percentages of calms (25 to 40%) at Christmas Island and Jakarta in Table 4.1.

In the following table (from Naval Int,1949) maximum wind velocities averaged over 1 hour and 1 day respectively are given for Semarang.

Table 4.2 Maximum wind velocities in Semarang (in ft/s !, from Naval Int,1949)

1918–1921	J.	F.	M.	A.	M.	J.	J.	A.	S.	O.	N.	D.	Year
Absolute max.	36	46	39	27	28	29	30	31	33	32	29	35	46
Mean max.	21	24	20	17	19	20	19	23	24	22	21	19	21

Source: C. Braak, *op. cit.*, ~~vol 1, part 2,~~ p. 38.

The daily rhythm of land and sea breezes sometimes even overrules the monsoon winds, e.g. on the west and north-east coasts of Sumatera. In the mean wind velocities, the influence of these breezes is clearly visible, see Table 4.3

Table 4.3 Mean wind velocities (in ft/s !, from Naval Int,1949)

KOEPANG (1913–17)

Period of day	J.	F.	M.	A.	M.	J.	J.	A.	S.	O.	N.	D.	Year
7 a.m.—12 noon	18	14	13	14	16	17	17	15	14	16	13	14	15
12—5 p.m.	18	15	13	14	16	16	17	15	15	16	14	14	15
5 p.m.—7 a.m.	8	7	6	6	7	7	7	7	7	7	7	8	7

BATAVIA (1866–1918)

Period of day	J.	F.	M.	A.	M.	J.	J.	A.	S.	O.	N.	D.	Year
7.30 a.m.—12.30 p.m.	6	6	6	6	5	5	6	6	6	6	5	6	6
12.30 p.m.—5.30 p.m.	7	7	7	7	7	7	8	9	9	9	7	7	8
5.30 p.m.—7.30 p.m.	2	2	2	2	2	2	2	2	2	2	2	2	2

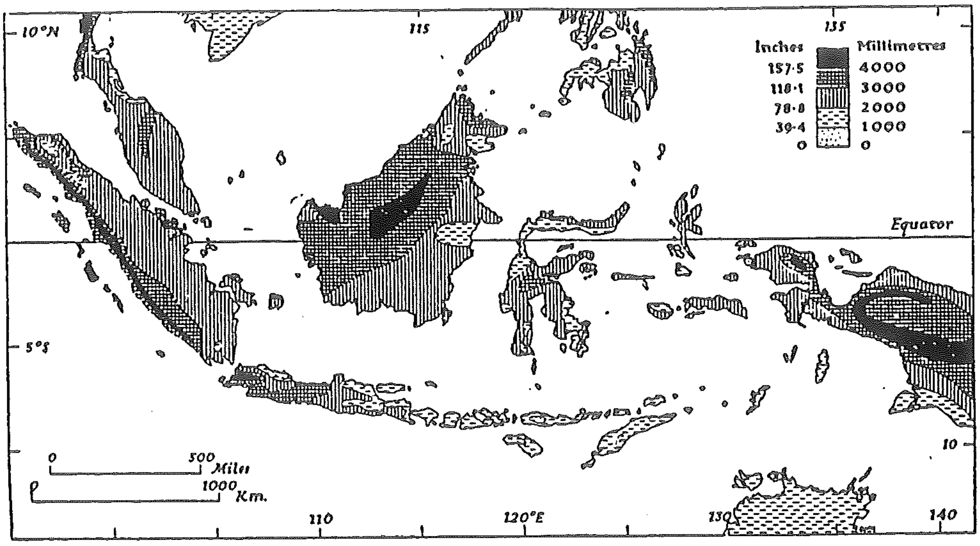
PASOEROEAN (1901–10)

Period of day	J.	F.	M.	A.	M.	J.	J.	A.	S.	O.	N.	D.	Year
7 a.m.—12 noon	5	5	5	5	5	5	6	6	7	7	6	6	6
12—5 p.m.	7	7	6	7	7	8	8	10	10	10	8	7	8
5 p.m.—7 a.m.	3	2	2	2	2	3	3	4	5	5	3	3	3

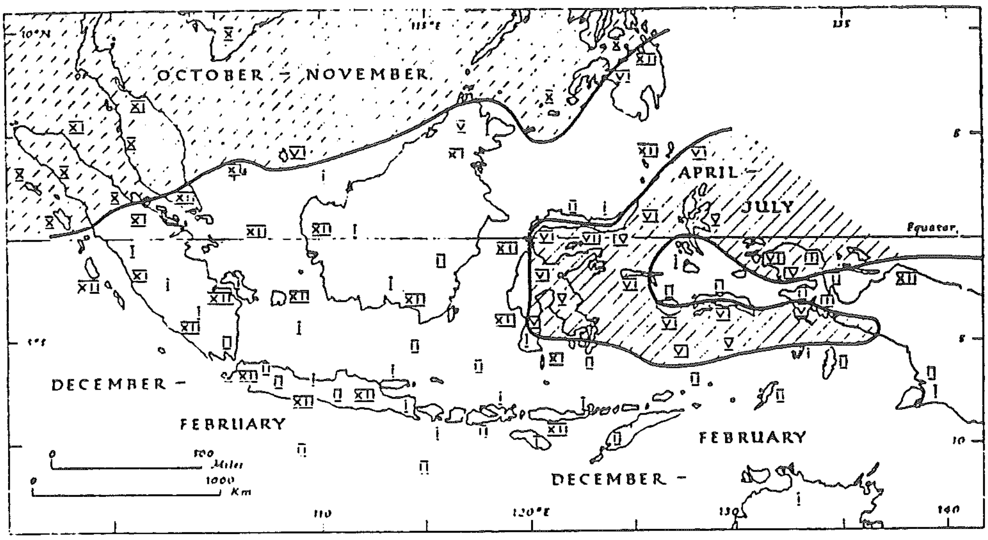
Source: C. Braak, 'Het Klimaat van Nederlandsch-Indië,' vol. 1, part 2, pp. 100–1 (Batavia, 1923–25).

Precipitation

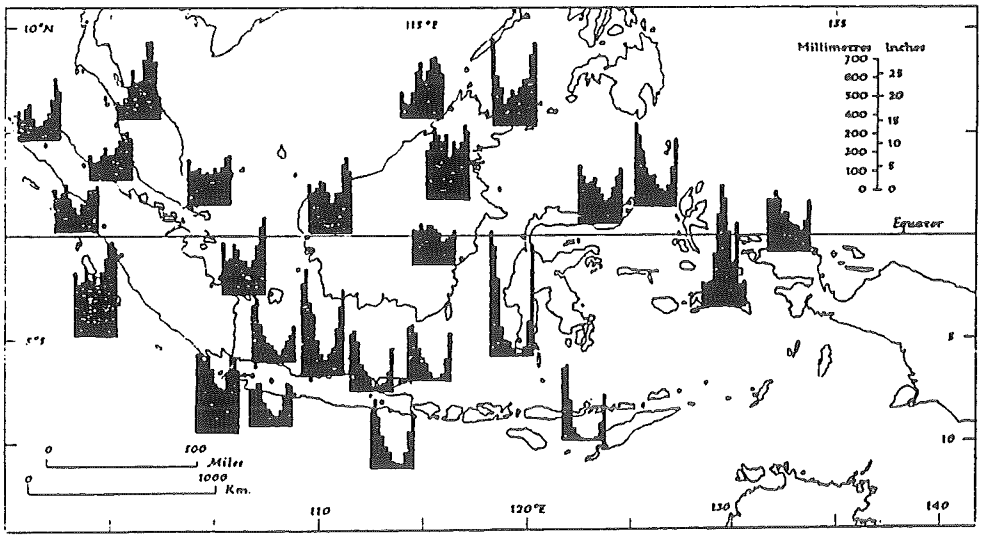
Indonesia probably forms the greatest area of heavy rainfall on earth. Stations with less than 1 m rain per year are exceptional and the average lies between 1.5 and 2 m. Only in the east, approaching Australia, is there a dry season, which occurs around July through September. It begins to be noticeable at the east end of Java with three dry months, while at Timor there are five dry months. Figure 4.3 shows some data.



Yearly total of rainfall



Months of maximum rainfall



Rainfall: Monthly means

Figure 4.3 Rainfall data for Indonesia (from Braak,1936)

4.1.2 Bathymetry

The bathymetry of the Indonesian seas, and more in general the geometry, is important for the behaviour of all kinds of waves and currents. The generation and decay of wind waves depends, beside of course of winds, on waterdepths and fetch of the seas. The propagation of tidal waves in Indonesia is highly influenced by the bathymetry of the seas and shelves as will be showed later on in this chapter. The bathymetry of the Indonesian seas is the outcome of primary geological processes, like tectonics of the earth crust and sea level rise. In chapter 3, a description of these processes is given. Here it will be sufficient to give a map of the Indonesian archipelago with the main depth contours of the various seas.

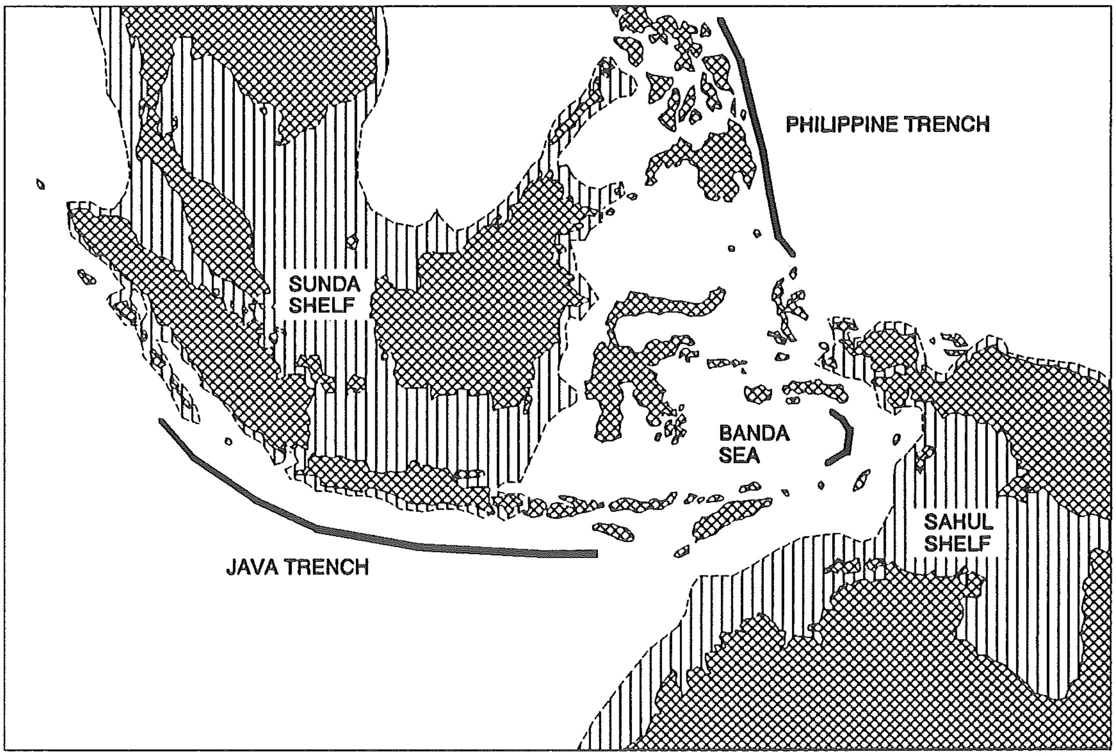


Figure 4.4 Depth contours in the Indonesian archipelago

******** In the final version of the Pedoman a detailed map should be inluded ********

4.2 *Wind waves*

4.2.1 *Wave generation and decay*

Wind blowing over a water surface generates waves. The process of energy transfer from moving air to the water is complicated and only partly understood. Turbulent pressure fluctuations in the air are probably more important in the generation of waves than the shear stress between the wind and the water surface. The complexity of the process involved makes that empirical relations have to be used. The most commonly used relation is the Sverdrup-Munk-Bretschneider (SMB) method for deep water waves, later extended for shallow water by Bretschneider. Waves are usually characterised by their height, their length and their period. The wave height is the difference between the trough and the top of the wave (two times the amplitude). For most (individual) waves a fixed relation can be derived between the wave-length and the wave period. This relation depends very much on the water depth. Parameters influencing wave height and wave period are:

- wind velocity (u), averaged over a certain period of time (e.g. 6 hours)
- fetch (F), the overwater distance where the wind is effective
- duration (t), the time period in which the wind can be considered constant (wind speed and direction)
- waterdepth (h), responsible for wave growth limitation due to mainly bottom friction

A higher wind velocity will always result in higher waves. Limitations come from the other parameters: a short fetch will limit the wave height and so does a short duration or a limited waterdepth. Reversely this means that, given a certain wind speed, e.g. a longer fetch will not result in higher waves, because the maximum wave due to that wind speed is already reached or one of the other parameters is limiting.

Without limiting factors, the empirical relations between wave height and wave period on one side and wind speed on the other side, are simply:

$$\frac{g\,H_s}{u^2} = 0.283 \qquad \frac{g\,T_s}{2\,\pi\,u} = 1.20 \tag{4.1}$$

in which H_S is the *significant wave height*, which is the average of the highest third part in a wave registration, and T_S the *significant wave period*, the average period of the same waves, see also section 4.2.2 on Wave statistics.

The other, limiting, parameters are added empirically. The duration is neglected here, since from the wind data it appeared that the (monsoon) winds are very steady in force and direction. Occasional hourly maxima have a duration that can be important for navigation, but is too short for wind growth of any importance for coastal engineering. So, all data will be valid for so-called fully arisen seas. The limiting influence of fetch and depth are added to equation (4.1) as hyperbolic tangents. The result looks rather grim, but it should be realized that a hyperbolic tangent is often a simple way of expressing a reducing factor that vanishes (becomes 1) when the argument becomes large ($\tanh(1.5) \approx 0.9$ and $\tanh(2.5) \approx 0.99$), see also Figure 4.9C.

The final result for the wave height is:

$$\frac{g H_s}{u^2} = 0.283 \tanh\left[0.578\left(\frac{g h}{u^2}\right)^{0.75}\right] \tanh\left[\frac{0.0125\left(\dfrac{g F}{u^2}\right)^{0.42}}{\tanh\left[0.578\left(\dfrac{g h}{u^2}\right)^{0.75}\right]}\right] \tag{4.2}$$

and for the wave period:

$$\frac{g T_s}{2 \pi u} = 1.20 \tanh\left[0.833\left(\frac{g h}{u^2}\right)^{0.375}\right] \tanh\left[\frac{0.077\left(\dfrac{g F}{u^2}\right)^{0.25}}{\tanh\left[0.833\left(\dfrac{g h}{u^2}\right)^{0.375}\right]}\right] \tag{4.3}$$

Figure 4.5 shows the wave heights and periods for a sea of 50 m deep (e.g. the Java Sea), with fetches up to 500 km and wind speeds up to 8 m/s, see section 4.1.1 on wind in Indonesia. From this figure can be seen that for the lower wind velocities, a longer fetch does not lead to higher waves anymore, or in other words, the value of the hyperbolic tangent in which the fetch and the wind speed are related, has become 1. For the given wind velocities, the depth of the Java Sea (50 m) is such that for the the wave heights the sea can be considered as deep water. For the wave periods, there is still some depth influence.

More about fetch

The fetch is the length over which the wind can blow unhindered. When the stretch of coast considered is sheltered by land stretches or large islands, the fetch is taken from the point where the wind gets free space. Small islands can be neglected, since the waves will not be influenced by them. "Small" is of course a relative notion. For distances downwind from islands, large compared to the island dimensions, the island can be neglected.

For complicated geometries in bays and estuaries, a procedure will be needed to determine the effective fetch for a given situation.

******** These refinements of fetch calculation should be given in the final version of the Pedoman. ********

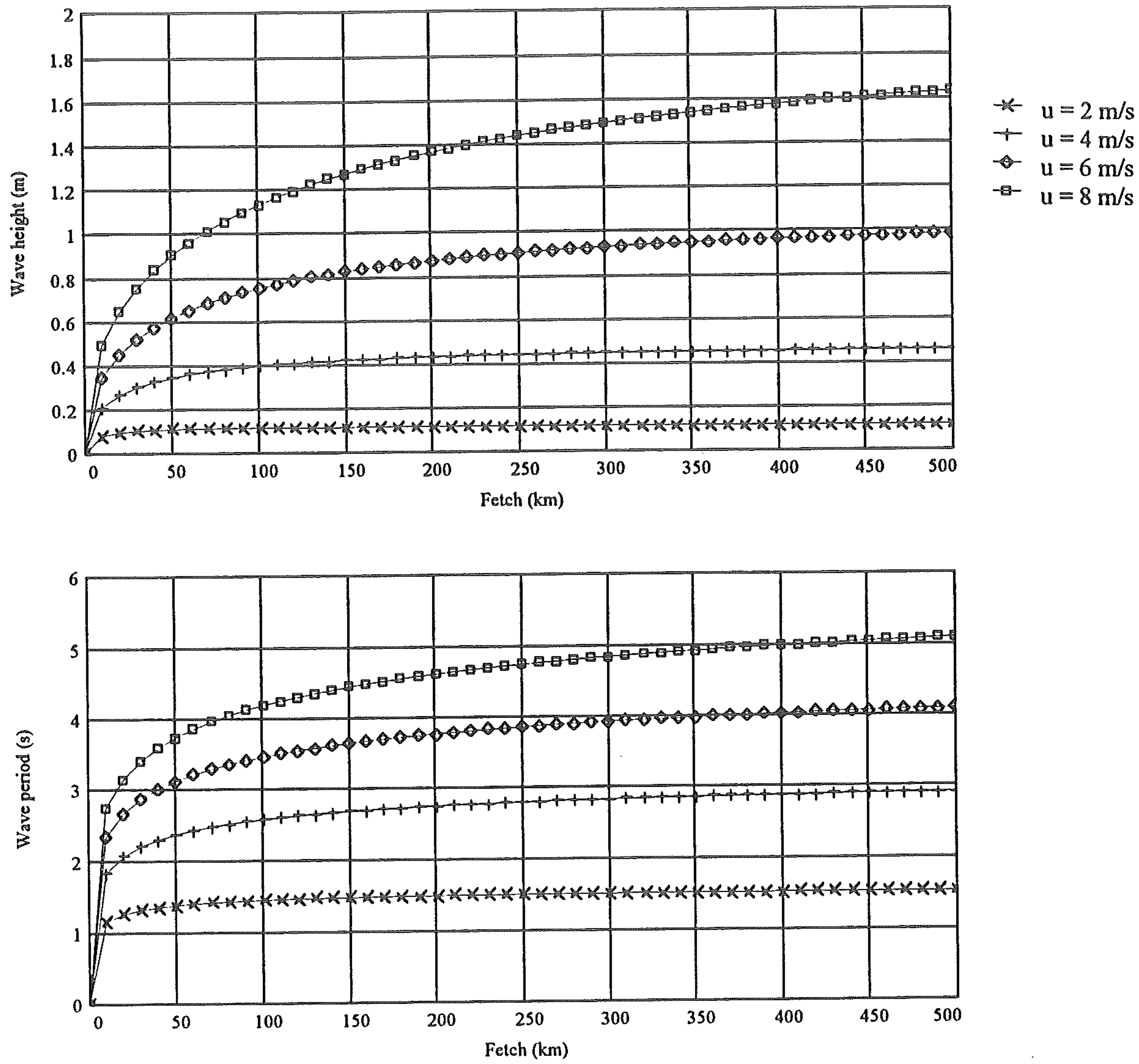


Figure 4.5 Wave heights and periods for monsoon waves in the Java sea

Swell

The above given procedure and formulae are valid for so-called *sea-waves* which are generated directly in the area considered and which are still under the influence of the generating wind. An example was given for monsoon waves at coasts along the Java Sea. On e.g. the west coast of Sumatera or the south coast of Java this procedure can also be applied for monsoon waves, but there, also *swell* will be important for coastal engineering. When a wave field leaves the area with the wind, the waves no longer increase in height, but (because of some friction and dispersion) slowly decrease in height. In the same process the wave period starts to increase. The wave motion resulting from this process is called swell. Wind waves, or sea-waves, are relatively steep (ratio wave height/wave period is high), have short crests-widths, and look quite randomly. Swell-waves are usually quite regular, long, relatively low waves. The wave height, wave period, and wave direction of swell-waves cannot be related any more to the local wind. Swell can come from sources more than 10,000 km away. This is due to the fact, that the energy of the waves does not disappear so easily after the driving force of the wind has vanished. Energy decay is relatively large in the beginning, but goes very slowly once the waves have reached moderate heights and very long periods, see Figure 4.6A. In this example can be seen that after waves on the ocean have reached a height of around 1 m, it can travel on 10,000 km with hardly any further decay. Although the height of a swell wave is usually low, this does not mean that they are not important. When they approach a coast, they shoal, see section 4.2.4. Because of the long period, the wave height increase is considerable. Because of this phenomenon, one can often observe at ocean coasts heavy breakers, while at sea hardly any wave can be observed.

Swell on the Indian Ocean coasts of Indonesia come from the belt around Antarctica where year round very frequently high wind velocities occur, especially around 55° S latitude. Although the wind direction in this area can vary considerably, the swell that comes from these storms has a tendency to come from south-westerly directions, due to the great circle paths along the globe, see Figure 4.6 B. For the Indonesian coasts this means that south-westerly swell can be expected year round. The occurrence of this swell can not be computed from wind statistics in the area, but has to come from direct wave measurements, see section 4.2.6.

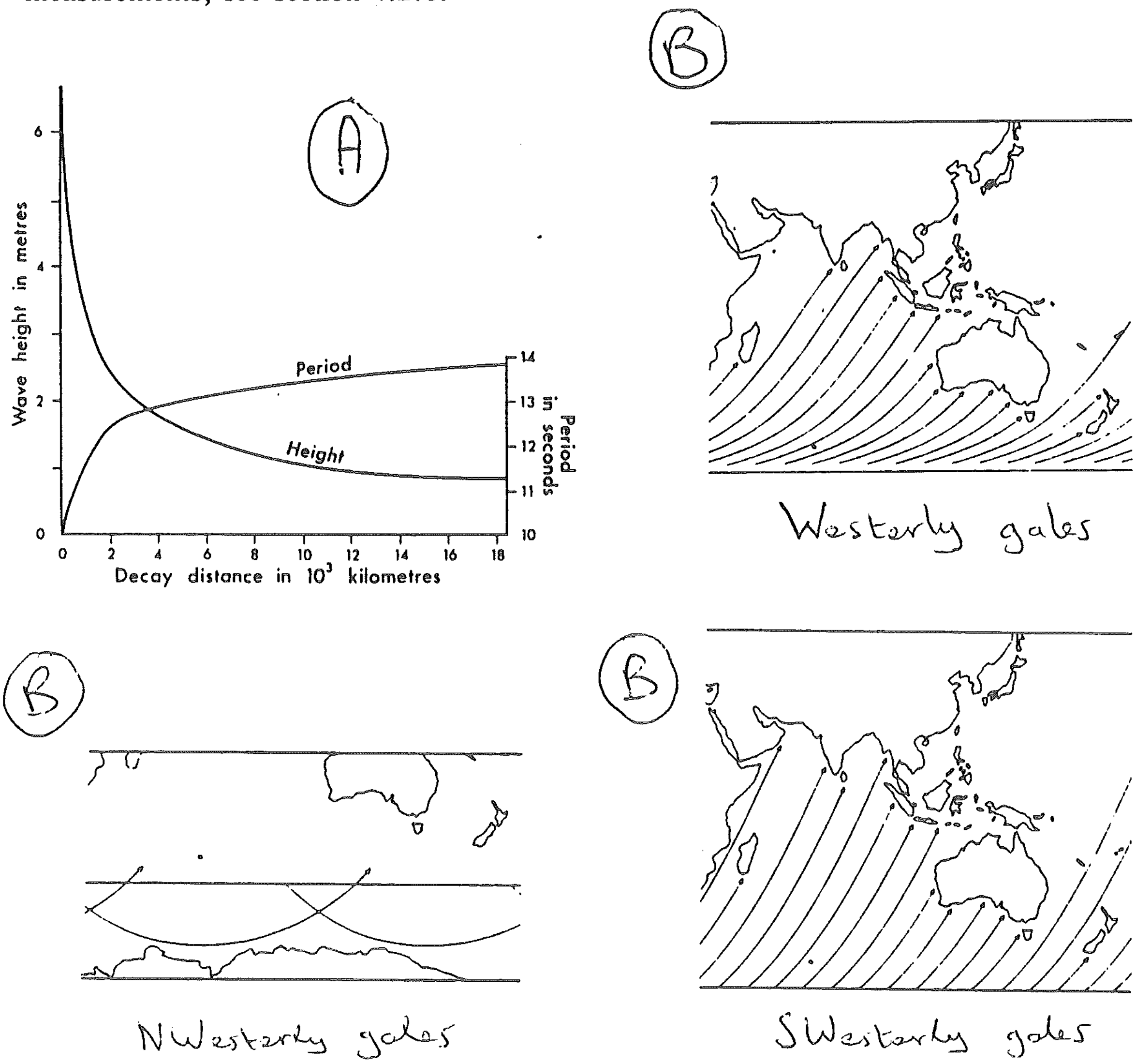


Figure 4.6 Decay of waves and sources of swell in Indonesia

4.2.2 Wave statistics

Wind is a turbulent flow with irregular velocity variations. So, when wind blows over a water surface, the waves that are the result, will be irregular too, (see Figure 4.7A). The water level is a stochastic variable. Generally, measurement of water levels are only made at a fixed location. This yields a stochastic record of water levels as a function of time, which can have a length of minutes hours or even a number of years. Statistics are necessary to condense and describe these data into more usable form without the loss of valuable details. What, specifically, are the statistical parameters needed ? Civil engineers are most often interested in wave heights. Therefore, it would seem most convenient to consider the statistical distribution of wave heights and describe it e.g. with a mean value and a standard deviation in the case of a Gaussian distribution or any other characteristic values that are appropriate for wave height distribution.

The value of such a characteristic waveheight determined from a wave record will be influenced in some extent by the duration of that record. Consider the following situation: A small sea is exposed to a constant light wind for a long time, so that a stable wave height has developed. Additionally, during a period of only 1/2 hour a severe thunderstorm with high winds passes over the area. During this short, but intense storm, higher waves will be generated which will then die out after the storm. If a characteristic wave height is determined using a record lasting the entire day, then the influence of the thunderstorm will be more or less lost in the rest of the data while, on the other hand, a characteristic wave height based upon, say, a single hour of record which includes all of the storm will be higher. The importance of the choice of a registration period becomes even more clear when wave conditions on different days or months are considered. The wave field will show completely different characteristics on different days, even in a rather steady monsoon climate as in Indonesia. Thus, the problem becomes twofold:

a. The determination of the statistical parameters necessary to characterize a portion of the wave record representing an interval of constant sea conditions (usually a few hours), the **short term wave distribution.**
b. The determination of the frequency with which these statistical characterizations of the earlier mentioned sea state occur, the **long term wave distribution.**

Short term wave distribution

The first difficulty encountered in determining statistical wave parameters is even more basic: what is actually a wave in the registration of Figure 4.7A? Figure 4.7B shows the answer: the period of an individual wave is defined as the time between two "zero-crossings" (upward or downward, the zero-line is Mean Sea Level), T_z and the individual wave height H is the highest crest minus the lowest trough between these zero-crossings.

The common statistical parameters can be used to describe the wave height distribution; the mean value is the most simple of these. In coastal engineering practice, however, the smaller waves are generally neglected and the mean value of the highest 1/3 of the waves (which are of the most interest) is chosen. This mean is called the significant waveheight, denoted by H_{sig} or H_s. By comparing significant wave heights determined from actual wave records to visual estimates made while recording it has been found that these correspond rather well; an experienced observer can usually estimate H_s rather closely.

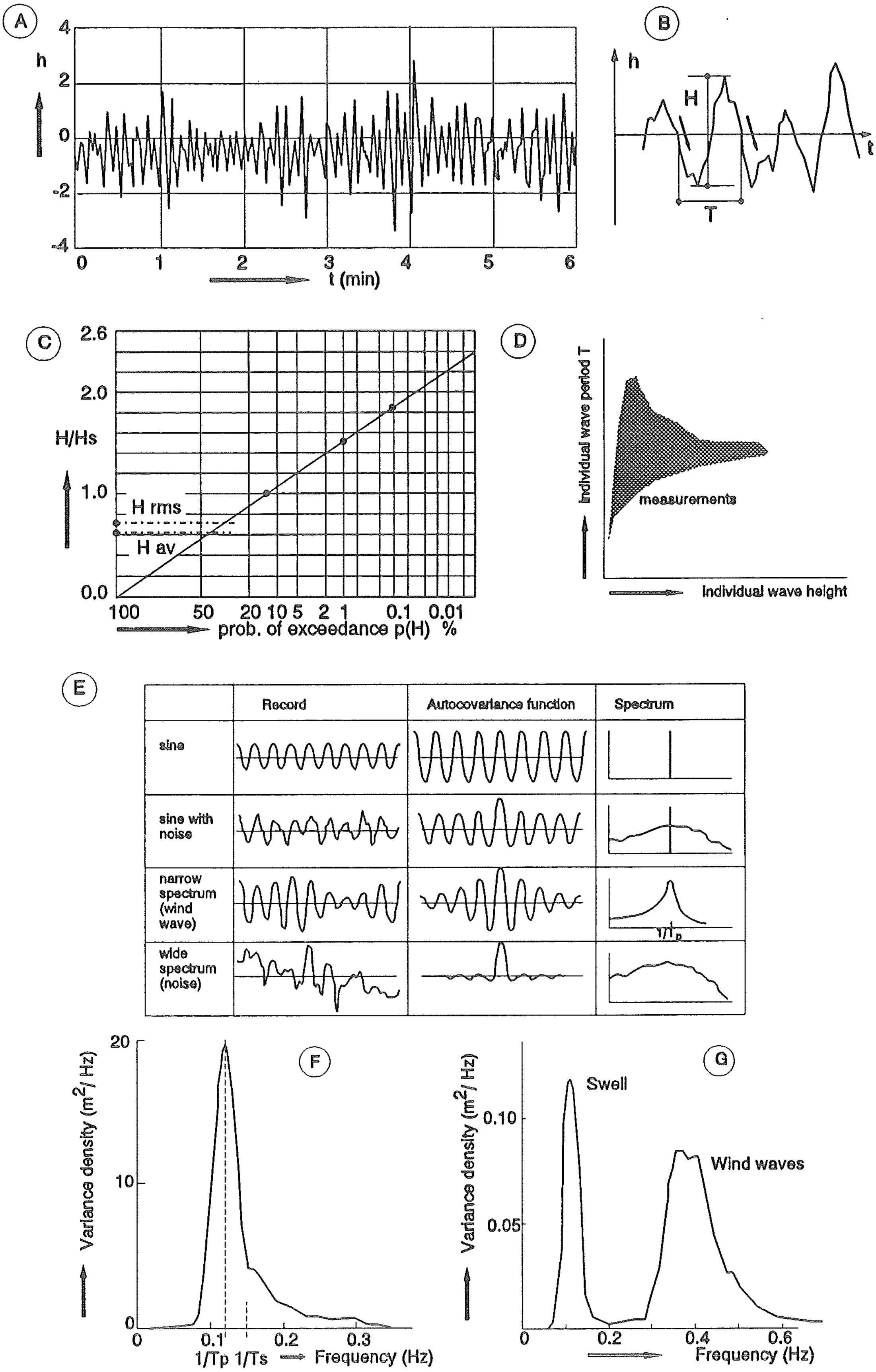


Figure 4.7 Short term wave distribution

Another wave height characterizing parameter of use in energy relationships is the root-mean-square wave height. For a group of N waves it is defined as:

$$H_{rms} = \sqrt{\frac{\sum_{i=1}^{N} H_i^2}{N}} \tag{4.4}$$

A disadvantage of such a simple parameter such as H_s is that it gives only a very rough description of the wave heights in a record. Fortunately, within reasonable accuracy, wave heights of natural, irregular waves also can be described via a theoretical distribution model: the Rayleigh Distribution. This distribution is completely characterized by a single parameter. Thus one parameter, for example the H_{rms} or the H_s is sufficient to completely characterize the distribution. Usually the H_s is selected as the describing parameter. In that case the Rayleigh distribution becomes:

$$P(H) = e^{-2(\frac{H}{H_s})^2} \tag{4.5}$$

Where:

- P(H) is the probability of exceedance of wave height H,
- H_s is the significant wave height of the record,
- e is the base of the natural logarithms.

Some values of P(H) as function of H_s are given in the following table. It is also possible to use the formula to construct a special graph-paper, which transforms the equation into a straight line. Figure 4.7C gives an example of such paper.
Obviously, using either the formula or graph-paper we can determine the probability of exceedance of any desired wave height occurring in an interval characterized by a given significant wave height. For example, according to the Rayleigh distribution 13.5% of the waves are higher than H_s and 1% of the waves are higher than 1.5 times H_s. Other extreme values are e.g. $H_{0.1\%} \approx 1.85 H_S$ and $H_{0.01\%} \approx 2.15 H_S$.

Some other handy relationships, also based upon the Rayleigh distribution are listed below:

$H_s = 1.41\ H_{rms}$
$H_{mean} = 0.886\ H_{rms}$
$H_s = 1.596\ H_{mean}$

Using the value H_s we may describe the wave field at a certain moment. The length of the record used to determine H_s must, on the one hand, be long enough to determine a dependable average (20 minutes is about a minimum) while the records must not be so long that the wave conditions vary dramatically during the observation period. Often significant wave heights are determined every 3, 6 or 12 hours, with 6 hours, corresponding to the interval between many other meteorological observations, being the most popular.

Wave spectra

Many attempts have been made to relate wind wave heights and periods. The relation between individual height and period within a wave registration is however rather weak, see Figure 4.7D. An irregular wave field is best described with a so-called variance-density or energy-density spectrum.

A variance-density spectrum can be used for the statistical description of any fluctuating signal, e.g. the deviation of a sailing ship from its course, the turbulent velocity fluctuation etc. Mathematically spoken, it is the Fourier transform of the auto-covariance of the waterlevel fluctuation. The waterlevel is assumed to be the sum of an infinite number of simple sine waves:

$$\eta(t) = \sum a_i \cos(2\pi f_i t + \phi_i) \tag{4.6}$$

Every component i has a distinct value for a and f, ϕ_i is the only random variable. η is considered to be only a function of t, hence equation (4.6) is usually indicated as the one-dimensional random phase model. When the process is assumed to be stationary (the characteristics of the process do not change within a registration period) and Gaussian, it can be characterized by a mean value (still water level = 0) and the autocovariance-function:

$$\begin{aligned} C(\tau) &= covariance(\eta_t, \eta_{t+\tau}) = \frac{1}{2}\sum a_i^2 \cos(2\pi f_i \tau) \\ (for\ \tau &= 0: \quad C(0) = variance(\eta_t) = \frac{1}{2}\sum a_i^2 = \sigma^2) \end{aligned} \tag{4.7}$$

The autocovariance (<u>co</u>, because it is the variance between two values and <u>auto</u> because it is within the same process) is a measure for the correlation between two values in a record, it is a kind of "memory". For $\tau = 0$, C reduces to σ^2, where σ is the standard deviation of the wave record. The variance-spectrum is now the Fourier transform of the autocovariance function:

$$E(f) = \int_{-\infty}^{\infty} C(\tau)\, e^{-i2\pi f\tau}\, d\tau \tag{4.8}$$

Figure 4.7E shows some examples of registrations of a fluctuating variable with the accompanying autocovariance functions and variance-density spectra. The first is a simple sine, which has only one period. The autocovariance function is therefore also periodical and the spectrum is infinitely high and narrow. When this sine wave is mixed with noise (noise contains "all" frequencies), the spectrum becomes wide with again a narrow peak at the frequency of the sine wave. For pure noise, the autocovariance function decreases rapidly (little correlation between values) and the spectrum is very wide. A registration like the one in Figure 4.7A, gives a spectrum with a clear peak, where the variance-density has a maximum.

The physical meaning of a spectrum becomes clear when it is realised that the variance of a signal is highly identical to the energy in waves. The specific energy, see the table on page 22, is with $H = 2a$, identical to the standard deviation in equation (4.7) apart from a factor ρg. The physical interpretation of a wave spectrum is then the distribution of energy over the various wave frequencies, hence the name energy-density spectrum. Figure 4.7F and G show two "real" spectra. The first belongs to the registration of Figure 4.7A ($H_S = 4$m, $T_S = 6.5$ s). The second is double peaked: "fresh" wind waves with $f \approx 0.4$ Hz and "old" wind waves from another area, swell, with $f \approx 0.1$ Hz.

******** In the final version of this manual, more about wave spectra should be included

Spectral description of an irregular wave field is preferable because it contains a lot of valuable information. For practical purposes, however, e.g. in design calculations, it is still handy to characterize a wave field with one characteristic wave height and period.

The relation between the wave height and the area of the spectrum is for a Rayleigh distribution:

$$H_S \approx 4\sqrt{m_0} \quad with \quad m_0 = \int_0^\infty E(f)\, df \tag{4.9}$$

In practice H_S is found to be about 5 % smaller.

For the use of one characteristic wave period, there are again some possibilities. The average value of T is sometimes used. It seems to have the advantage of simplicity: x waves during y seconds gives $T_{av} = y/x$. A better measure is T_p, the so-called peak period, where the energy density has its maximum, see Figure 4.7F. $T_{1/3}$ or T_S is still widely used; it is the average period of the highest third part of the waves. In Figure 4.7D it can be seen that the scatter in the relation between wave height and period becomes much smaller for higher waves, which can justify the use of T_s. $T_s \approx 0{,}9\ T_p$ can be used, rather independent of the spectrum. The relation between the mean period and the other periods shows more scatter. As a first approximation: $T_m \approx 0.8 \sim 0.9 T_s$ can be used.

Long term wave distribution

Once the statistical characterizations of one interval of a long record have been completedet, this process can be repeated for each interval in the recording period which could extend over a period of years or even decades. Obviously, each interval produces a single value of H_s and this collection of significant wave height values can also be studied statistically.
From theoretical considerations one can find that the (long-term) distribution of the values of H_s should be a so-called Weibull distribution. The Weibull distribution has 3 coefficients. Two of these coefficients are for nearly all wave records exactly the same, so that the Weibull distribution in fact reduces to a simple logarithmic distribution:

$$P(H_s) = e^{-\frac{H_s}{a}} \qquad (4.10)$$

which is a straight line if log $[P(H_s)]$ is plotted versus H_s. In fact this is plotting the values of H_s on semi-logarithmic paper. This simplification works pretty well, especially for the low values of $P(H_s)$, say less than 10 % -the area which is of most interest. An example of such a long term distribution -based on data for the northern North Sea- is shown in the figure 4.2. This figure is compiled for "storms" --intervals over which a single value of H_s has been determined-- lasting for 6 hours. It is unimportant whether the observations were made using a continuous set of exact six hour intervals --such as a meteorological office might do-- or whether the measurements were made in atue tandom way. If H_s is being determined only at irregular intervals, these must be selected truly at random.

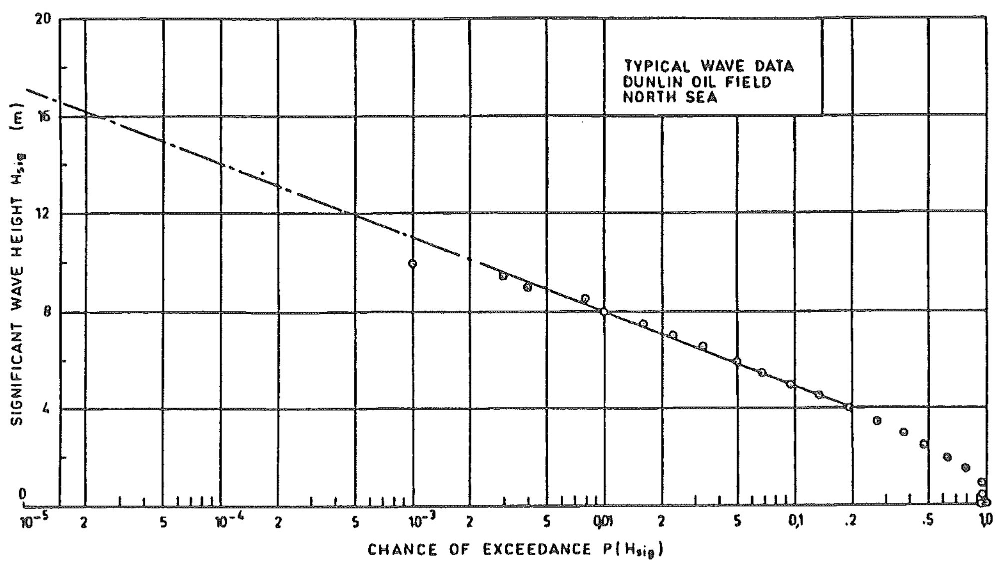


Figure 4.8 Long term distribution of H_S for the northern North Sea

******** In the final version this example should be an Indonesian one ********

4.2.3 *Wave mechanics*

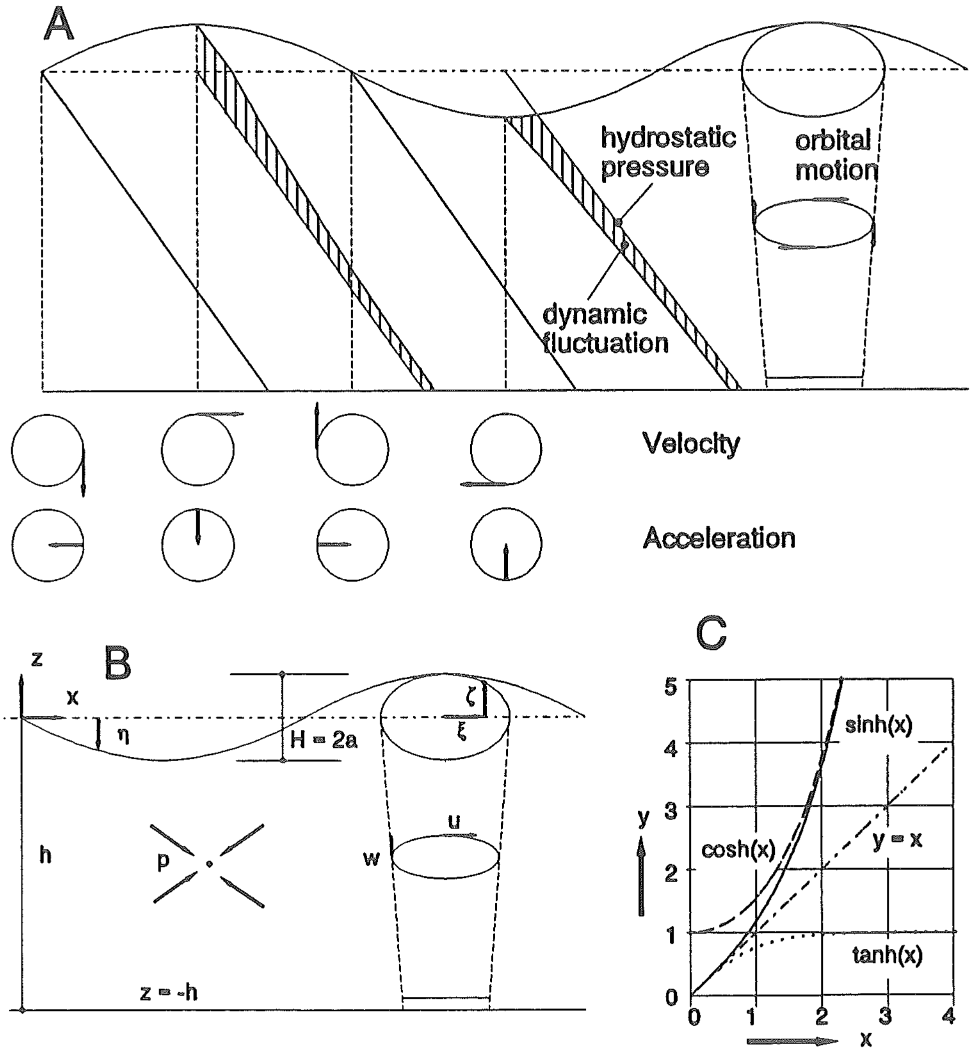


Figure 4.9 Water motion in periodic wave, definitions and behaviour of hyperbolic functions

Figure 4.9 shows the main characteristics of pressures and velocities in a simple, periodic, unbroken sine wave. The pressures deviate from the hydrostatic values, due to the vertical accelerations. The particle velocities are related to the orbital motion. In the general case these are ellipses, in deep water circles and at the bottom straight lines. It is good to bear in mind, that there are two velocities in waves: the propagation speed or celerity, which is the speed with which the wave shape travels, and the particle velocity. A clear demonstration of this difference can be seen during sporting events, when people just rise and sit down again, while "The Wave" is travelling through the crowd.

Water in motion is fully described with the Navier-Stokes equations, but, in different situations, different terms can be neglected. In open channel flow, friction has an important influence on the flow. Often, the boundary layer is equal to the water depth. In short waves the situation is completely different and the fluid motion can be described as an irrotational motion, apart from a thin boundary layer. This makes analytical solutions possible for the pressures and velocities in a wave, provided the shape of the wave is assumed as boundary condition. This has lead to many wave theories, all with a limited validity and application. In long waves, pressures (waterlevels) and velocities, are nowadays computed numerically for long periods at many locations quite easily. The possibility of a numerical solution for a complete irregular wave field, is still far away for practical application, mainly due to the large amount of calculation points in time and space for these literally short (in time and space) waves. But with the ever increasing speed and capacity of computers, this day may come.

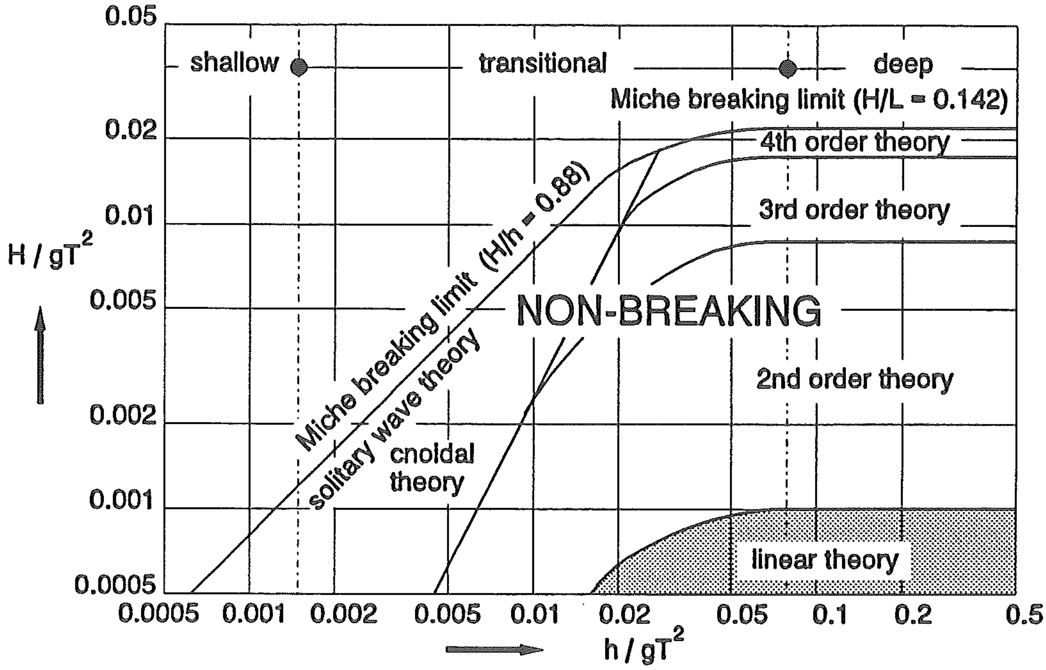


Figure 4.10 Validity of wave theories (LeMehaute,1976)

Figure 4.10 gives an overview of the validity of various wave theories, adapted from LeMehaute,1976. On the horizontal axis, the waterdepth is given, made dimensionless with the acceleration of gravity and the square of the wave period, which is a measure for the wave length. The vertical axis represents the wave height, again made dimensionless with the wave length. The vertical axis is a measure for the wave steepness ($\propto$ H/L) and the horizontal axis for the relative waterdepth ($\propto$ h/L). Upwards, the validity (or better: the very existence of waves) is limited by breaking due to steepness and to the left by breaking due to restricted waterdepth. Both limits are described with the breaking criterion by Miche:

$$H_b = 0.142\, L \tanh\left(\frac{2\pi}{L}\, h\right) \tag{4.11}$$

For deep water, this simply becomes: $H_b/L = 0.142$ ($h/L > 0.5 \rightarrow \tanh(2\pi h/L) \approx 1$). This is the maximum steepness for an individual wave. Applied to H_s in irregular waves, about half this value is found, since H_{max} in irregular waves is about $2H_s$, see the Rayleigh distribution in section 4.2.2. In practice, the steepness, H_s/L, is seldom more than 0.05. For shallow water, equation (4.11) leads to $H_b/h = 0.88$ ($h/L < 0.1 \rightarrow \tanh(2\pi h/L) \approx 2\pi h/L$). The solitary wave theory gives a somewhat different limit: $H_b/h \approx 0.78$. In section ? we will see that this quotient also depends on the slope of the bottom. Applied to H_s in irregular waves, values of $H_s/h = 0.4 - 0.5$ are usually found.

The shape of waves with increasing steepness in deep water (upper right corner in Figure 4.10) has to be described with more sine components, leading to more complex solutions of the equation of motion. Waves with considerable wave height in shallow water, can be described with the cn-oidal wave theory and in the limit with the solitary wave theory. Here only the linear or first order wave theory will be used. In the table of 22, an overview of the formulae in this theory is given. According to Figure 4.10, this theory may only be applied with relatively small waves in deep water (another name for this theory is small amplitude wave theory). The approximation of waves with a simple sine function is a crude simplification of what is really happening. For an adequate understanding however, the linear theory is very useful and attractive, since it gives a simple, but complete description of the pressure and velocity field. Calculated values outside the range of validity can serve as an indication. For the purpose of a preliminary design, this indication is often enough and for a detailed design, more accurate computations or laboratory tests can be done.

Starting point is the Navier-Stokes equation (in two dimensions), neglecting boundary effects. Outside the boundary layer viscosity can be neglected and the flow can be considered irrotational, so a velocity potential can be defined:

$$u = \frac{\partial \phi}{\partial x} \qquad w = \frac{\partial \phi}{\partial x} \tag{4.12}$$

Conservation of mass and momentum give the following equations:

$$\begin{gathered} \frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \\ \frac{\partial \phi}{\partial t} + \frac{p}{\rho} + gz + \frac{1}{2}\left(\frac{\partial \phi}{\partial x} + \frac{\partial \phi}{\partial z}\right)^2 = 0 \end{gathered} \tag{4.13}$$

the basic equations can be solved when a surface-profile is assumed (see also Figure 4.9B) and taking the boundary conditions at bottom and surface into account:

$$\eta = \frac{H}{2} \sin\left(\frac{2\pi t}{T} - \frac{2\pi x}{L}\right) = a \sin(\omega t - kx) = a \sin\theta \tag{4.14}$$

leading to:

$$\phi(x,z,t) = \frac{\omega a}{k} \frac{\cosh k(h+z)}{\sinh kh} \cos(\omega t - kx) \tag{4.15}$$

Note: z = 0 represents the still water level and z is positive upwards.

From this expression for the potential ϕ as a function of x, z and t, all other quantities in the wave can be derived. In the following table (Table 7.1), a summary of the various parameters is presented. The extremes for deep and shallow water in the table follow from the values for transitional waterdepth, since for deep water: $kh \rightarrow \infty$, so: tanh = 1, sinh = ∞ and cosh = ∞ and for shallow water: $kh \rightarrow 0$, so: tanh = kh, sinh = kh and cosh = 1, see Figure 4.9C.

With the table on page 23, it is possible to determine the wave length and other values that can be handy in wave computations, like subsurface pressures or orbital velocities. Starting with a known waterdepth, h, and wave period, T, it is easy to compute the relation h/L0 from $L0 = gT^2/2\pi$. With this value, the relation h/L can be found in the table from which the local wave length can be computed.

Relative depth Characteristics	Shallow Water $\frac{h}{L} < \frac{1}{20}$	Transitional water $\frac{1}{20} < \frac{h}{L} < \frac{1}{2}$	Deep Water $\frac{h}{L} > \frac{1}{2}$
Wave Celerity	$c = \frac{L}{T} = \sqrt{gh}$	$c = \frac{L}{T} = \frac{gT}{2\pi} \tanh kh$	$c = c_0 = \frac{L}{T} = \frac{gT}{2\pi}$
Wave Length	$L = T\sqrt{gh}$	$L = \frac{gT^2}{2\pi} \tanh kh$	$L = L_o = \frac{gT^2}{2\pi}$
Group Velocity	$c_g = c = \sqrt{gh}$	$c_g = nc = \frac{1}{2}\left[1 + \frac{2kh}{\sinh 2kh}\right] * c$	$c_g = \frac{1}{2} c_0 = \frac{gT}{4\pi}$
Energy Flux (per m width)	$F = Ec_g = \frac{1}{2}\rho g a^2 \sqrt{gh}$	$F = Ec_g = \frac{1}{2}\rho g a^2 nc$	$F = \frac{T}{8\pi}\rho g^2 a^2$
Particle velocity horizontal vertical	$u = a\sqrt{\frac{g}{h}} \sin\theta$ $w = \omega a\left(1 + \frac{z}{h}\right)\cos\theta$	$u = \omega a \frac{\cosh k(h+z)}{\sinh kh} \sin\theta$ $w = \omega a \frac{\sinh k(h+z)}{\sinh kh} \cos\theta$	$u = \omega a e^{kz} \sin\theta$ $w = \omega a e^{kz} \sin\theta$
Particle displacement horizontal vertical	$\xi = -\frac{a}{\omega}\sqrt{\frac{g}{h}} \cos\theta$ $\zeta = a\left(1 + \frac{z}{h}\right) \sin\theta$	$\xi = -a \frac{\cosh k(h+z)}{\sinh kh} \cos\theta$ $\zeta = a \frac{\sinh k(z+h)}{\sinh kh} \sin\theta$	$\xi = -a e^{kz} \cos\theta$ $\zeta = a e^{kz} \sin\theta$
Subsurface pressure	$p = -\rho g z + \rho g a \sin\theta$	$p = -\rho g z + \rho g a \frac{\cosh k(h+z)}{\cosh kh} \sin\theta$	$p = -\rho g z + \rho g a e^{kz} \sin\theta$

Table Summary of Wave Characteristics

$$a = \frac{H}{2} \qquad \omega = \frac{2\pi}{T} \qquad k = \frac{2\pi}{L} \qquad \theta = \omega t - kx$$

h/L_0	h/L	kh	tanh kh	sinh kh	cosh kh	K_s	n
0	0	0	0	0	1	∞	1
.005	.02836	.1782	.1764	.1791	1.0159	1.692	.9896
.010	.04032	.2533	.2480	.2560	1.0322	1.435	.9792
.015	.04964	.3119	.3022	.3170	1.0490	1.307	.9690
.020	.05763	.3621	.3470	.3701	1.0663	1.226	.9588
.025	.06478	.4070	.3860	.4184	1.0840	1.168	.9488
.030	.07135	.4483	.4205	.4634	1.1021	1.125	.9388
.035	.07748	.4868	.4517	.5064	1.1209	1.092	.9289
.040	.08329	.5233	.4802	.5475	1.1401	1.064	.9192
.045	.08883	.5581	.5066	.5876	1.1599	1.042	.9095
.050	.09416	.5916	.5310	.6267	1.1802	1.023	.8999
.055	.09930	.6239	.5538	.6652	1.2011	1.007	.8905
.060	.1043	.6553	.5753	.7033	1.2225	.9932	.8811
.065	.1092	.6860	.5954	.7411	1.2447	.9815	.8719
.070	.1139	.7157	.6144	.7783	1.2672	.9713	.8627
.075	.1186	.7453	.6324	.8162	1.2908	.9624	.8537
.080	.1232	.7741	.6493	.8538	1.3149	.9548	.8448
.085	.1277	.8026	.6655	.8915	1.3397	.9481	.8360
.090	.1322	.8306	.6808	.9295	1.3653	.9422	.8273
.095	.1366	.8583	.6953	.9677	1.3917	.9371	.8187
.100	.1410	.8858	.7093	1.006	1.4187	.9327	.8103
.110	.1496	.9400	.7352	1.085	1.4752	.9257	.7937
.120	.1581	.9936	.7589	1.165	1.5356	.9204	.7776
.130	.1665	1.046	.7804	1.248	1.5990	.9169	.7621
.140	.1749	1.099	.8002	1.334	1.667	.9146	.7471
.150	.1833	1.152	.8183	1.424	1.740	.9133	.7325
.160	.1917	1.204	.8349	1.517	1.817	.9130	.7184
.170	.2000	1.257	.8501	1.614	1.899	.9134	.7050
.180	.2083	1.309	.8640	1.716	1.986	.9145	.6920
.190	.2167	1.362	.8767	1.823	2.079	.9161	.6796
.200	.2251	1.414	.8884	1.935	2.178	.9181	.6677
.210	.2336	1.468	.8991	2.055	2.285	.9205	.6563
.220	.2421	1.521	.9088	2.178	2.397	.9231	.6456
.230	.2506	1.575	.9178	2.311	2.518	.9261	.6353
.240	.2592	1.629	.9259	2.450	2.647	.9291	.6256
.250	.2679	1.683	.9332	2.599	2.784	.9323	.6164
.260	.2766	1.738	.9400	2.755	2.931	.9356	.6076
.270	.2854	1.793	.9461	2.921	3.088	.9390	.5994
.280	.2942	1.849	.9516	3.097	3.254	.9423	.5917
.290	.3031	1.905	.9567	3.284	3.433	.9456	.5845
.300	.3121	1.961	.9611	3.483	3.624	.9490	.5777
.320	.3302	2.075	.9690	3.919	4.045	.9553	.5655
.340	.3468	2.190	.9753	4.413	4.525	.9613	.5548
.360	.3672	2.307	.9804	4.974	5.072	.9667	.5457
.380	.3860	2.425	.9845	5.609	5.697	.9717	.5380
.400	.4050	2.544	.9877	6.329	6.407	.9761	.5314
.420	.4241	2.665	.9904	7.146	7.215	.9798	.5258
.440	.4434	2.786	.9924	8.075	8.136	.9832	.5212
.460	.4628	2.908	.9941	9.132	9.186	.9860	.5173
.480	.4822	3.030	.9953	10.32	10.37	.9885	.5142
.500	.5018	3.153	.9964	11.68	11.72	.9905	.5115

4.2.4 *Waves on coasts*

General

When waves approach the coast, several parameters in the wave-motion change, mainly because of the changing depth. When the coast is a (nearly) vertical wall, the wave will be ***reflected***, regardless of any change in depth. The wave propagation speed, the celerity, become less in shallower water and so does the wave length. Neglecting bottom friction, hence a constant energy flux, this means that the wave height will first decrease and in really shallow water increase considerably, the so-called ***shoaling***. When waves approach the coast with an angle, the shallowing will come together with ***refraction***. In very shallow water, the wave can not longer exist and it will ***break***. Behind a breakwater, an isthmus, or at any place where there is a difference in wave height perpendicular to the direction of propagation, ***diffraction*** will occur. Processes like reflection, refraction and diffraction occur for any type of waves, like sound or light and the phenomena are very much the same. One parameter does not change during all these processes: the (individual) wave period. When waves approach the coast continuously, the number of waves during a certain time interval that will pass a certain line will be the same number that arrive at the coast, from which follows that the wave period will remain the same. It is possible, however, that higher order waves are generated due to friction, so beside the "basic" period, other super- and subharmonics could be found. This also means that it is possible that in an irregular wave field, the wave period that is characteristic for the whole wave field, can change somewhat. The mentioned phenomena will now be described briefly.

Shoaling

When waves enter shallower water and the asssumption is made that there is no energy loss, the energy flux will be constant. From the table on page 22, it is found:

$$F = E c_g = E n c = \frac{1}{8} \rho g H^2 * \frac{1}{2}\left[1 + \frac{2kh}{\sinh 2kh}\right] * \frac{gT}{2\pi} \tanh kh \qquad (4.16)$$

note that now the wave height is used instead of the amplitude. From the energy flux being constant, the change in wave height can now be computed. Usually this is expressed in the relation between the wave height in waterdepth h and in deep water, also called the shoaling coefficient, K_{SH}:

$$K_{SH} = \frac{H_h}{H_0} = \sqrt{\frac{1}{\tanh kh\,(1 + \frac{2kh}{\sinh 2kh})}} \qquad (4.17)$$

In the table on page 23, K_{SH} is directly given as H/H_0.

Refraction

When waves approach shallower water with their crests at an angle to the depth contours, the wave crests appear to curve so as to decrease this angle. This results from the fact that the wave celerity decreases as the waterdepth decreases. In deep water refraction does not take place, since the wave speed there is independent of waterdepth. The same phenomenon is known in optics, where light rays passing from one medium to another, change direction, due to the different transmission velocity. At coasts there is a gradual change in celerity , leading to a gradual change in wave direction and curved wave crests and rays or orthogonals.

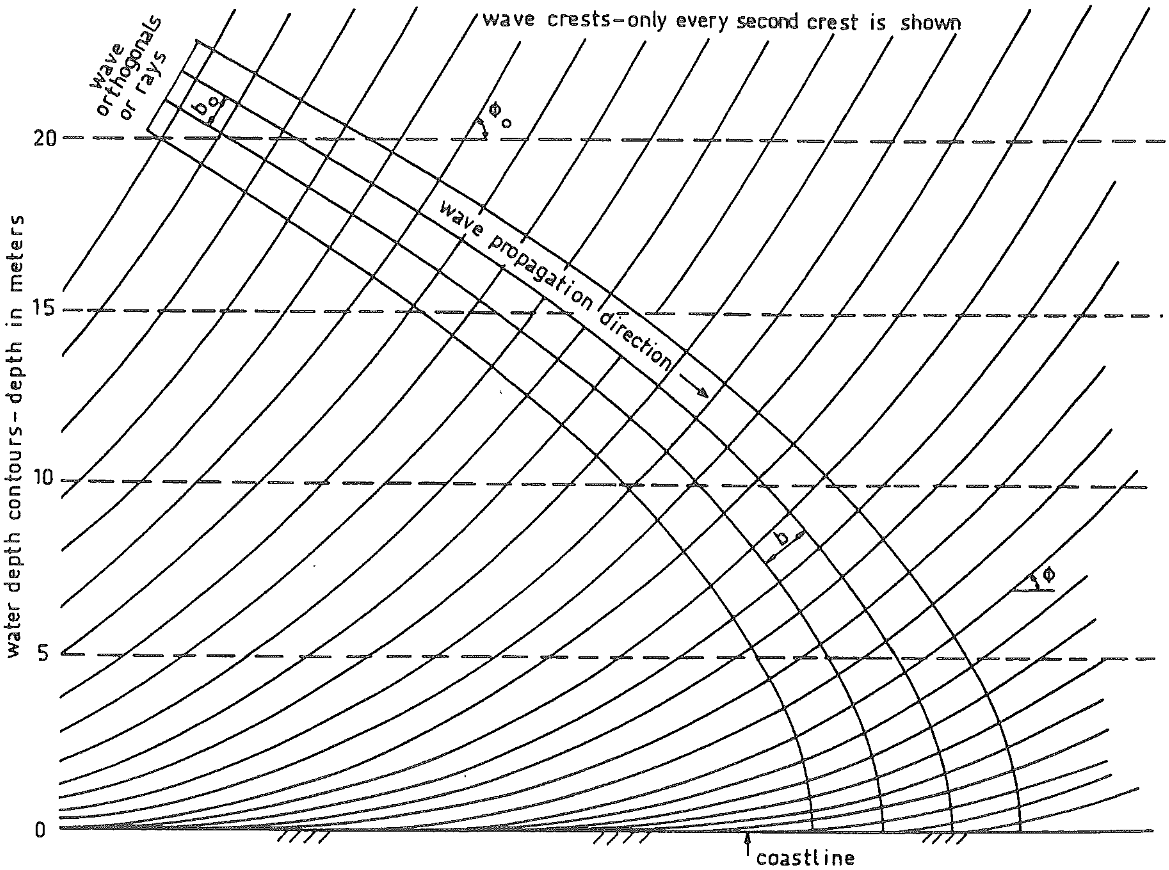


Figure 4.11 Refraction pattern

Figure 4.11 shows the principles of refraction at a coast with parallel and evenly spaced depth contours. It can easily be seen that the wave crests tend to become parallel to the depth contours and that the distance between the wave rays increases as the water becomes shallower. The change in wave direction can be derived from Snell's law (Snell is American for Snellius, a Dutch scientist who's name was also used for two expeditions in Indonesia in the thirties and eighties of this century):

$$\frac{\sin\phi}{\sin\phi_0} = \frac{c}{c_0} \qquad \textit{and:} \qquad \frac{b}{b_0} = \frac{\cos\phi}{\cos\phi_0} \tag{4.18}$$

The effect of refraction on wave height is computed again by assuming a constant energy flux between two orthogonals:

$$Fb = Encb \quad \rightarrow \quad \frac{H_h}{H_0} = K_{SH}\sqrt{\frac{b_0}{b}} = K_{SH}K_R \tag{4.19}$$

From this equation and the widening of the distance between wave rays, it can be seen that refraction decreases the wave height as the water becomes shallower.

******** In the final version of this manual information on graphical and digital techniques to determine refraction patterns should be included. Also the problem of crossing wave rays and models to avoid that must be part of the information in the manual ********

Breaking

Breaking of a wave is instability and means in fact that the wave cannot longer exist. The particle velocity u becomes greater than the celerity c. The particles "come out of the wave profile". This instability makes the flow highly complex and lends itself hardly to computation. Breaking means also energy dissipation, where kinetic energy is transformed into heat, with a high degree of turbulence. A dimensionless parameter which is of crucial importance to all kinds of problems in shore protection is the so-called Irribarren number, given by:

$$\xi = \frac{\tan\alpha}{\sqrt{H/L_0}} \tag{4.20}$$

This parameter indicates that the notions "steep" and "gentle" for a slope are relative ones. A slope of 1:100 is very "gentle", but for a tidal wave it can be just as steep as a vertical wall for a wave with a period of a few seconds. This is illustrated by the fact that a tidal wave on a beach does not break and is completely reflected (the phenomena described in this section are completely different from those in tidal waves; the example only shows the relativity of all notions in hydrodynamics). $\sqrt{(H/L_0)}$ can roughly be seen as the wave steepness (roughly, because H is the local wave height and L_0 is the deep-water wavelength). This indicates that an incoming wave that is already very steep will easily break, even at a very steep slope. Another way to look at L_0 is that it represents the influence of the wave period, since $L_0 = gT^2/2\pi$.

For different values of ξ, waves break in a completely different way. Figure 4.12 shows the various types. The transition between breaking and non-breaking lies around $\xi \approx 2.5$ - 3. For higher values the wave surges up and down the slope with minor air entrainment. The behaviour of the waves from $\xi \approx 3$ to 5, is therefore often called surging breaker although it can be questioned whether this is a breaking or a standing wave. A collapsing breakers could be seen as the transition between breaking and non-breaking. The most photogenic breaker is the plunging type ($\xi \approx 0.5$ to 3). In plunging breakers the crest becomes strongly asymmetric; it curls over, enclosing an air pocket, after which it impinges on the slope like a water jet. With decreasing slope angle, the crest of a plunging breaker becomes less asymmetric and the forward projected jet of water from the crest becomes less and less pronounced. This leads to the spilling breaker type ($\xi <$ 0.3). The transition between the various breaker types is gradual and these values for ξ

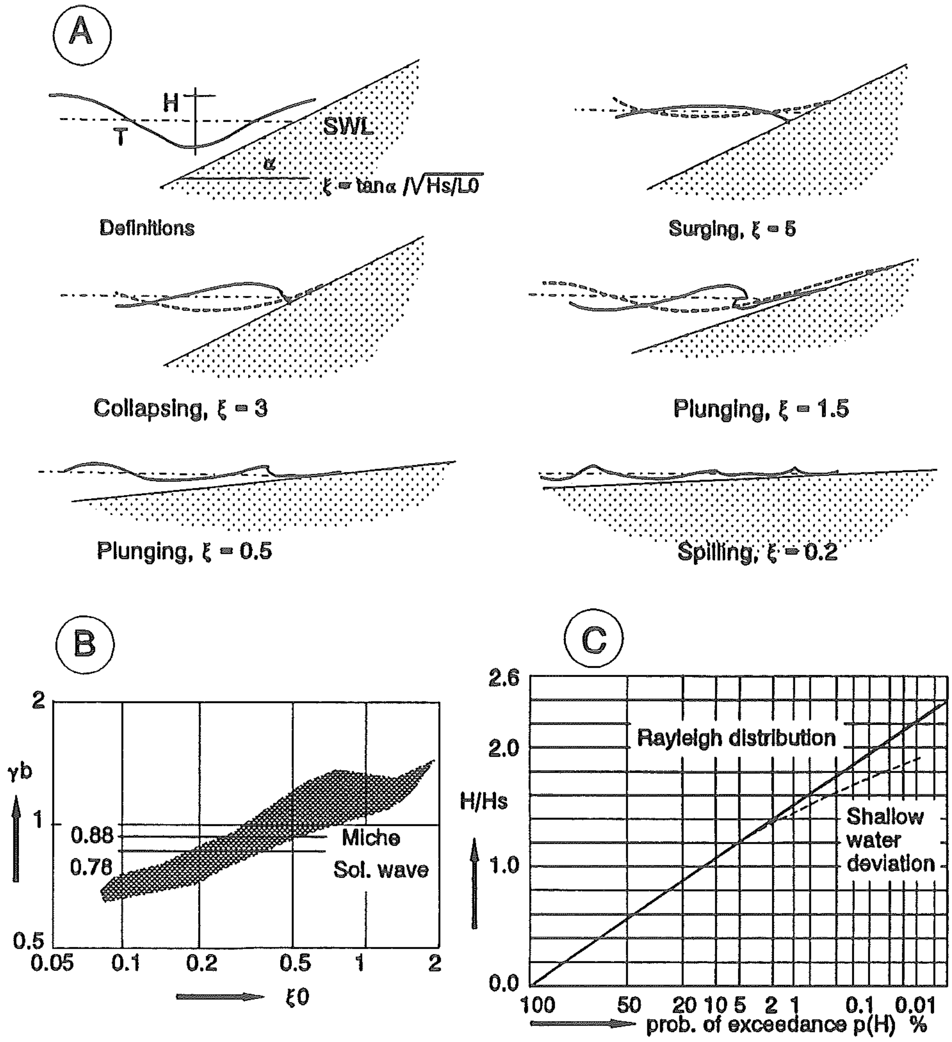


Figure 4.12 Breaker types and influence of waterdepth on breaking

are just an indication. For more detail, see Battjes,1974. After breaking, the wave travels on with a celerity about equal to $\sqrt{(gh)}$ and the wave behaviour can be described by a bore or a moving hydraulic jump, see for more detail Fredsøe,1992.

For a horizontal bottom, the solitary wave theory gives a relation for breaking between the wave height and the waterdepth: $\gamma_b = H/h \approx 0.78$, while equation (4.11) gives $\gamma_b = 0.88$. For slopes, γ_b depends on ξ. Figure 4.12B shows some experimental results (using ξ_0 with the deep water wave height H_0, since H is hard to define at the point of breaking). For low ξ these seem in reasonable agreement with the solitary wave theory; for steeper slopes, the waves break at a smaller depth. Apparently, the waves need time to break, in which they can travel to a smaller depth. These values are valid for individual waves; for irregular waves $\gamma_b = H_S/h \approx 0.5$ can be seen as a reasonable first guess on a mild sloping foreshore. Since the highest waves are broken in this situation, the wave heights are no longer Rayleigh distributed, see Figure 4.12C. $H_{1\%}$ e.g. is $1/(1 + H_S/h)^{1/3}$ times smaller than would follow from the Rayleigh-distribution, see also CUR/CIRIA,1991.

******** In the final version of this manual more details on breaking irregular waves on coasts should be inluded, e.g. the Battjes-Jansen model, which gives the wave heights in the breakerzone for an irregular wave field ********

Reflection

Progressive waves travel on without being halted. Walls, dikes and beaches do stop the waves, either by reflecting or absorbing them. Vertical walls act like a mirror: the incoming waves are sent back and the result is superposition of two progressive waves with celerity +c and -c. This leads to a stand-still of the resulting wave, hence the name standing wave. The height is twice of that of the incoming wave.

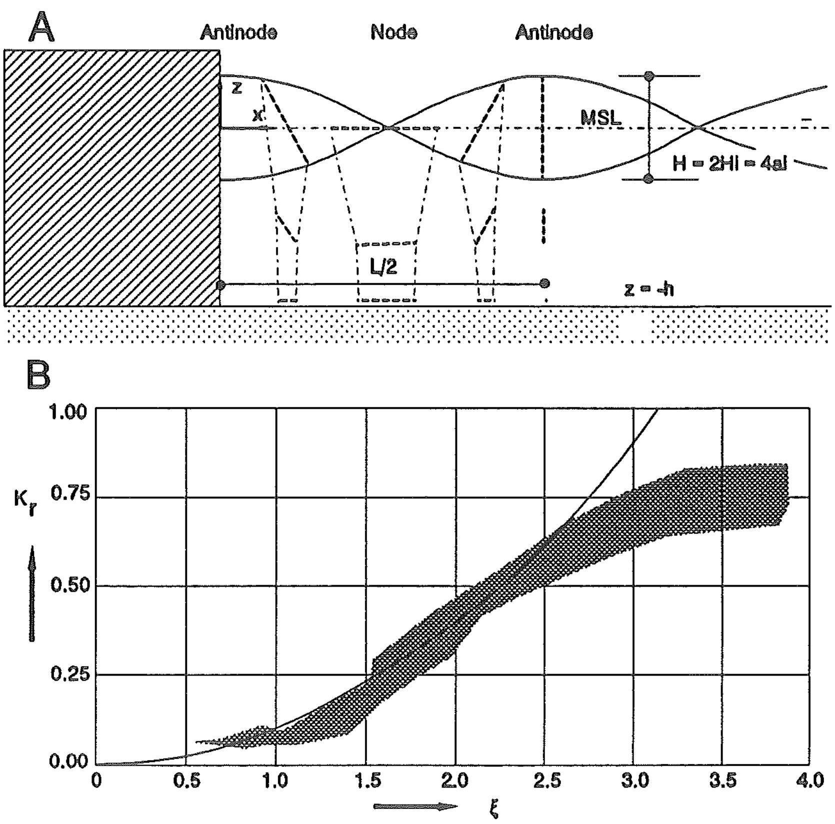


Figure 4.13 Standing wave (A) and reflection on slopes (B)

Figure 4.13A gives the pattern of a standing wave (or "clapotis"). Note that the orbits of the particle movement have degenerated into straight lines. At the antinodes there is only a vertical and at the nodes a horizontal movement. The wave profile for a standing wave in first approximation according to linear theory is given by:

$$\eta = a_s \cos kx \sin \omega t = 2a_i \cos kx \sin \omega t \tag{4.21}$$

in which H_i is the height of the incoming wave. Other characteristic expressions for standing waves are:

$$u = \omega a_s \frac{\cosh k(h+z)}{\sinh kh} \sin kx \cos \omega t$$
$$w = \omega a_s \frac{\sinh k(h+z)}{\cosh kh} \cos kx \sin \omega t \qquad (4.22)$$
$$p = -\rho g z + \rho g a_s \frac{\cosh k(h+z)}{\cosh kh} \cos kx \sin \omega t$$

Vertical walls fully reflect incoming waves. When the wall tilts backwards, the vertical motion of the water at the interface increases: the maximum waterlevel becomes higher than H_S as is the case in Figure 4.13. But the standing wave pattern remains till the waves break (depending on the slope and the wave steepness).

The amount of reflection is defined as the height of the reflected wave in relation to the incoming wave height: K_R (reflection-coefficient) = H_R/H_I. It can be reasoned that K_R is proportional to ξ^2. Experimentally was found $K_R \approx 0.1\ \xi^2$, see Figure 4.13B, for values of ξ below the breaking limit. For $\xi > 2.5$, K_R slowly tends to 1, the value for complete reflection.

Run-up

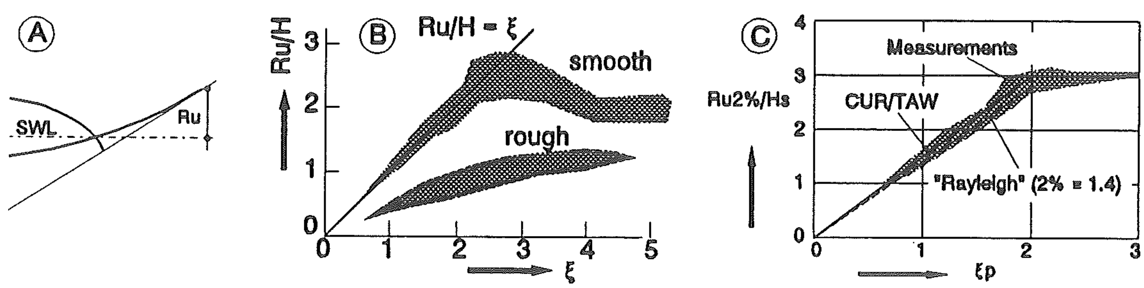


Figure 4.14 Run-up, definitions and empirical data (regular and irregular waves)

Run-up is defined as the maximum waterlevel on a slope during a wave period with regard to the still water level, see Figure 4.14A. For smooth slopes the formula of Hunt gives for breaking regular waves ($\xi < 2.5$-3):

$$\frac{R_u}{H} = \xi \qquad (4.23)$$

Rip rap slopes give values which are about 50% lower, see also Figure 4.14B. Run-up appears to be maximum around $\xi = 2.5$-3, which means just at the transition between breaking and non-breaking. For $\xi \rightarrow \infty$ the theoretical value would be $R_U = H$ (standing wave).

Most wave run-up formulae for irregular waves use a value in the distribution that is exceeded by only a small number of the waves, because usually the considered slope is meant to protect something and not too much water should pass the slope. In the

Netherlands, $R_{u2\%}$ is used in the design of seadikes. Empirically it was found, see Figure 4.14C and CUR/TAW,1992:

$$R_{u2\%} = 1.5\, H_S\, \xi_P \quad (R_{u2\%\,\max} = 3\, H_S) \tag{4.24}$$

The maximum when $\xi > 2$ is similar what was found for regular waves, see Figure 4.14. Note: When simply applying the Rayleigh distribution for the 2% highest waves on the expression for regular waves, equation (4.23), a similar expression as equation (4.24) is found with a factor 1.4 instead of 1.5.

This run-up formula is valid for smooth slopes like asphalt or smooth concrete blocks. For a rip-rap slope, again, a reduction factor of $\gamma_r \approx 0.5$ is found.

When the wave attack is not perpendicular to the slope ($\beta = 0$), a reduction factor is applied: $\gamma_\beta = \sqrt{\cos\beta}$ (from the energy flux, which is $\propto H^2\cos\beta$) for long crested waves like swell or ship's waves, with a minimum of 0.6. For short crested waves (wind waves, which are not unidirectional like swell): $\gamma_\beta = 1 - 0.0022\beta$ (β in degrees).

Diffraction

A wave train which meets an obstacle, such as a breakwater or an island, may be reflected, but the wave crests can also bend around the obstacle and thus penetrate in the zone to the lee of the obstacle (the "shadow" area in terms of light). This phenomenon is called diffraction. Diffraction is important since it determines the wave penetration in a sheltered area e.g. behind a detached breakwater or in a harbour. The degree of diffraction depends on the ratio of a characteristic lateral dimension of the obstacle to the wavelength. Figure 4.15A and B show sketches of the wave crest pattern around an obstacle and through a gap. The reflected waves are omitted in these sketches.

The mathematical theory of diffraction is outside the scope of this manual. Only applications will be presented.

******** In this preliminary version only very roughly, in the final version this should be extended with graphical and numerical solutions.********

Figure 4.15C and D give a cross section and a plan view respectively of the wave heights in the lee zone and in the exposed zone. These wave heights are expressed as the diffraction coefficient, $K_D = H/H_I$, the local wave height divided by the incoming wave height. From these figures can be seen that at the so-called shadow line (separating the exposed zone and the lee zone), there is not a discontinuity, but a smooth transition with a value $K_D = 0.5$. Inside the shadow zone the amplitude decreases monotonically, while in the exposed zone it oscillates around 1 with a maximum of approximately 1.15. Graphs like these can be made for various situations, see e.g. SPM,1984.

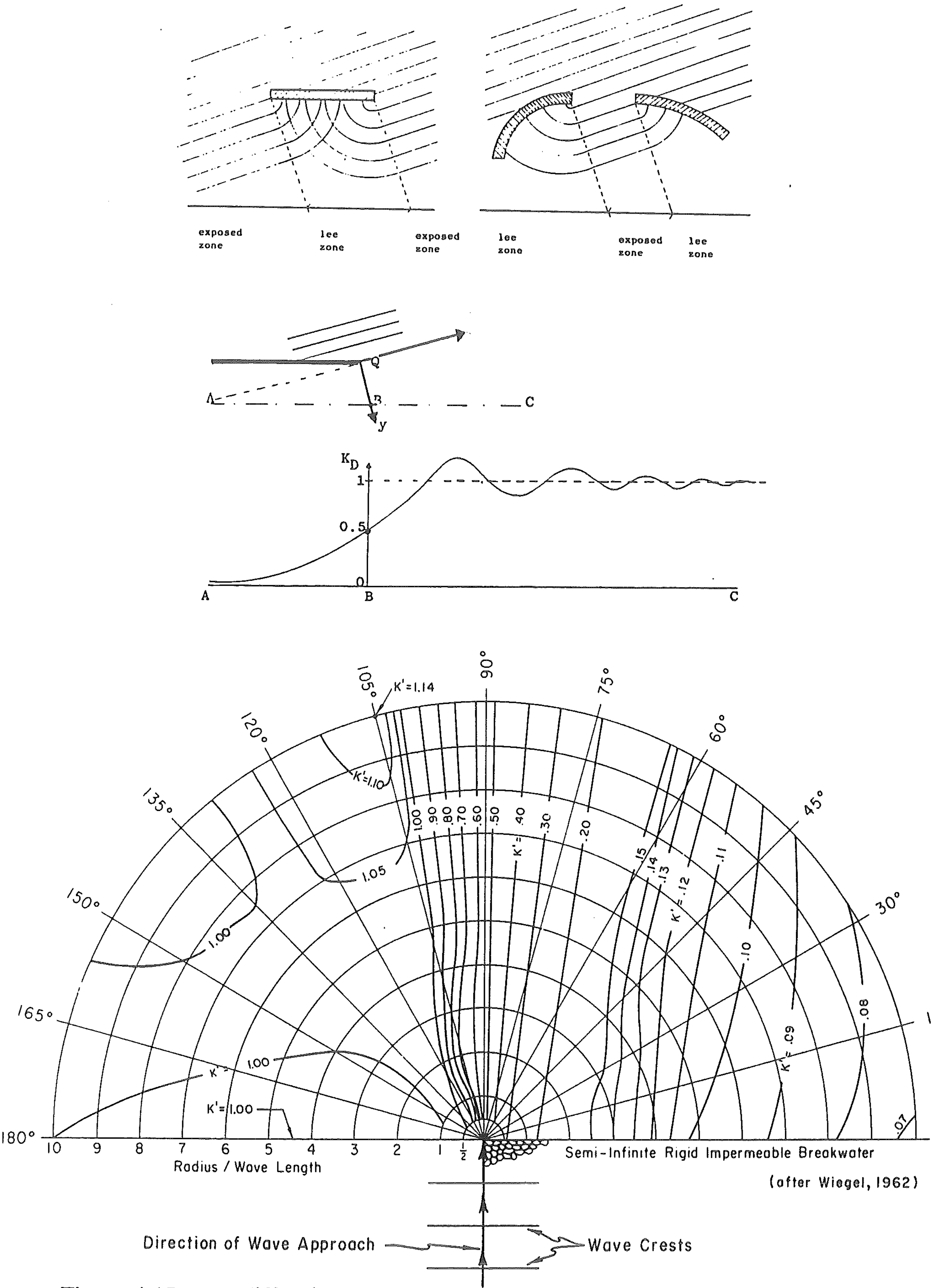


Figure 4.15 Wave diffraction

4.2.5 *Wave currents*

Waves, approaching a coastline under an oblique angle, cause a transport of sediment in longshore direction. The breaking waves cause a current parallel to the coastline, and this current transports sediment along the coast. Differences in longshore transport along the coast cause erosion and sedimentation. In order to assess the stability of a coastline it is therefore important to understand the longshore sediment transport. In some cases along a coast there is also a strong tidal current. Tidal currents also may transport sediment along the coast; this is especially important for mud. However in the case of a sandy coast, the effect of tidal longshore transport is less important than the longshore transport caused by breaking waves.

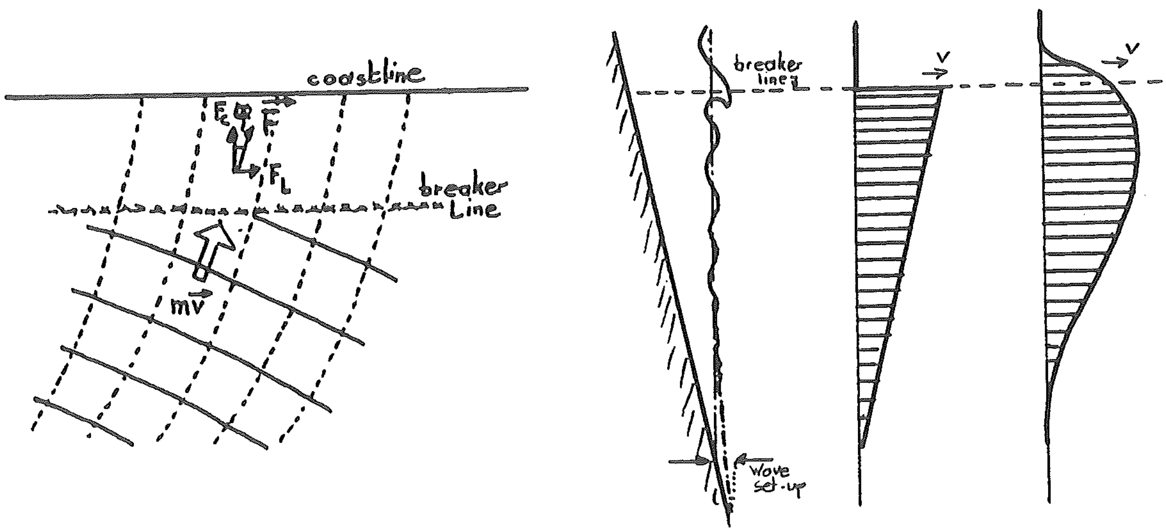


Figure 4.16 Wave momentum and distribution of longshore velocity

In basic mechanics, there are two important conservative entities, energy and momentum. Conservative means that they cannot disappear, only transformed. During the process of wave-breaking most of the wave energy is transformed into heat and noise, two properties of energy difficult to describe. In order to investigate what happens after breaking, it is therefore more convenient to look to momentum. The change of momentum is described by Newton as

$$\vec{F} = m \cdot \vec{a} = \frac{d(m\vec{v})}{dt} \tag{4.25}$$

Two important observations can be made from this equation. First that momentum is a vector, so the direction is important. The other observation is that a change in *(mv)* leads to a force. In simple words we may conclude that if a wave has to stop, because it arrives near the coastline, its momentum is transformed into a force. This force we can split up in a component parallel to the coast and a component perpendicular to the coast. The longshore component $F_l = F \sin\phi$, while the cross-shore component is $F_c = F \cos\phi$, see Figure 4.16A.

The magnitude of the momentum is a function of the wave height, so the force is a function of the wave height, de longshore component is therefore a function of both waveheight and wave direction:

$$F_L = f\,(H_b,\, \phi_b) \tag{4.26}$$

The cross-shore force F_c pushes the water towards the beach. Because of this pushing, the water level will raise somewhat, and the hydrostatic force of the raised water level will balance the force F_c. This raise of water level is called wave set-up.

The force F_L pushes the water along the shoreline. Because in longshore direction there is nothing to push against a hydrostatic force cannot be build up to balance the longshore force F_L. This means that the water will be accelerated in longshore direction. Consequently the water will start to flow parallel to the coast, the longshore current. When the water is flowing, a friction force F_f between water and bottom is activated. The magnitude of this force will increase with an increasing velocity. At a certain moment the friction force equals the longshore force, so $F_L = F_f$. At that moment the water is no longer accelerated, but there will be permanent flow with a constant velocity v_L. Because $F_L = f(H_b, \phi_b)$ and $F_f = f(v_L)$ it is possible to express v_L as a function of H_b and ϕ_b.

In order to derive the formulas for wave set-up and longshore current properly, it is better not to use the total force, but the force per square meter. So, instead of forces we have to deal with stresses. The stresses caused by breaking waves are called radiation stresses. Using these radiation stresses, Longuet-Higgins and Stuart derived the formulas for longshore current and wave set-up. Derivation of these stresses goes too much into detail, only the expressions for the radiation stresses perpendicular and parallel to the coast will be given here:

$$\begin{aligned} S_{yy} &= (2n - \frac{1}{2})\,E \\ S_{yx} &= E\,n \cos\phi \sin\phi \end{aligned} \tag{4.27}$$

in which S_{yy} is the radiation stress perpendicular to the coast, from which the wave et-up can be computed and S_{yx} the radiation stress parallel to the coast, from which the longshore current can be determined. E (wave energy) and n come from wave theory, see the table on page 22. Note that the longshore radiation stress is maximum when $\phi = 45°$ ($\sin\phi * \cos\phi$). This will be of importance for longshore sediment transport due to waves.

If one applies linear wave theory and a fixed ratio between wave height after breaking and the water depth, the velocity profile will have a triangular shape (see middle figure in Figure 4.16B). In reality the shape is more like in the right figure. This is due to the fact that

- the bottom is not plane, but there is usual a breaker bar
- non-linear effects in water and internal friction in water
- the waves are Rayleigh distributed and not monochromatic

In more advanced models of longshore current computation these aspects are also considered.

4.2.6 Waves in Indonesia

******** To be filled in later on, based on measurements and/or computations ********

The example on this page comes from Sulaiman et al,1994

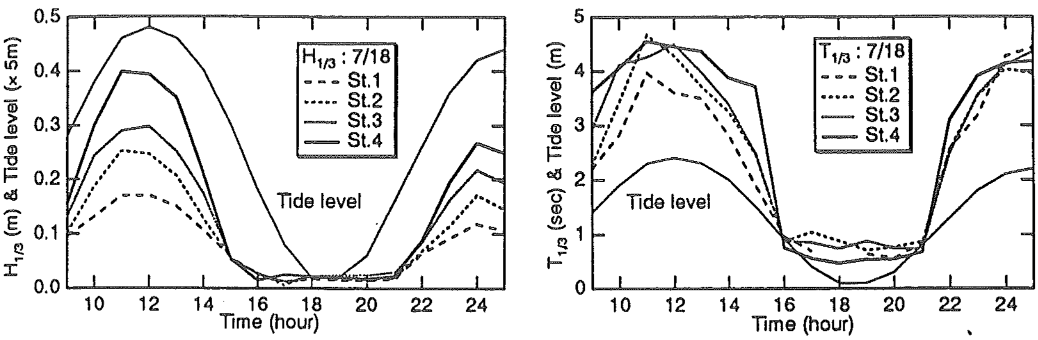
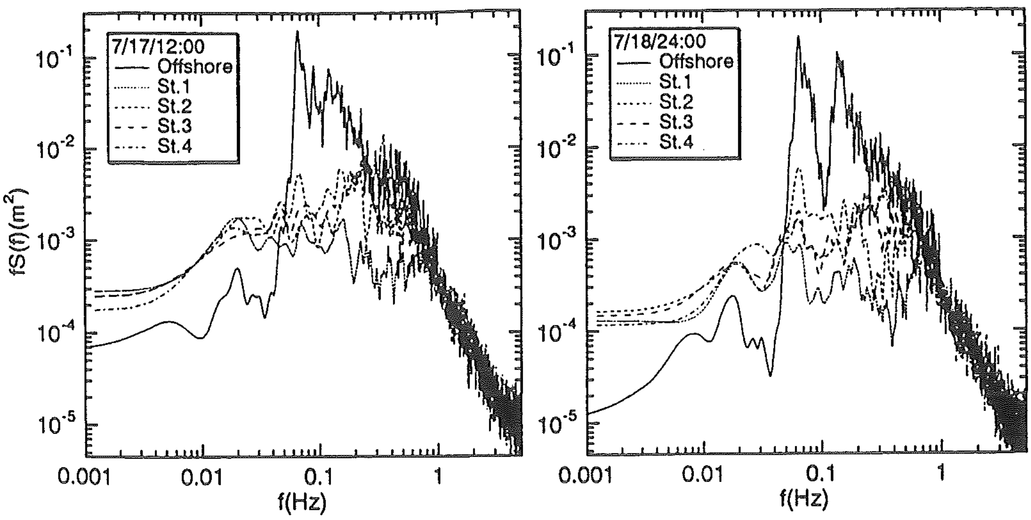


Figure 5. Significant wave heights (left) and periods (right), varying with the tide level on the coral flat (July 18).



(1) July 17 at 12:00

(2) July 18 at 24:00

Figure 6. Frequency spectra for waves in and outside the reef at high tide.

4.3 *Tides*

4.3.1 *Tidal generation*

Tides are generated in the oceans by the attracting forces between earth, sun and moon. A theoretical treatment of the tidal forces is outside the scope of this manual, see e.g. Here only the dominating constituents will be treated and demonstrated for Indonesia. It is not exaggerated to say that Indonesia is the only country in the world that has practically all features of tidal movement that can be found on earth: purely semi-diurnal and diurnal tides and everything in between, resonating bays with strong tidal amplification of the tides, tidal bores etc.

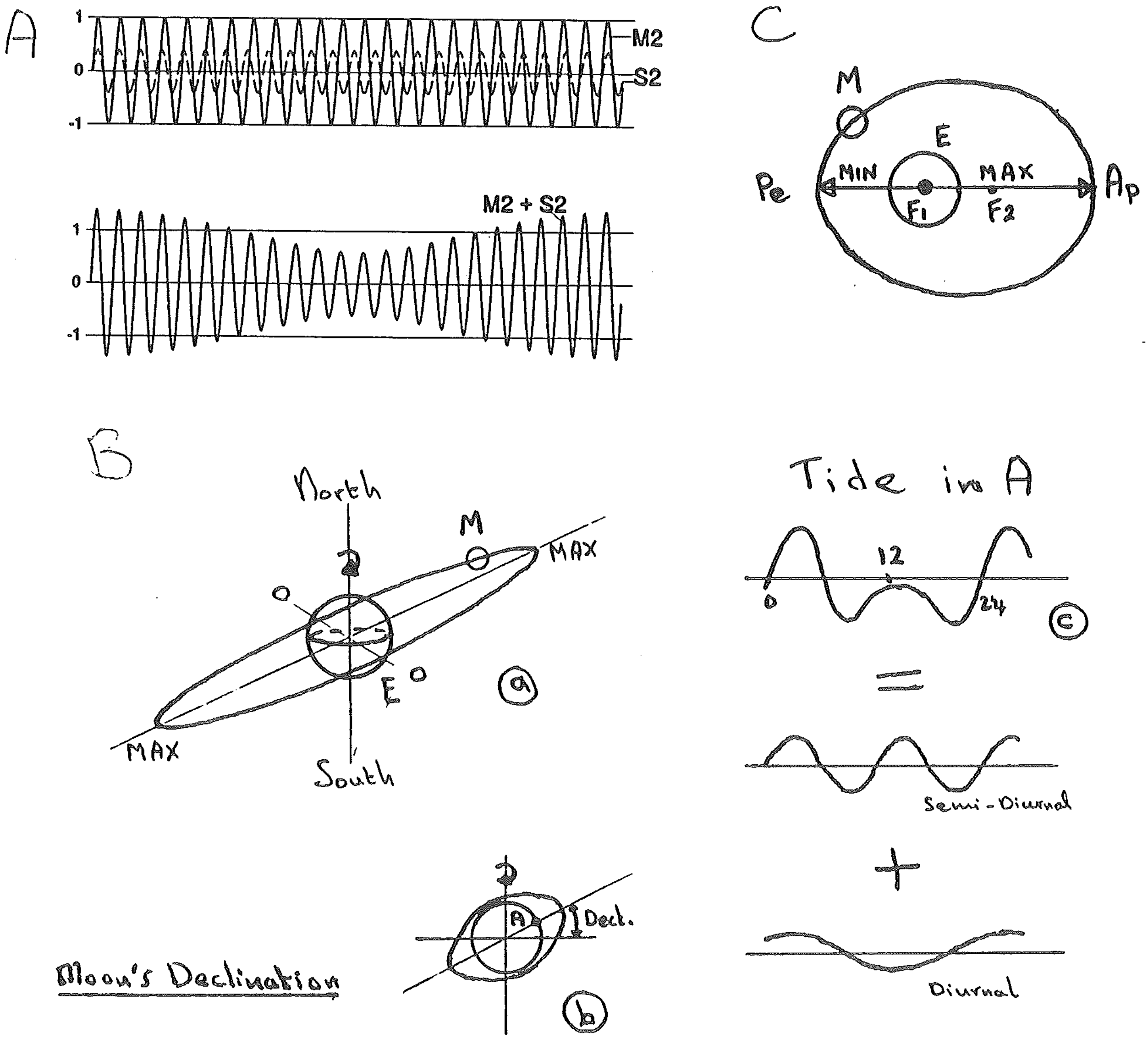


Figure 4.17 Spring and neaptides and movement of celestial bodies

The **principal** semi-diurnal tidal constituents are M2 and S2. They are the direct result of the attracting forces moon-earth and sun-earth respectively. M2 has a period of 12.42 hours and S2 of 12 hours. Together they form a so-called beat with a period of:

$$T_{BEAT} = \frac{T_1 * T_2}{|T_1 - T_2|} = \frac{12 * 12.42}{0.42} = 355 \; hour = 14.8 \; days \tag{4.28}$$

This is also demonstrated in Figure 4.17A where it is shown that, due to the (small) difference in period, the constituents alternately reinforce and weaken each other during the beat period: spring and neap tides, where springtides occur after full moon and new moon when the lunar and solar forces cooperate. More in general, beats between the various constituents make that the physical influences that strengthen and weaken the principal tides become apparent in tidal analysis and prediction.

The most important diurnal constituents are K1, O1 and P1, with periods of 23.93, 25.82 and 24.07 hours respectively. They are the result of the fact that the earth's orbit around the sun and the moon's orbit around the earth does not take place in the plane of the equator, but in an inclining plane, see Figure 4.17B. They are therefore named **declination** tidal constituents. K1 is of both solar and lunar origin, O1 is of lunar origin, while P1 is of solar origin. They form beats with again periods of about two weeks (K1 with O1 and P1 with O1). These form spring and neap tides in the diurnal tides due to the declination. Note that these spring and neaptides have nothing to do with full or new moon, but with the declination of the moon, which comes at a different moment! K1 and P1 have together a beat of 182.6 days or exactly half a year (check with equation (4.28). This takes care of the fact that the sun's declination is maximum in June and December, while the sun is in the plane of the equator in March and September. This will give stronger diurnal tides in June and December.

The declination of earth and moon also influences the principal semi-diurnal tides, because the direct attracting forces will be less when the declination is large. The main tidal constituent that takes care of this is K2 with a period of 11.97 hours. K2 is of both lunar and solar origin and has a beat together with M2 of 13.7 days and with S2 of 182.6 days, arising from the fact that moon and sun have a maximum declination twice during their orbit of 27.3 and 365.2 days respectively. For S2 and K2 this is easily to connect with the calendar: in March and September S2 and K2 reinforce each other, since the sun is in the equator and declination is zero; in June and December their influence is opposite and the result is a weaker semi-diurnal tide.

Beside declination of sun and moon with respect to the earth's equator, there is also influence of the distance between the celestial bodies during their orbit. This orbit is not a circle, but an ellipse, see Figure 4.17C, with the central body (earth and sun respectively) in one of the focal points of the ellipse. The distance, varying between a minimum, the so-called perigee, and a maximum, the apogee, again weakens or strengthens the principal semi-diurnal tides. The most important constituent of these so-called **elliptic** tides, that express this influence, is N2, which is of lunar origin and has a period of 12.66 hours. Together with M2 this has a beat of 27.3 days, which is again the orbital time of the moon around the earth, during which the distance has a maximum and a minimum once.

Finally, due to bottom friction, tidal waves generate during their travel through shallow water, higher harmonics: **shallow water** tides, e.g. M4 with a period of 6.21 hour.

4.3.2 *Tidal prediction*

When describing the origin of tides in the previous section, it was stated that a tide does not consist of one single vertical movement, but is a combination of various components, due to various influences. Each of these constituents, for a given place and time interval, is characterized by three factors:

- the amplitude a_j : the vertical difference in height between the highest (or lowest) level and the average level (m).
- the period ω_j : the time (in hours) required for a component effect to re-occur (expressed in degrees/hr).
- the phase lag α : the time between the passage of a celestial body (moon or sun) through the meridian of the place considered, and the real time of occurrence (expressed in degrees).

j : the constituent subscript

The nine most important constituents were discussed in the previous section and are summarized in the following table:

Symbol	Origin	ω_j (°/hr)	T (hr) (= 360°/ω_j)
M_2	Main lunar tide	28.98410	12.42
S_2	Main solar tide	30.0000	12.00
K_1	Sun/Moon declination tide	15.04107	23.93
O_1	Moon declination tide	13.94303	25.82
K_2	Sun/Moon declination tide	30.0821	11.97
N_2	Moon elliptic tide	28.4397	12.66
P_1	Sun declination tide	14.9589	24.07
M_4	Moon shallow water tide	57.9682	6.21
MS_4	Sun/Moon shallow water tide	58.9841	6.11

These constituents are the ones most frequently used, but the total number of constituents is theoretically infinite and can sometimes add up to more than two hundred. Their contribution to the tidal level in most places in the world, however, is relatively small.

Note: the data for ω_j and T are the same for any place in the world, at any time. The values of a_j and α_j are different for every place on earth and follow from field observations.

From the previous explanation it is now obvious, that in a certain place and for a given period when the amplitudes a_j and phase angles α_j are known, the tidal level h_t can be expressed as follows:

$$h_t = h_o + \sum_{j=1}^{9} \left[a_j \cos\left(\omega_j t - \alpha_j\right)\right] \tag{4.29}$$

in which:
h_t = water level at time t (m)
h_o = average water level (= the average of a long series of hourly tidal observations (m)
t = time considered (hr)

The phase angle α_j of each constituent consists of an astronomical argument, which can be calculated from the well known movements of earth, moon and sun and a so-called kappa number, which comes from observations at a certain place. For about 60 stations in the Indonesian archipelago (from Sabang to Merauke), the 9 tidal constituents from the table above, as far as relevant for that station, are known and predictions are given by HIDRAL every year, based on these constituents.

The values of the first four constituents of the table, are usually the most important and are used to determine the character of the tide by calculating the ratio f:

$$f = \frac{a_{K_1} + a_{O_1}}{a_{M_2} + a_{S_2}} \tag{4.30}$$

When	$f < 0.25$	SEMI-DIURNAL TIDE
	$0.25 < f < 1.50$	MIXED TIDE, predominantly SEMI-DIURNAL
	$1.50 < f < 3.00$	MIXED TIDE, predominantly DIURNAL
	$3.00 < f$	DIURNAL TIDE

Figure 4.18 gives examples of tidal predictions in four Indonesian stations, showing the rich variety of tides in the archipelago and the phenomena mentioned in section 4.3.1. Figure 4.18A also gives the astronomical data for the moon.

In Tanjung Pandan (Figure 4.18A), where the tide is purely diurnal, springtide is at March 19 and on June 22, which is just after maximum declination of the moon. K1 and P1 work together in June and against each other in March, giving much higher tides in June. In March at neaptide, there is hardly any waterlevel fluctuation.

In Muara Banyuasin (Figure 4.18B), mixed, predominantly diurnal, the counteracting diurnal tides in March make that the appearance during neaptide becomes even semi-diurnal, while in the other three cases the character is clearly diurnal.

At the Muara Batanghari (Figure 4.18C), mixed, predominantly semi-diurnal, the reverse is true. In three of the four cases the tide has a semi-diurnal appearance, while at springtide in June, the appearance is diurnal. Note the interaction between M2 and N2 related to the Perigee and Apogee of the moon.

In Bagan Siapi-api (Figure 4.18D), the tide is purely semi-diurnal and springtide is at March 10 and June 23, which comes after New Moon and Full Moon respectively. Note that S2 and K2 work together in March, and against each other in June. Again the beat between N2 and M2 is related to Paerigee and Apogee of the moon.

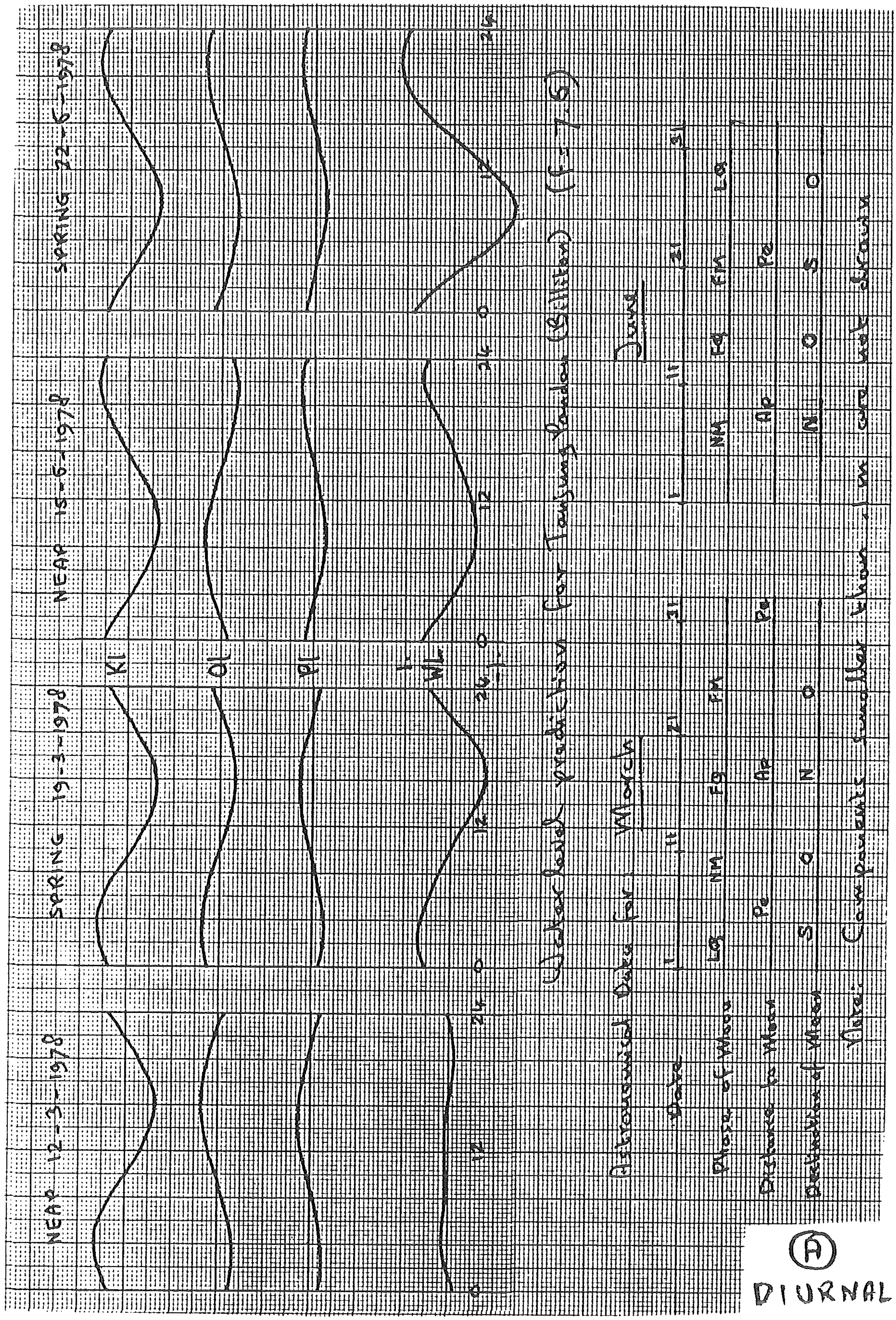
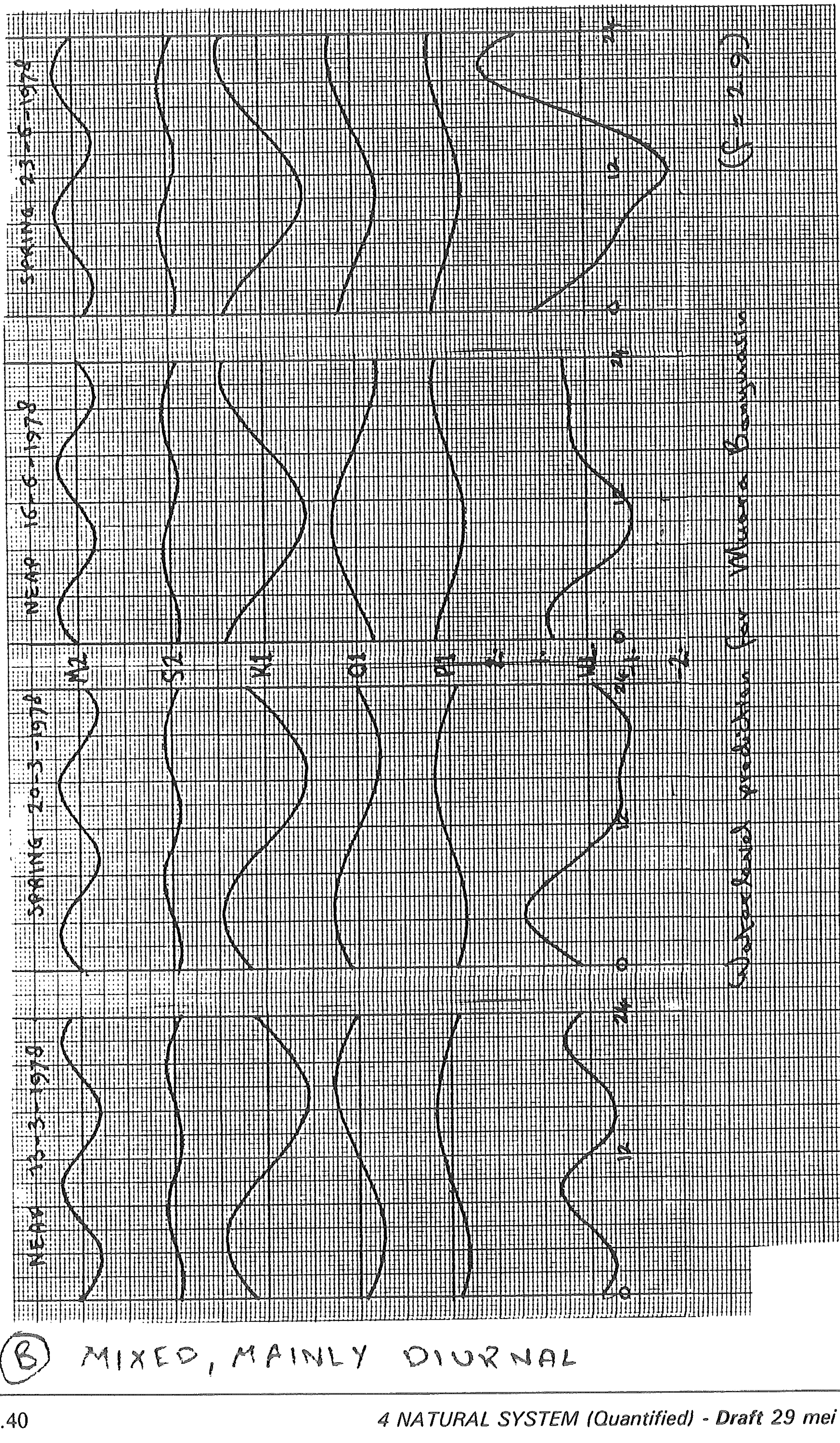
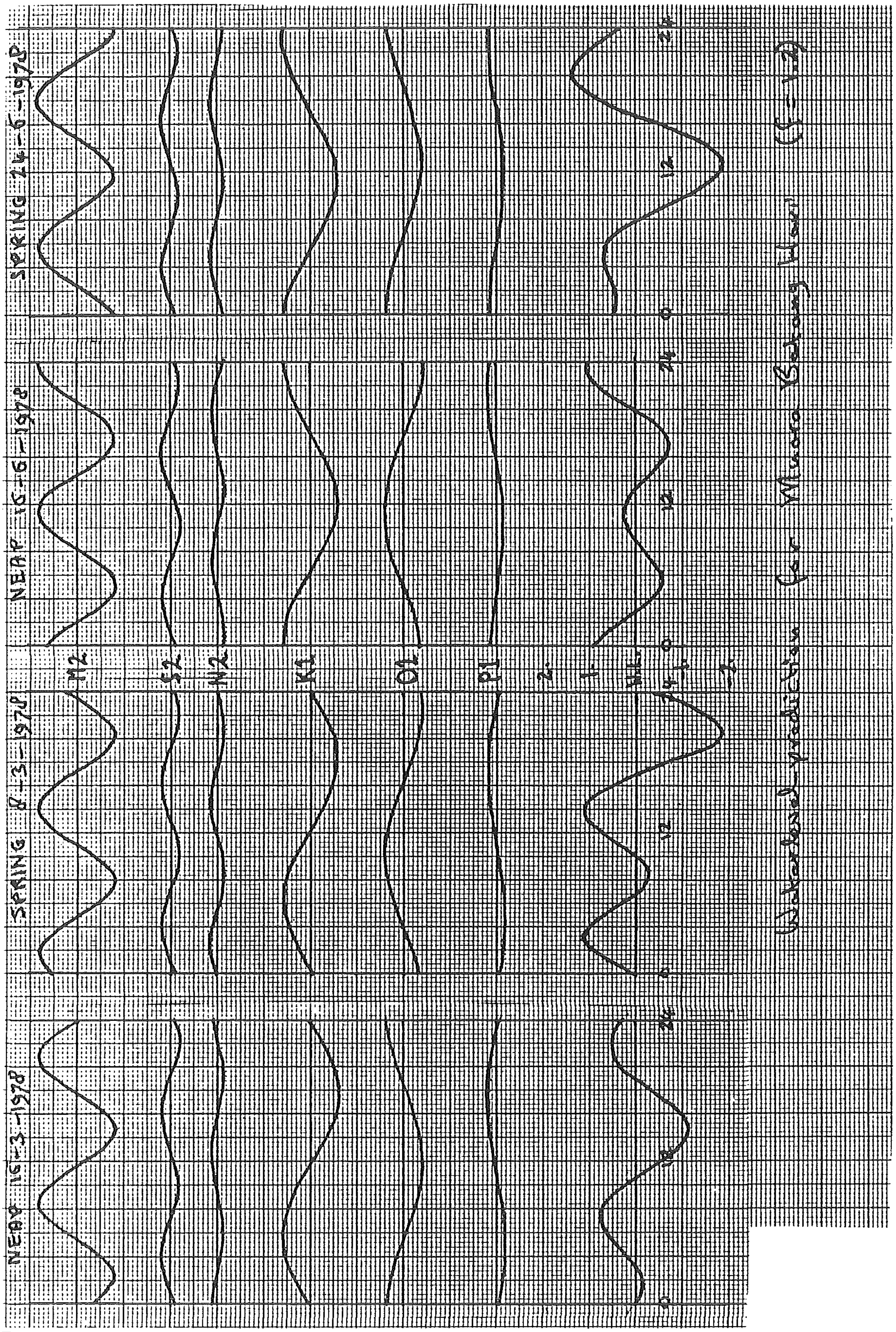


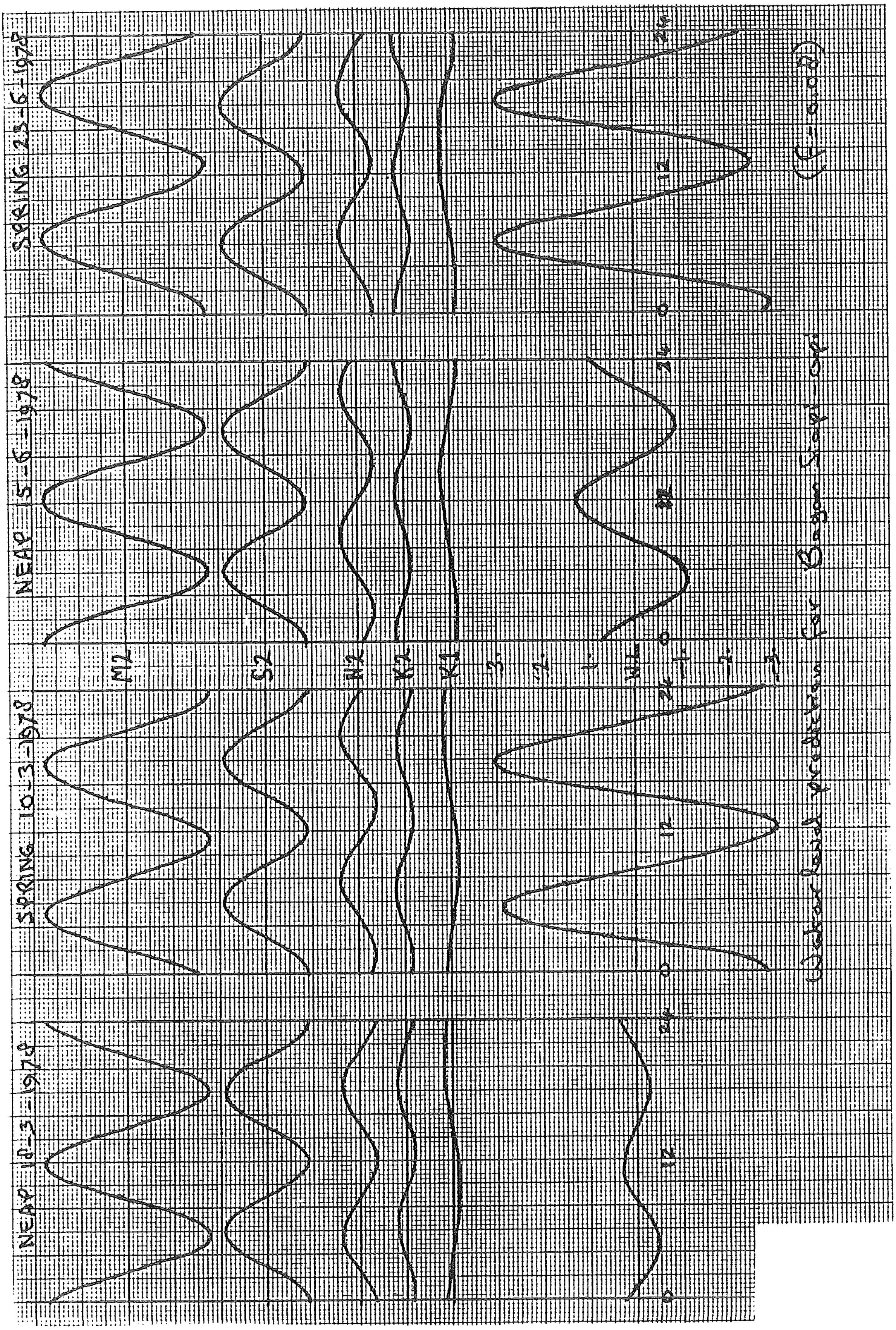
Figure 4.18 Spring- and Neap tides in March and June for four stations in Indonesia



(B) MIXED, MAINLY DIURNAL



(C) MIXED, MAINLY SEMI-DIURNAL



Ⓓ SEMI-DIURNAL

time of a tidal wave in a basin and, hence, is coupled to the length of the basin and the celerity of a long wave, which is in turn coupled to the depth ($c = \sqrt{gh}$). For seas, enclosed and open, this leads to:

$$\textit{Enclosed:} \quad T_N = \frac{2L}{N\sqrt{gh}} \qquad\qquad \textit{Open:} \quad T_N = \frac{4L}{M\sqrt{gh}} \tag{4.32}$$

in which L is the length of the sea or bay, with N = 1,2,3,4 .... and M = 1,3,5,7 .... See also Figure 4.19.

Tidal currents

When information about tidal currents or levels are necessary in a project, measurements have to be done to obtain insight in the values. But usually a coastal project will influence the waterlevels and currents. Therefore, tidal computations will have to be done, mostly with a numerical computer model. When the geometry is dominantly one-dimensional, like in a tidal river or estuary, a one-dimensional model will be sufficient. In most coastal problems however, the flow will be at least two dimensional. Both models are based on the equation of motion for gradually varying horizontal flow, in which friction is included, and the continuity equation. Figure 4.20 gives some examples.

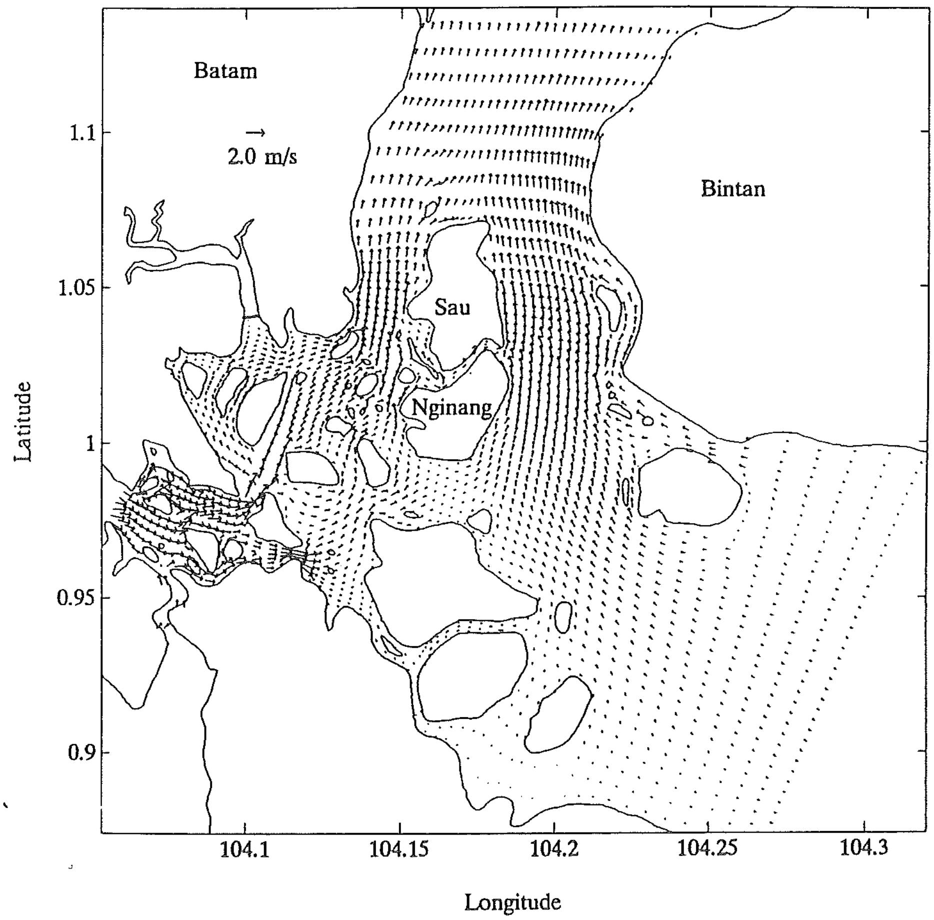


Figure 4.20 Example of tidal computation (2-dimensional)

4.3.4 *Tides in Indonesia*

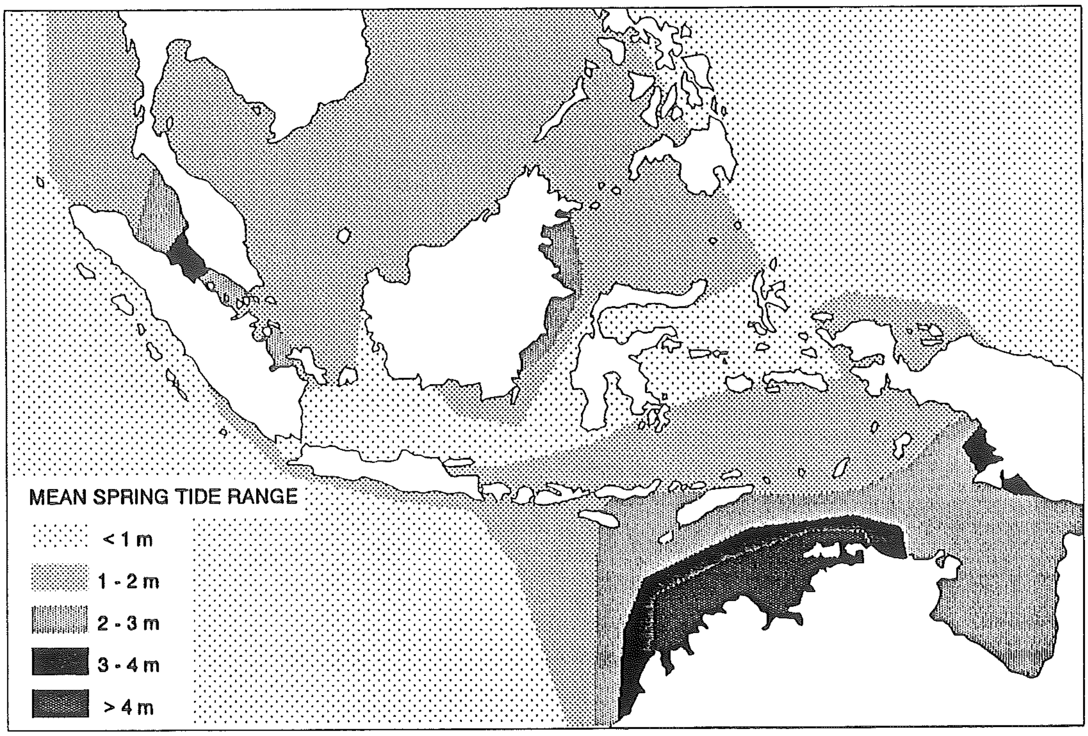


Figure 4.21 Mean tidal range in SE Asia (provisional)

Figure 4.21 shows the tidal range in Indonesia. High tides only occur in the Strait of Malacca and at the South coast of Irian Jaya. An even more intrigueing picture is given by the character of the tides (f from equation (4.30)), see Figure 4.22A. From the purely semi-diurnal tides in Malacca Strait to the purely diurnal tide near Palembang is no more than 500 km, and from purely diurnal North of Surabaya to mainly semi-diurnal in the Madura Strait is about 25 km. This is all rather unique in the world and has to be explained from the behaviour of the tidal waves in the geometry of the archipelago. This will be done here only for the tides on the Sunda shelf between Sumatera, Kalimantan and Java.

"Tidal statistics"

The following definitions are important when using tide tables or sea charts:

H.A.T. (Highest Astronomical Tide). L.A.T. (Lowest Astronomical Tide). The highest and lowest levels, respectively, which can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions. These levels will not be reached every year. H.A.T. and L.A.T. are not the extreme levels which can be reached, as storm surges may cause considerably higher and lower levels to occur.

M.H.W.S. (Mean High Water Springs). M.L.W.S. (Mean Low Water Springs). The height of mean high water springs is the average, throughout a year when the average maximum declination of the moon is 23½°, of the heights of two successive high waters during those periods of 24 hours. (approximately once in a fortnight) when the range of the tide is greatest. The height of mean low water springs is the average height obtained by the two successive low waters during the same periods.
M.H.W.N. (Mean High Water Neaps). M.L.W.N. (Mean Low Water Neaps). The height of mean high water neaps is the average, throughout a year as defined above, of the heights of two successive high waters during those periods (approximately once in a fortnight) when the range of the tide is least.
The height of mean low water neaps is the average height obtained from the two successive low waters during the same periods.
Note: The average value of M.H.W.S., etc. varies from year to year in a cycle of approximately 18.6 years.

M.S.L. (Mean Sea Level). Mean sea level is the average level of the sea surface over a long period, preferably 18.6 years, or the average level which would exist in the absence of tide.

M.H.H.W. (Mean Higher High Water). The height of mean higher high water is the mean of the higher of the two daily high waters over a long period of time. When only one high water occurs on a day, this is taken as the higher high water.

M.L.H.W. (Mean Lower High Water). The height of mean lower high water is the mean of the lower of the two daily high waters over a long period of time. When only one high water occurs on some days "Δ" is printed in the M.L.H.W. column indicating that the tide is usually diurnal.

M.H.L.W. (Mean Higher Low Water). The height of mean higher low water is the mean of the higher of the two daily low waters over a long period of time. When only one low water occurs on some days "Δ" is printed in the M.H.L.W. column indicating that the tide is usually diurnal.

M.L.L.W. (Mean Lower Low Water). The height of mean lower low water is the mean of the lower of the two daily low waters over a long period of time. When only one water occurs on a day this is taken as the lower low water.
Note: The average value of M.H.H.W., etc., varies from year to year in a cycle of approximately 18.6 years.

4.3.3 *Tidal wave mechanics*

The tidal ranges due to the direct attracting forces of the celestial bodies are low. In the open ocean they do not exceed a few dm. On the continental shelves, especially in bays and estuaries tidal ranges can sometimes reach values of more than 10 m. This cannot be explained from the astronomical features as discussed in section 4.3.1, but comes from the hydrodynamical behaviour as discussed for waves in section 4.2.4.

A tidal wave behaves as a very long, shallow water wave even in the ocean. Using the relations of page 22, we can see that the wave length ($L = c.T = \sqrt{gh}.T$) of a semi-diurnal wave in the ocean (5000 m deep) is about $\sqrt{(10*5000)}*45000 \approx 30{,}000$ km! The behaviour of these waves can, in first approximation, be described with the same realtions as for wind waves. When currents and flow patterns are important, more advanced numerical models will have to be applied, taking among other things friction into consideration.

The first factor that increases the tidal wave height is **shoaling**, while also **narrowing**, e.g. in a bay, will lead to higher tides. Neglecting friction, reworking of equation (4.19) gives, with $E = 1/2\rho g a^2$, $n = 1$ and $c = \sqrt{gh}$, for tidal waves:

$$\frac{a}{a_0} = K_{SH} K_B = \left[\frac{h_0}{h}\right]^{\frac{1}{4}} \left[\frac{b_0}{b}\right]^{\frac{1}{2}} \qquad \textbf{(4.31)}$$

From this can be seen that the height of a tidal wave that comes from the ocean with a depth of 5000m on a continental shelf of 50 m deep, will become three times as high ($100^{1/4}$). In a bay that narrows a factor 4, the tidal wave will double again ($\sqrt{4}$).

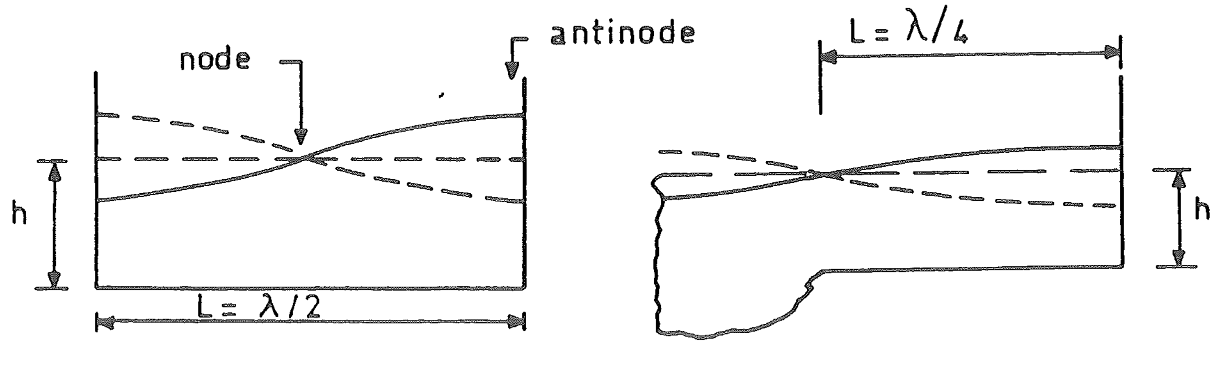


Figure 4.19 Resonance modes for closed and open basins

This shoaling and narrowing will only take place when the changes in depth and width are gradual. With abrupt changes, also **reflection** will occur, giving again a standing wave. Given a certain geometry, now also **resonance** can occur. This is the phenomenon when an exciting force (in this case the tidal wave) has about the same period as the natural period, or one of its harmonics, of a sea or bay. These periods are related to the travel

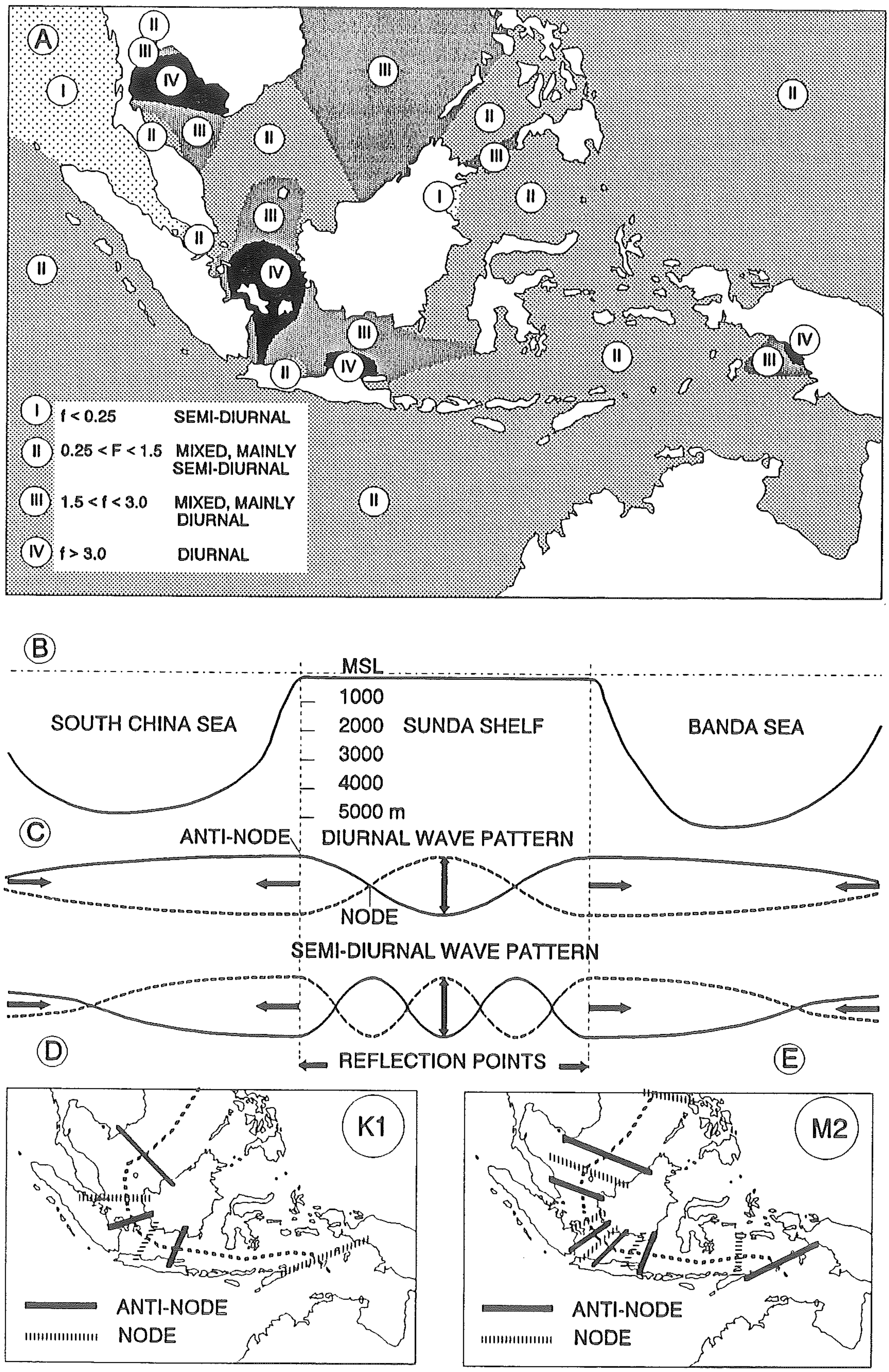


Figure 4.22 Tidal character in the Indonesian archipelago (from Schiereck,1978)

The tidal waves enter from the Pacific Ocean into the South China Sea between Taiwan and the Philippines and from the Indian Ocean between Timor and Irian Jaya (all other "entrances" are considered less important). Consider now the line Pacific - South China Sea - Sunda Shelf - Banda Sea - Indian Ocean as a straight channel. Figure 4.22B shows the depths along this channel. South China Sea and Banda Sea reach maximum depths of 5000 m with an average of about half that value, rising steeply to the Sunda shelf with typical depths of 50 m. For a (very long) tidal wave this transition acts like a reflecting coastline. The lengths of each of the three basins is about 2000 km. The length of the deep basins is less than a quarter of the wave length ($\lambda = \sqrt{(g*h)}*T \approx \sqrt{25000*85000} \approx 7000$) km for diurnal tides, while the Sunda shelf is approximately equal to the wave length ($\lambda = c*T = \sqrt{(g*h)}*T \approx \sqrt{500*85000} \approx 1900$ km. For semi-diurnal tides, the wave lengths are about half these values. This leads to the wave pattern on the shelf of Figure 4.22C. Of course, Figure 4.22C is a highly simplified and idealized picture, but indeed, this pattern can be recognised in the amplitudes of the two main tidal constituents (K1 and M2), indicated in the Figure 4.22D. There are no real nodes where the amplitude is zero, but as nodes and anti-nodes are taken those places where the amplitudes are relatively low or high compared with their surrounding. The reflection points at the shelf edges are visible as anti-nodes, while K1 has two nodes and one anti-node on the shelf (Figure 4.22D) and M2 has four nodes and three anti-nodes on the shelf (Figure 4.22E). The areas with purely diurnal tides (type IV) coincide completely with the K1 anti-nodes near Banka and Surabaya, where M2 has nodes, while the mainly semi-diurnal area near Cirebon in West Java, coincides with an M2 anti-node, where K1 has a node.

4.4 Other water motions

4.4.1 Monsoon currents

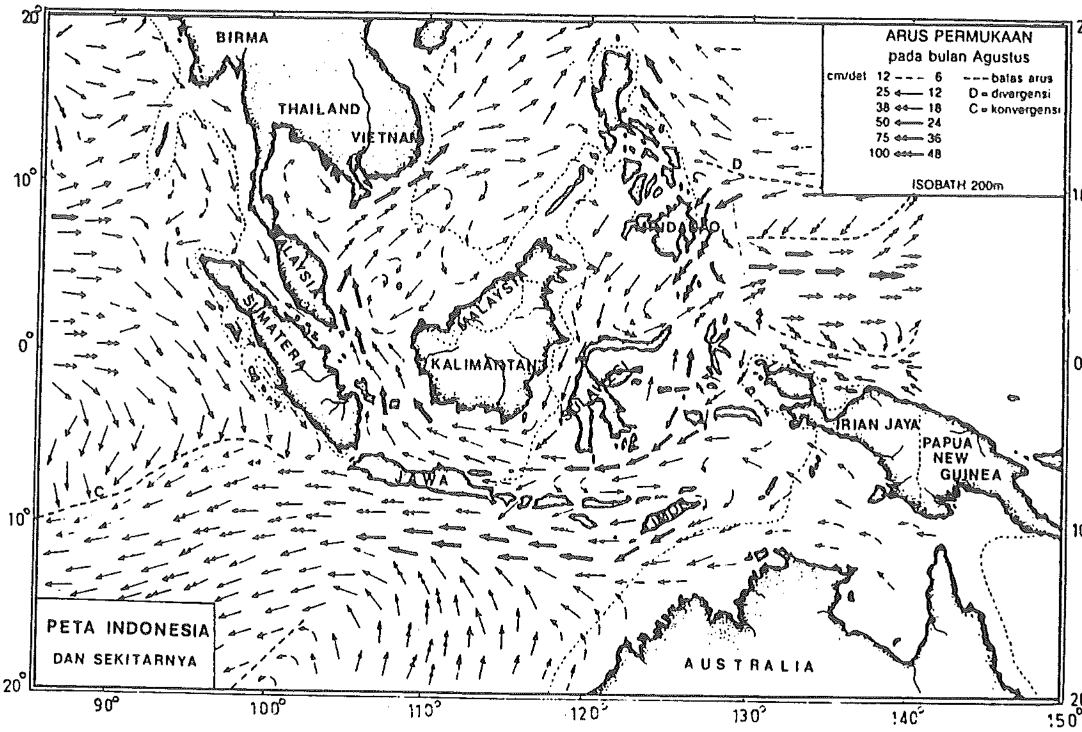


Fig. 1. Surface currents in August (source: PPPO, Jakarta).

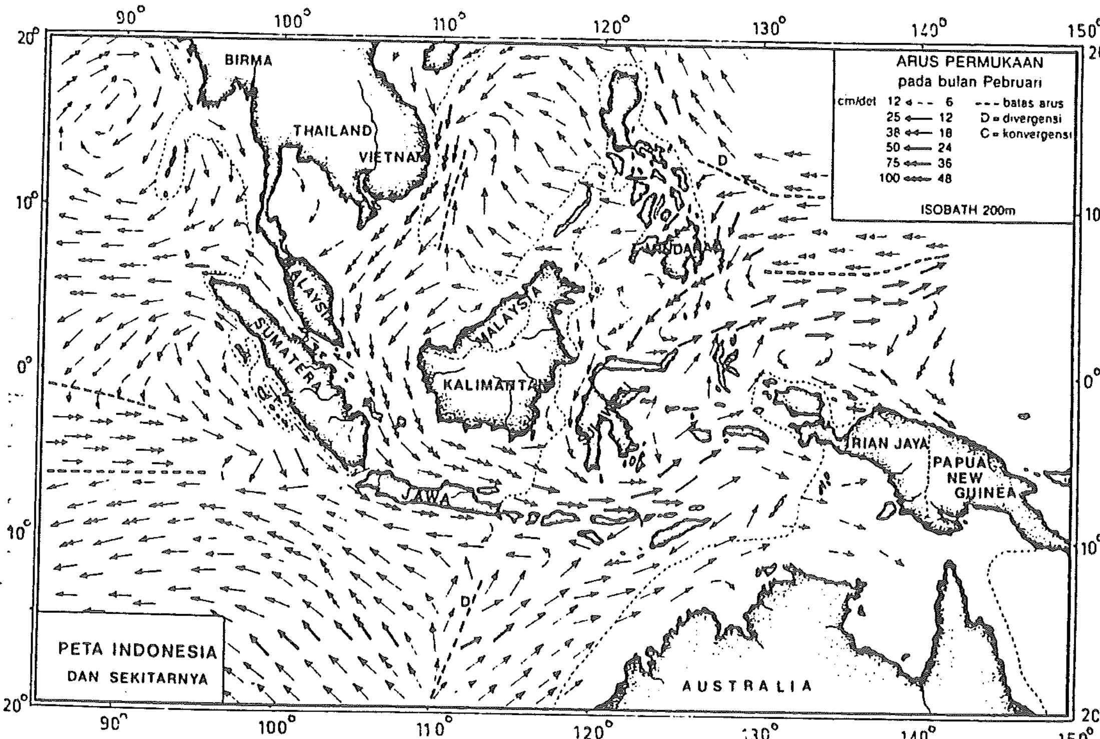


Fig. 2. Surface currents in February (source: PPPO, Jakarta).

Figure 4.23 Monsoon currents (from Wyrtki,1961)

The monsoon currents are reasonably predictable since the monsoon winds show a regular pattern. Typical velocities, e.g in the Java Sea, are 0.25-0.50 m/s. The wet season, when sediment delivery from the rivers to the coast is maximum, is mainly in the Northern winter, hence coupled to an Eastern going current in the Java Sea. The strength, however, can vary from week to week and even from year to year.

4.4.2 Tsunami

Tsunami are seismically induced gravity waves, characterized by wave lengths that are in the order of minutes, rather than seconds. The often originate from earthquakes below the ocean, where water depths can be more than 1000 m, and may travel long distances without reaching any noticeable wave height.

However, when approaching coastlines the height may increase considerably. Due to the large wave length, increases that result from shoaling and refraction coming inshore from quite large depths can be calculated using shallow-water wave theory. Wave reflection from relatively deep slopes of continental shelves may also be important.

Although theoretical work is available, the only published observations for height and period of actual tsunami, are those found in Japanese sources, and concern tsunami observed at coasts within a range of about 750 km from the epicentre of the earthquakes (see Figure 4.24).

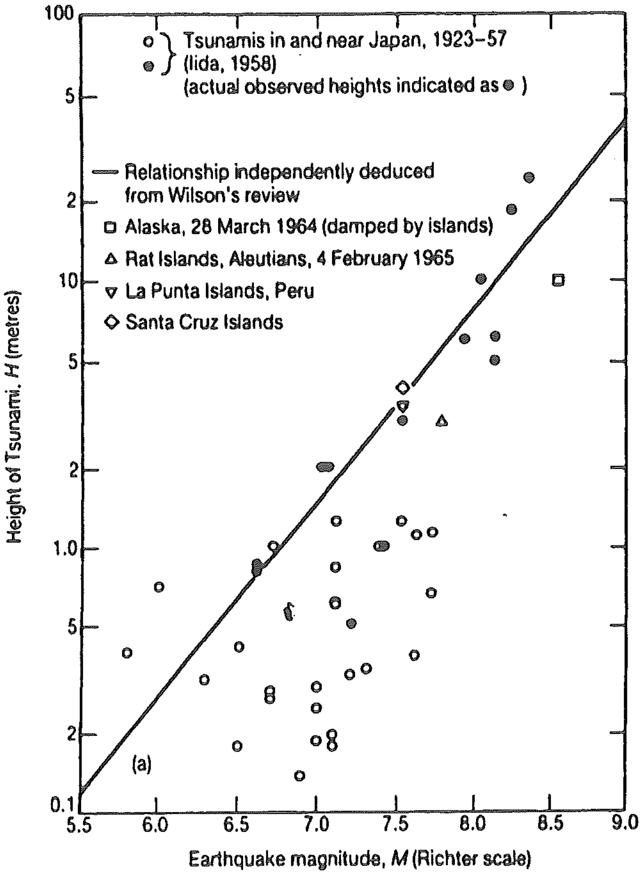


Figure 4.24 Height of a tsunami as a function of earthquake magnitude (from CUR,1995)

After a tsunami has arrived at the coast the inundation height is roughly equal to the wave height H (see Camfiled,1980). The velocity in such a wave, propagating over a "dry" bottom, can then be estimated with $u \approx 2\sqrt{(gH)}$, which leads for a tsunami wave height of 5 m to velocities of about 14 m/s. The forces exerted by such a wave are tremendous and since tsunami are as unpredictable as earthquakes, use of a "no-damage" criterion may not lead to an economically optimal solution. For specifically tsunami-susceptible areas, a special risk-analysis should be performed, in order to determine whether it is wise to take special measures, e.g. to avoid building close to the shoreline.

4.5 Sediment transport

4.5.1 General

Critical velocity

Sediment transport only occurs when the velocity exceeds some critical value, which, for non-cohesive sediments, depends mainly on the grain diameter and density. For cohesive sediments, see section 4.5.4. Probably the most well known relation for uniform flow is the one by Shields from 1936. Shields gives a relation between a dimensionless shear-stress and the so-called particle Reynolds-number:

$$\psi = \frac{\tau_c}{(\rho_s - \rho_w) g d} = \frac{u_{*c}^2}{\Delta g d} = f(\Re e_*) = f\left(\frac{u_{*c} d}{\upsilon}\right) \tag{4.33}$$

Shields chose the shear stress as the active force. Note that the grain diameter and the shear velocity appear on both sides of the formula. For high $\Re e_*$-numbers, Ψ becomes constant with a value of about 0.055, see Figure 4.25a.

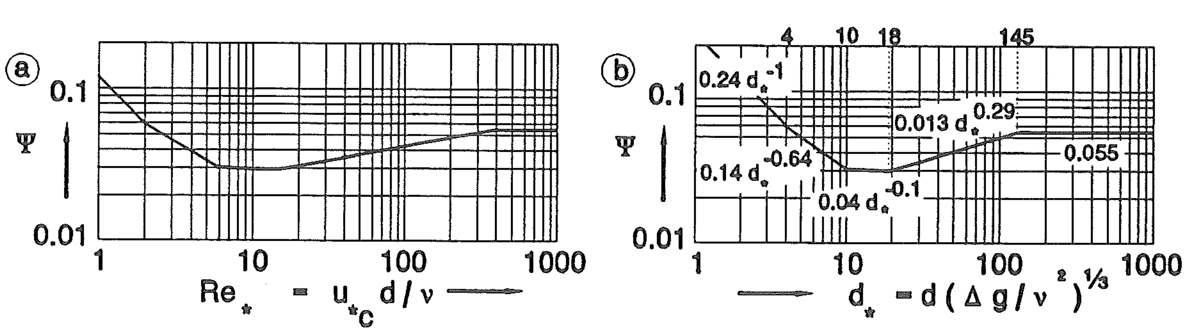


Figure 4.25 Critical shear stress according to Shields - van Rijn

Figure 4.25b gives the same stability relation, but now the particle $\Re e$-number is replaced by a dimensionless particle diameter as used by vanRijn in his presentation of the Shields relation, which has the advantage that it makes iteration on u_{*c} unnecessary. The elimination of u_{*c} from the particle $\Re e$-number is possible because for every grain diameter there is a single value of u_{*c} coupled to it. Moreover, vanRijn gives a numerical presentation of the graph, also indicated in Figure 4.25b. In chapter 9 more about the critical velocity is presented, focussing on the stability of (coarse) protection material.

Fall velocity

Another parameter that is often used in sediment transport is the fall velocity. For small spherical particles the fall velocity is given by Stokes law:

$$w = \frac{\Delta\, g\, D^2}{18\, \upsilon} \tag{4.34}$$

in which:

Δ = relative density = $(\rho_S - \rho_W)/\rho_W$ [-]
D = particle diameter [m]
υ = kinematic viscosity [m^2/s]

Note: The viscosity decreases with increasing temperature. In tropical waters υ will be about half of the value in temperate waters, hence the fall velocity will be twice as high !

4.5.2 *Cross-shore sand transport*

There exists a relation between the equilibrium profile and the "average" morphological "dominant" waves. One of the first descriptions of a coastal profile shape is given by BRUUN [1954]. This expression has the general form:

$$h = Ax^m \tag{4.35}$$

where h is the still water depth, x is the horizontal distance from the shoreline and A and m are coefficients, based on a best fit. A mean value of m = 2/3 will fit most of the data. The coefficient A is not dimensionless, it has the dimension of $m^{1/3}$. A value of A = 0.1 $m^{1/3}$ is a reasonable first approximation.

Instead of this approximation, one may also use the so-called "Vellinga-profile". This is given by:

$$\left(\frac{7.6}{H_o}\right) h = 0.47 \left[\left(\frac{7.6}{H_o}\right)^{1.28} \left(\frac{w}{0.0268}\right)^{0.56} x + 18\right]^{0.5} - 2.00 \tag{4.36}$$

where:

H_o is the deep water wave height,
w the settling velocity of the sediment (of the D50),
h is the waterdepth and
x is the distance from the water-line.

This profile has been developed for dune-erosion processes, so in fact it is the shape of a coastal profile attacked by only one single wave. The profile of Vellinga is tested on many large-scale model tests and prototype situations, and is therefore very reliable.

For Dadap, near the Cimanuk estuary at the northern coast of Java, detailed profile data and sediment data are known. The grain-size accordidng to the survey is approximately 130 μm. With this information one can fit the measured profile with the computed Vellinga profile, using the wave-height and de period as fitting parameters.

A value of H_s = 0.35 m and T = 4 sec (using D_{50} = 130 μm and W = 0.013 m/s) gives a good correlation for the profile until a depth of 12 m. At 12 m is clearly the end of the wave-induced profile shape (correlation coefficient 99.7 %).

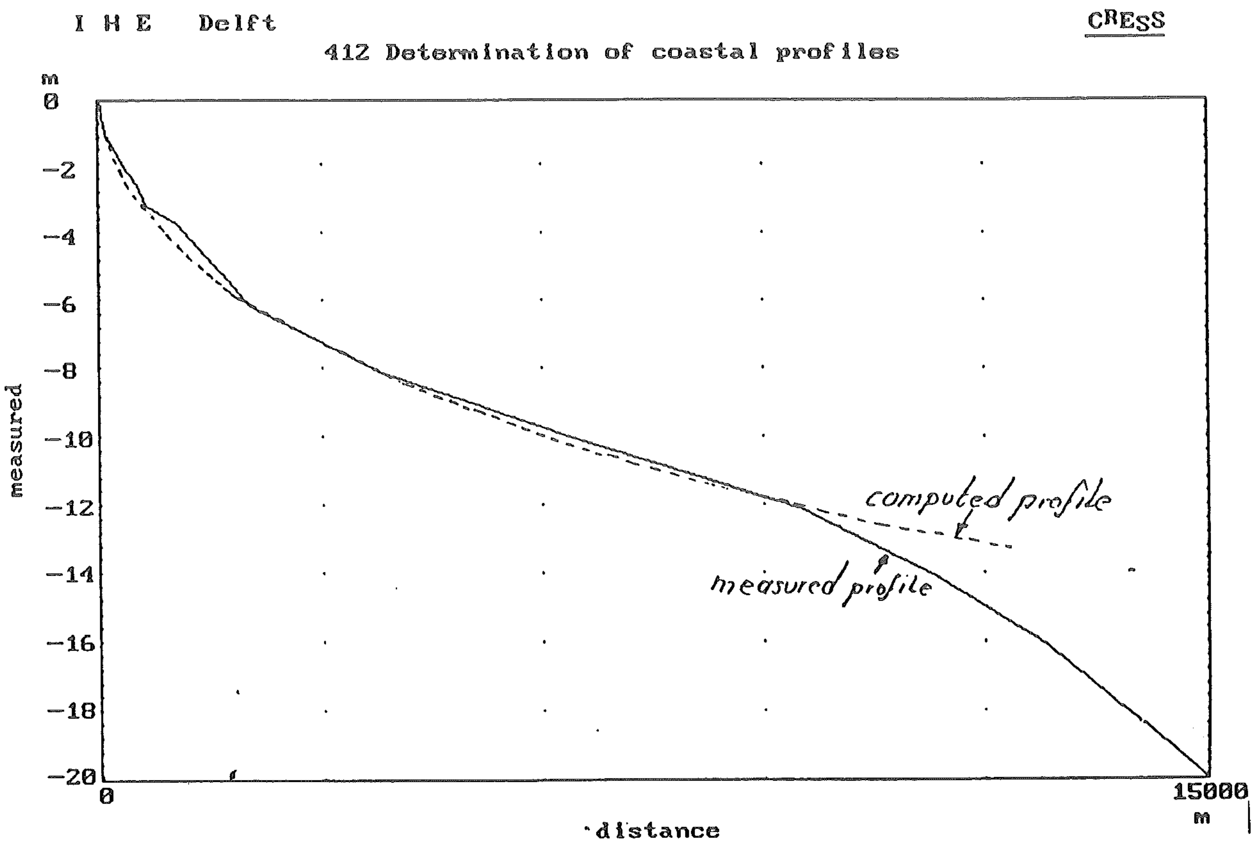


Figure 4.26 Comparison of measured and computed profile at Dadap

From this exercise one may conclude that the morphologically "dominant" wave, i.e. the wave that shapes the coast, is rather small. For Dadap one should use H_s = 0.35 m and T = 4 sec in morphological computations.

4.5.3 Longshore sand transport

Due to the action of waves and currents sediment is transported along the coast. It is not possible to use for longshore transport the formulas normally applied for sediment transport in current only (for example in rivers), because there is a big influence of the wave action on the transport capacity. In fact in sediment transport one may distinguish 2 steps:

- the sandgrains have to be brought into suspension
- the suspended grains are transported by the current.

Because in the breaker zone by definition there is considerable wave action near the bottom, the orbital movement of the waves stirs up a lot of sediment. So, in a condition with currents + waves the concentration will be much higher than in case of only a current.

Based on the above considerations one may calculate the sediment transport in a given point in the breaker zone as follow:

1. Determine in this point the wave-height
2. Determine the orbital velocity near the bottom
3. Calculate the longshore velocity using the radiation stress method

4. Calculate the sediment transport with an applicable "currents + wave" formula.

If one calculates in this way the sediment transport in all points of a coastal profile and integrates the values over the breaker zone one gets the total transport along the coast. This is quite a lot of work, therefore formulas have been developed to assess the total load directly. Analysing the above procedure leads to the conclusion that the major factors in the computation are the longshore velocity, the wave height and the period. As was indicated earlier, the longshore velocity also mainly depends on wave height and period.

The Cerc-formula

Combining the longshore current formula of Longuet-Higgins with some calibration constants leads to the following formula:

$$S = AH_b^2\ n_b c_b \sin\phi_b \cos\phi_b \tag{4.37}$$

in which:

- S - Sediment transport (m^3/s)
- H_b - breaker height
- c_b - wave-celerity at the breaker line
- ϕ_b - breaker angle
- n_b - wave number at breakerline (is approx. 1)

Note that the CERC-formula contains the same $\cos\phi.\sin\phi$ as the longshore current.

In case of parallel depth contours, one may also write:

$$S = AH_0^2\ n_0 c_0 \sin\phi_b \cos\phi_0 \tag{4.38}$$

in which:

- H_0 - significant wave height in deep water (m)
- c_0 - wave-celerity in deep water (m/s)
- ϕ_0 - wave-angle in deep water (-)
- n_0 - wave number at deep water (is approx. 0.5)

This is the so-called CERC-formula, developed in the Coastal Engineering Research Centre of the USArmy Corps of Engineers. The parameter A is determined by using a lot of prototype -data from many types of coasts. The original value for A is 0.028 (this gives S in m^3/s) or 880,000 (this gives S in m^3/yr). In the newer versions of the Shore Protection Manual a value of 0.050 is recommended, in the Netherlands often a value of 0.040 is used. But on should realise that the standard deviation in this value is considerable, a reasonable range is 0.02-0.08.

Because the data used for the determination of A mainly come from the ocean coasts of the USA, application in other places is doubtful. In any case one should apply this formula only

- at beaches with normal beach sand (around 200 μm)
- at beaches facing an ocean coast (H $\approx \sqrt{T}$)

- at beaches without tidal currents
- at beaches with a normal beach slope (m $\approx$ 1:100)
- at straight beaches

The Queens-formula

Recent research in the framework of the *Canadian coastal sediment study* (Kamphuis,1986) has lead to a variant of the Cerc-formula. The work for this new development is mainly done by Kamphuis from Queens University, therefore this formula is called the Queens formula. A dimensional analysis of bulk sediment rate in the turbulent breaking zone leads to the following general formula:

$$S = 1.2 * 10^{-3} \frac{\rho_w}{(1-p)\rho_s} \frac{H_b^{\,4}}{T D_{50}} \xi \cos\phi_b \sin\phi_b \qquad (4.39)$$

in which:

S	=	sediment transport [m^3/s]
p	=	porosity (-)
ρ_s	=	density of the sediment [kg/m^3]
ρ	=	density of sea water [kg/m^3]
H_b	=	significant wave height at breaker point [m]
T	=	peak period of the spectrum [s]
ξ	=	Iribarren number at breakerpoint [-] (equation (4.20)
D_{50}	=	Median grain size [m]
ϕ_b	=	Wave angle at breakerpoint
α	=	beach slope
L_0	=	deep water wave length

The coefficient in the formula was determined by experiments, see Kamphuis,1986.

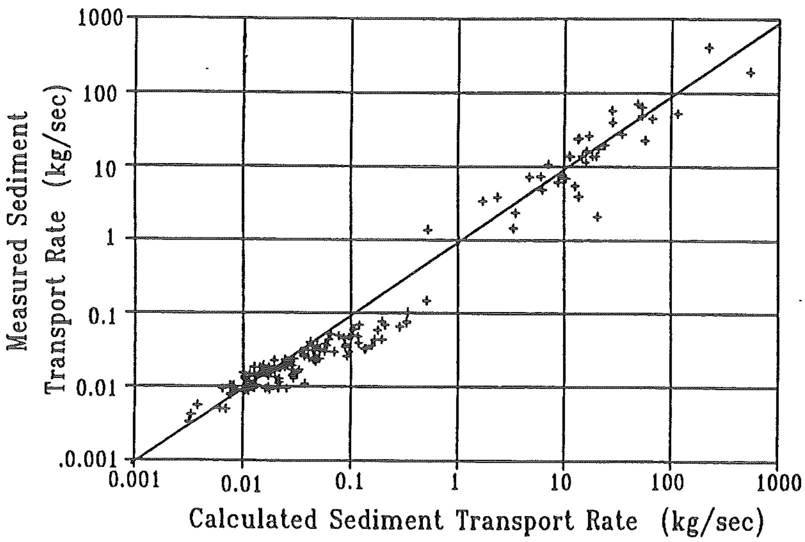


Figure 4.27 Results of the Queen formula (from Kamphuis,1986)

The field data and their fit to the equation are shown in Figure 4.27. The result is quite good, and somewhat better than the result obtained by application of the Cerc-formula. However, there is still a wide spread in the results. As can be seen in the figure, the fit to lab-data is not optimal. Therefore Kamphuis proposed in 1990 an adapted formula, in which he splits up the ξ in two components, while also the exponents in the formula get different values.

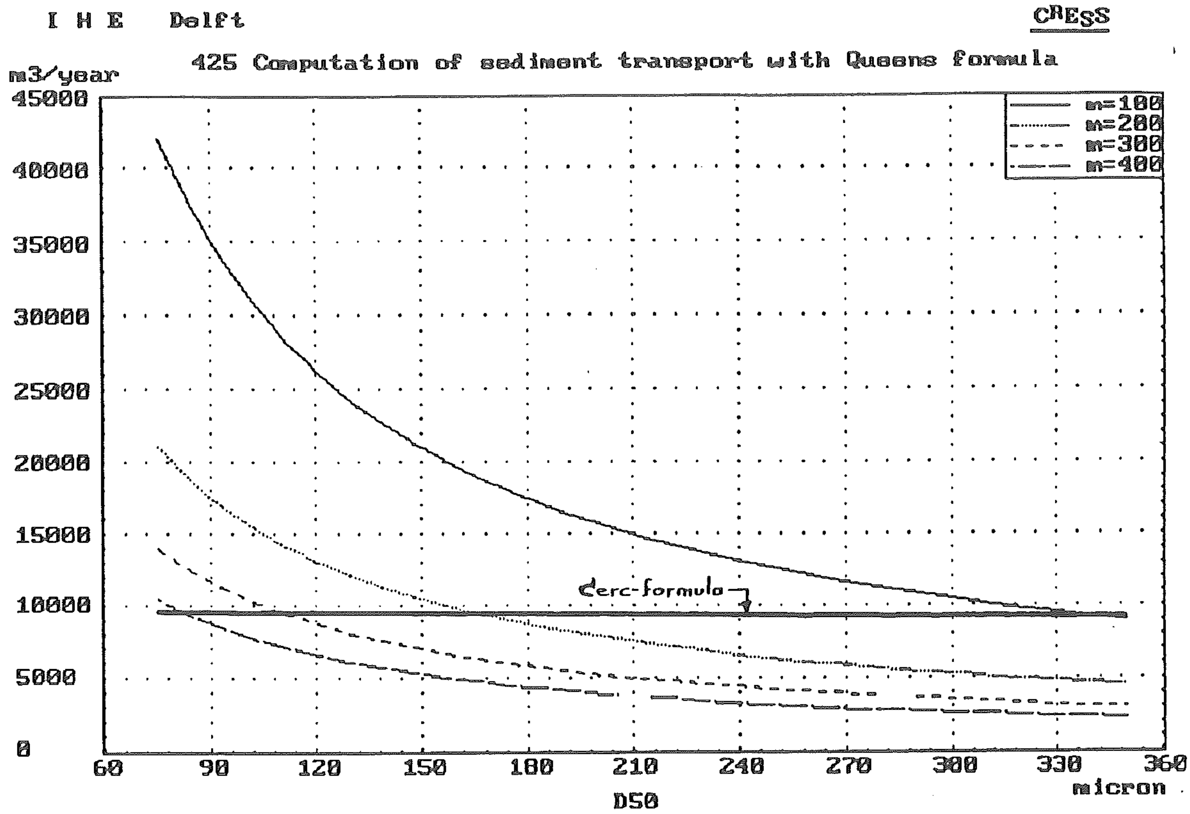


Figure 4.28 Comparison of the CERC and Queens formula for H=2m, T=5s and ϕ=30°

A comparison between the Cerc formula and the Queens formula is given in Figure 4.28. Although the influence of the beach angle and the grain-size looks very big, one should realize that the accuracy of both formulas is quite low and the real transport can easily be five times as much or as less as computed. See also Figure 4.27, which is on log-log scale.

The Queens-formula is much more applicable than the CERC-formula. It is therefore recommended not to apply the CERC formula any more, but in case a total transport formula is needed to use the Queen's formula.
However, the Queens-formula is also <u>only</u> valid in the following conditions.

- no tidal current is present
- straight beach, no groyne-fields or offshore breakwater present
- plane beach, no complicated breaker-bar systems

In the case of a tidal curent that cannot be neglected, an approach as developed by Bijker,1968 can be used. In his approach one first calculates the wave-height at every point in the breaker-zone, followed by a calculation of the longshore velocity in that point. This can be done with the methods described in the previous section.

The next step in the Bijker-approach is to determine the local sediment transport (in m^3/s per running meter) with an appropriate sediment transport formula. Bijker himself developed a formula to compute the bed-load transport as a function of velocity and wave-height. When the bed-load is known, the suspended load can be computed with the method of Einstein. The disadvantage of the Einstein-method is that it contains a numerical integration which results sometimes in numerical inaccuracy. Bhattacharya developed an alternative numerical solution.

In principle Bijker developed his formula by starting with a normal bed-load transport formula for transport in a uniform current (like in rivers). In wave conditions, however, the waves stir up a lot of extra sediment. The real momentaneous velocity at the bottom is the vector-sum of the longshore velocity and the orbital velocity. After time-averaging the orbital component disappears (because that component is symmetrical). The bottom shear stress cannot be averaged out, because the shear stress is a higher power of the velocity. So, waves cause a higher shear stress, see also chapter 9. Because of this, Bijker replaced in the normal sediment transport formula the shear stress by a combined current-wave shear stress. This results finally in a formula including a stirring factor.

******** In the final version the Bijker formula will be presented in detail ********

As an example for the Dadap coast a computation is made /with the Cerc, the Queens and the Bijker-formula for waves of 0.35 m (based on the results from the cross-shore profile at the Dadap coast). These method are applicable for sand, not for mud. At the Dadap coast one finds fine sand ($d_{50} \approx 130\ \mu m$). So, in principle one may apply the methods, however, one should realize that there is a lot of additional suspended load in the water. The results of these computations are:

H (m)	T (sec)	Cerc	Queens[1]	Queens[2]		Bijker
0.35	4	41	20	4	1:260	2

1 Using the Queens formula with the slope at the waterline (1:60)
2 Using the Queens formula with the slope of the nearshore zone
Other input: $\phi_o = 20°$
$\gamma = 0.65$
$D_{50} = 130\ \mu m$
$r = 0.02$ m

Sediment computations for Dadap; transports given in 1000 m^3/year.

The CERC-formula overestimates the real transport in this area considerably.

4.5.4 *Mud transport*

******** For the time being the following pages on mud transport are included. They are a reprint of the IHE lecture notes of mr L. van Rijn. ********

4.6 *Data management*

******** To be formulated later on ********

4.7 *References*

11. TRANSPORT OF COHESIVE MATERIALS

11.1 Introduction

Sediment mixtures with a fraction of clay particles (d < 4 μm, Am. Geoph. Union Scale) larger than about 10% have cohesive properties because electro-statical forces comparable to or higher than the gravity forces are acting between the particles. Consequently, the sediment particles do not behave as individual particles but tend to stick together forming aggregates known as flocs whose size and settling velocity are much larger than those of the individual particles.

Most clay minerals have a layered (sheet-like) structure. The most important types of clay minerals are:

- kaolinite (two-layer structure)
- montmorillonite (three-layer structure)
- illite (three-layer structure)
- chlorite (four-layer structure)

Mud is herein defined as a fluid-sediment mixture consisting of (salt)water, sands, silts, clays and organic materials. Proper classification requires the determination (using standarized methods) of a minimum set of parameters, as follows:

sampling location : coordinates, water depth, maximum current velocity, maximum wave height,
bed structure : dry sediment density with depth, critical shear stress for erosion of surface,
native fluid : temperature, salinity, pH, chemical composition,
sediment : clay, silt, sand percentage, organic content, mineralogy, grain sizes, fall velocities, specific area, cation exchange capacity (CEC),
pore fluid : temperature, salinity, pH, sodium absorption ratio (SAR), cation/anion composition.

The composition of samples of natural muds found in The Netherlands and that of kaolinite material are presented in the following Table 11.1.

Mud type	Lutum < 2 μm (%)	Clay < 16 μm (%)	Sand > 63 μm (%)	Organic material (%)	Stokes diameter of deflocculated material d_{50} (μm)	Fall velocity of deflocculated material $w_{s,50}$ (mm/s)
Kaolinite	40	95	0	0	3	0.008
Hollands Diep 1 (lake)	30	75	9	10	5	0.02
Hollands Diep 2 (lake)	25	60	23	9	10	0.1
Ketelmeer (lake)	28	67	7	12	7	0.05
Biesbosch (lake)	28	65	8	8	8	0.06
Maas (river)	16	42	36	8	25	0.5
Breskens Harbour (estuary)	24	55	27	5	10	0.1
Delfzijl Harbour (estuary)	15	30	60	2	80	5.5
Loswal Noord (sea)	14	23	69	2	100	8.5

Table 11.1 Composition of kaolinite and natural muds

In a natural environment there is a continuous transport cycle of mud material which consists of: erosion, settling, deposition, consolidation, erosion and so on. As a result of the complexity and the lack of fundamental knowledge, the description of the various processes is largely empirical. Most information is based on laboratory experiments. Figures 11.1.1 and 11.1.2 show typical laboratory methods to study mud properties. Laboratory results are, however, often not representative because the biogenic mechanisms and the organic materials are missing, the water depths are small, the deposition and the consolidation history of the bed is different from nature. Therefore, more in-situ research should be carried out.

This chapter summarizes the most important properties and processes:

- viscosity and yield stress
- flocculation
- settling
- deposition
- consolidation
- erosion
- transport

11.2 Cohesion, plasticity, viscosity and yield stress

If a cohesive soil sample with a low water content is submitted to shear stresses (τ) under various normal pressures (σ) to the point of failure, the relationship between τ and σ can be expressed as (Law of Coulomb):

$$\tau = \tau_y + \sigma \tan \phi \qquad (11.2.1)$$

in which:
τ_y = yield stress
ϕ = angle of internal friction

The yield stress is generally interpreted as the "cohesion" of the sample. Thus, a cohesive sediment sample is able to withstand a finite shear stress for $\sigma = 0$ (no deformation). The angle of internal friction represents the mechanical resistance to deformation by friction and interlocking of the individual particles.

Plasticity is the property of cohesive material to undergo substantial permanent deformation without breaking.

Types of basic rheological behaviour of mud are shown in Fig. 11.2.1. Dilute suspensions with concentrations smaller than 10 kg/m³ show a Newtonian behaviour. Deviations from this latter behaviour tend to occur at concentrations larger than 10 kg/m³.
High-concentrations (> 50 kg/m³) suspensions of water, fine sand, silt, clay and organic material usually have a pseudo-plastic or a Bingham plastic shearing behaviour, which means that the relationship between shear-stress (τ) and shear rate (du/dz) is non-linear (Fig. 11.2.1). The slope of the curve expresses the effective dynamic viscosity (η) of the material. The intercept with the shear stress axis is called the yield stress (τ_y) and represents the initial particle interaction force which must be overcome to give a distortion of the material. The yield stress of an ideal Bingham type mud is called the Bingham stress (τ_B). Laboratory observation of static slopes of settled mud beds indicates that a yield stress is a physical reality.

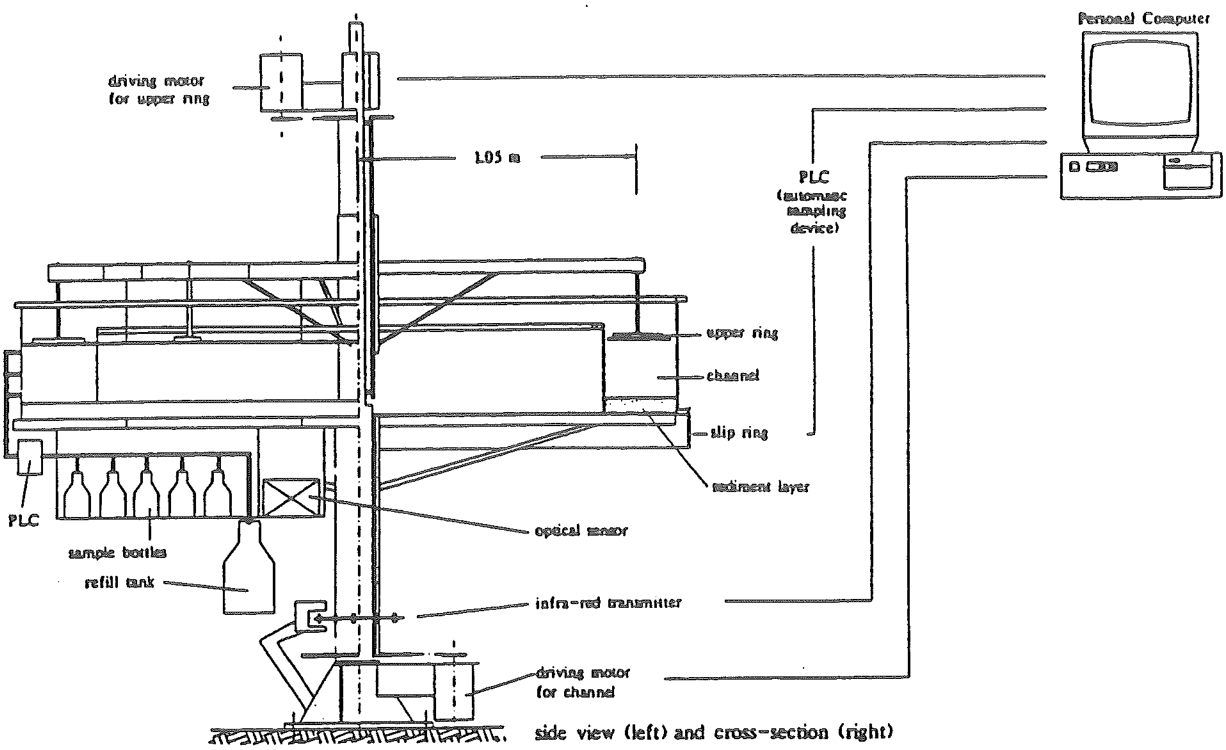


Figure 11.1.1 Circular laboratory flume (carousel)

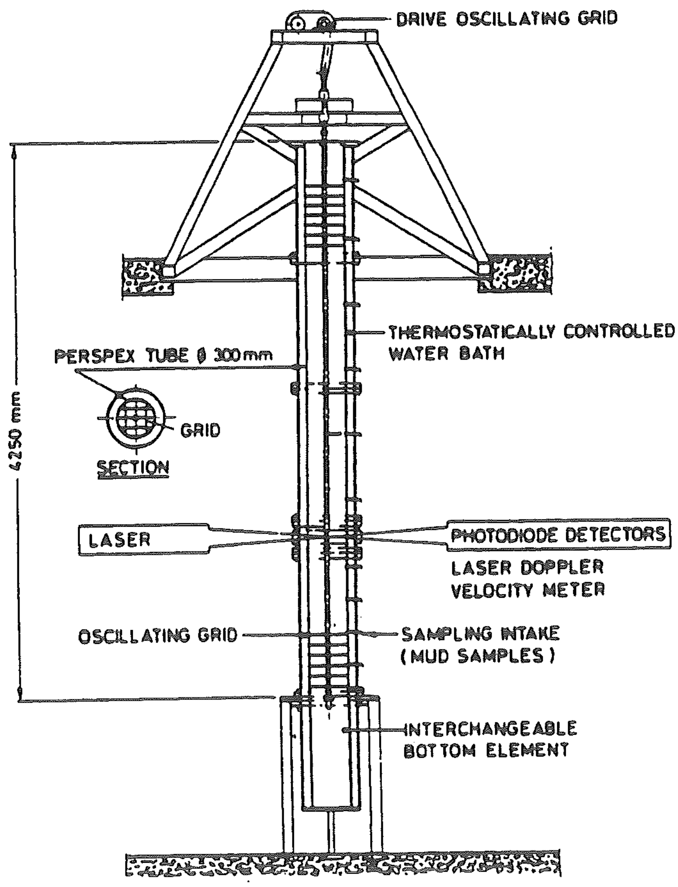


Figure 11.1.2 Laboratory settling tube

General formulations of rheological models are:

$$\begin{aligned}
&\text{Newtonian} &&: \tau = \eta \frac{du}{dz} \\
&\text{Pseudo plastic} &&: \tau = m \left(\frac{du}{dz}\right)^n \\
&\text{Ideal Bingham} &&: \tau = \tau_B + \eta \left(\frac{du}{dz}\right) \\
&\text{Bingham plastic} &&: \tau = \tau_y + m \left(\frac{du}{dz}\right)^n
\end{aligned} \tag{11.2.2}$$

in which:

τ = shear stress
τ_y = yield stress
τ_B = Bingham stress
η = dynamic viscosity coefficient
n = empirical coefficient (< 1)
m = empirical coefficient
du/dz = velocity gradient

Viscosity and yield stress can be measured in a roto-viscometer. This instrument consists of two concentric cylinders in which the sediment material is placed (Fig. 11.2.2). One cylinder is rotated at a constant rate giving a constant shearing rate (du/dz), while the force is measured on the other cylinder giving the shear stress (τ). Figure 11.2.3 shows the results for a natural mud based on the use of a Haake roto-viscometer (Winterwerp et al, 1991). The down-curve intercept with the vertical axis is defined as the yield stress (τ_y). The results show a hysteresis loop for increasing shear rates (up curve) and decreasing shear rates (down-curve). This phenomenon is known as thixotropy (time-dependent changes of the viscosity under constant shear) and is due to shear thinning effects.
Based on experimental data, the yield stress was found to be proportional to the sediment concentration ($\tau_y \approx c^\alpha$ with α = 2 to 6). Many attempts have been made to relate the yield stress to the chemical and physical properties of the fluid-sediment mixture. General relationships, however, do not exist. Krone (1986) measured yield stresses of various natural muds and found values in the range of 0.01 to 0.1 N/m^2 for concentrations in the range of 30 to 60 kg/m^3. Other values are given in Table 11.2, based on the tests (using native fluid) of Winterwerp et al, 1991. Figure 11.2.4 shows the yield stress as a function of concentration of these latter test results and the experimental ranges of others (Migniot, 1968; Owen, 1975 and Krone, 1985).

Comparison of the results for kaolinite in tap and saline water shows a considerable decrease (factor 2 to 3) of the yield stress due to the effect of salinity.
Comparison of the results for the natural muds shows a considerable decrease (factor 2 to 3) of the yield stress when the percentage of fine sand is larger than about 40% (Maas, Delfzijl Harbour, Loswal Noord, see Table 11.1), which seems logic because the sand particles do not take part in the cohesive processes.
The natural muds with the smallest median particle diameter seem to have the highest yield stresses (Hollands Diep 1, Ketelmeer, Biesbosch, see Table 11.2).
The functional relationships observed by Migniot (1968), Owen (1975) and Krone (1986) in their experimental ranges are in good agreement with the results of Winterwerp et al (1991), see Fig. 11.2.4.

Material		sediment concentration (kg/m³)				
		50	100	200	300	500
Kaolinite	(tap)	τ_y = 0.01-0.02 N/m²	0.08 - 0.15	0.5 - 1.0	1.5 - 3.0	5 - 10
Kaolinite	(saline)	τ_y = -	0.02 - 0.05	0.15 - 0.3	0.8 - 1.5	2 - 5
Hollands Diep 1	(lake)	τ_y = 0.03-0.06 N/m²	0.1 - 0.2	0.5 - 1.0	3 - 6	-
Hollands Diep 2	(lake)	τ_y = 0.04-0.08 N/m²	0.08 - 0.15	0.3 - 0.6	1 - 2	-
Ketelmeer	(lake)	τ_y = 0.05-0.10 N/m²	0.1 - 0.2	0.4 - 0.8	2 - 4	20 - 50
Biesbosch	(lake)	τ_y = 0.10-0.15 N/m²	0.10 - 0.15	0.3 - 0.6	1 - 2	20 - 40
Maas	(river)	τ_y = 0.02-0.05 N/m²	0.08 - 0.15	0.15 - 0.3	0.3 - 0.6	4 - 8
Breskens Harbour	(estuary)	τ_y = -	0.05 - 0.10	0.4 - 0.8	2 - 4	20 - 40
Delfzijl Harbour	(estuary)	τ_y = 0.01-0.02 N/m²	0.03 - 0.06	0.15 - 0.3	0.3 - 0.6	2 - 4
Loswal Noord	(sea)	τ_y = 0.02-0.04 N/m²	0.06 - 0.12	0.15 - 0.3	0.4 - 0.8	2 - 4

Table 11.2 Yield stresses (N/m²)

Relative dynamic viscosity coefficients (η_m/η, with η_m = viscosity of mixture, η = viscosity of clear fluid) as a function of concentration for natural muds in The Netherlands are shown in Fig. 11.2.5. The viscosity of a fluid-sediment mixture begins to deviate significantly from that of clear water for concentrations larger than about 50 kg/m³. Mud samples with a high percentage of fine sand have a lower relative viscosity (Loswal Noord, Delfzijl Harbour, Maas river). The relative viscosity of kaolinite in saline water is relatively low compared with that of natural muds (low sand percentage).
Einstein (1906) considered the effect of elastic, spherical particle movements on the viscosity of dilute suspensions, resulting in an increase of the effective viscosity, see Chapter 3.
Based on experiments with large volume concentrations, Bagnold (1954) also found an increase of the effective viscosity, see Chapter 3.
Both equations yield values which are considerably too small compared with those of natural muds (Fig. 11.2.5). More realistic values can be obtained by taking the volume concentrations of the particle aggregates or flocs (which are however unknown).

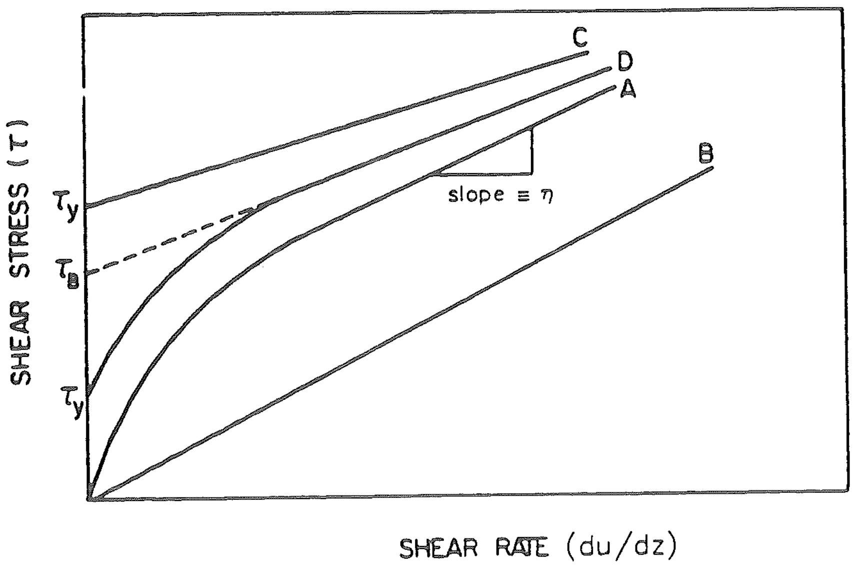


Figure 11.2.1 Rheological behaviour: A. Pseudoplastic, B. Newtonian, C. Ideal Bingham, D. Bingham plastic

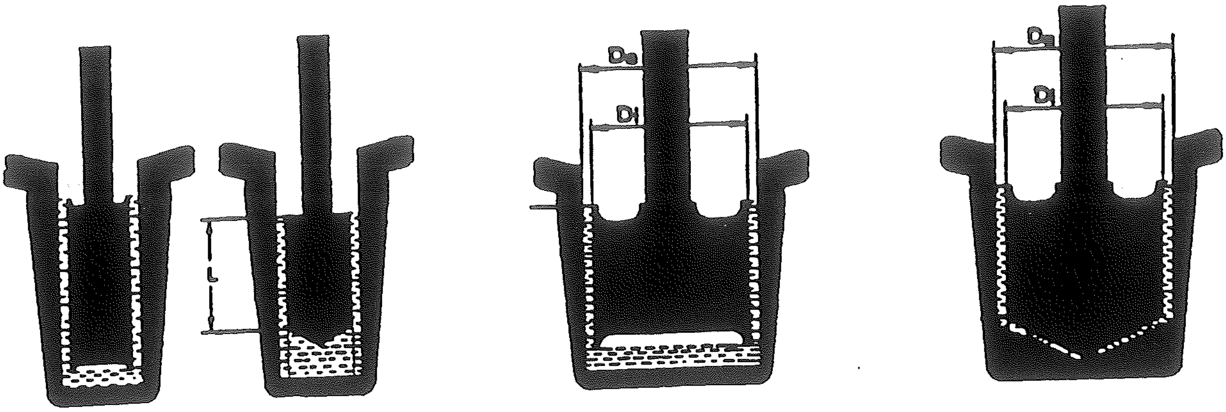


Figure 11.2.2 Examples of roto-viscometers

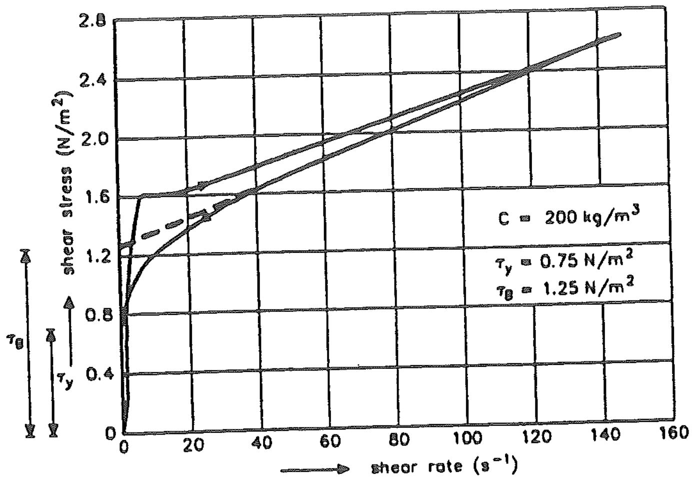


Figure 11.2.3 Bingham plastic behaviour of natural mud (Hollands Diep 1) at a concentration of 200 kg/m³

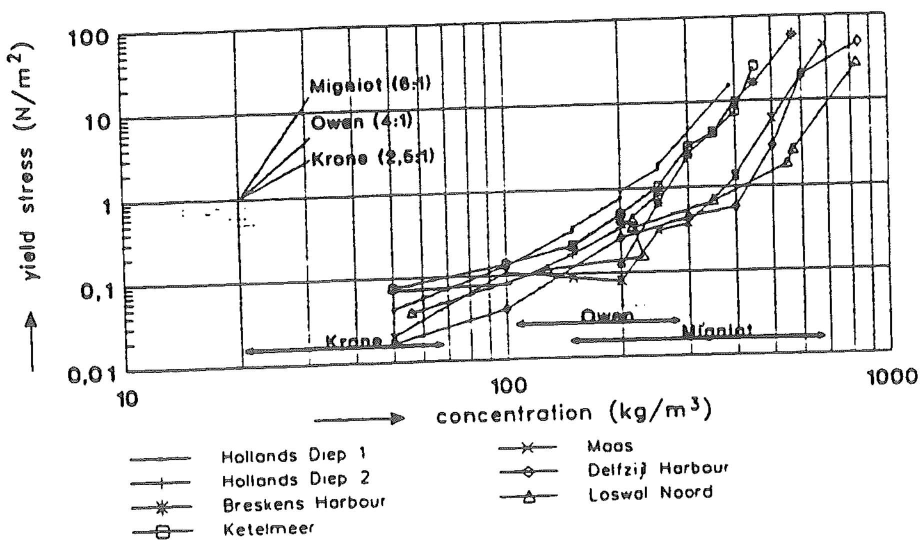


Figure 11.2.4 Yield stress as a function of concentration for natural muds in The Netherlands (Winterwerp et al, 1991)

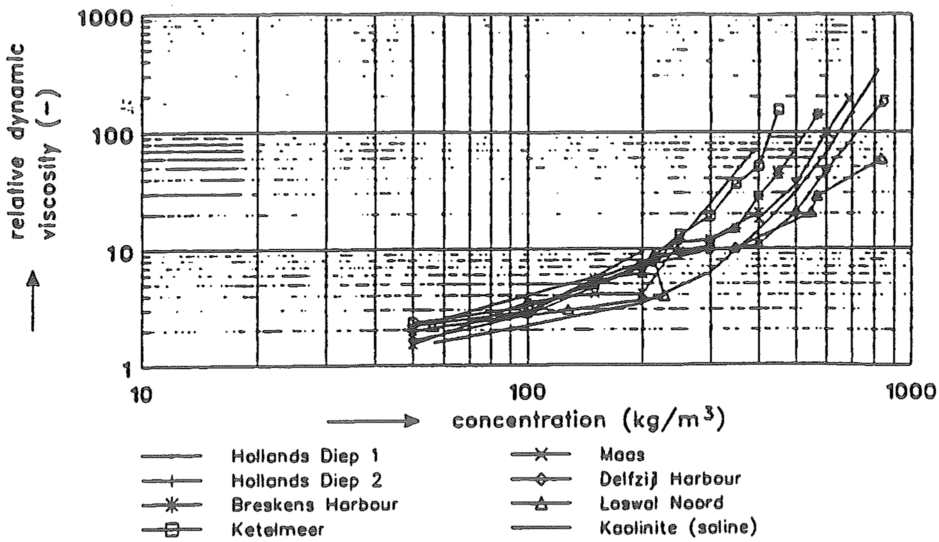


Figure 11.2.5 Dynamic viscosity coefficient as a function of concentration for natural muds in The Netherlands (Winterwerp et al, 1991)

11.3 Flocculation

Most of the individual clay particles have a negative charge. The mutual forces experienced by two or more clay particles in close proximity are the result of the relative strengths of the attractive and repulsive forces. The attractive forces Van der Waals (V_A) are due to the interaction of the electrical fields formed by the dipoles in the individual molecules (Fig. 11.3.1). The repulsive forces (V_R) are due to ion clouds of similar charge repelling each other. Positive ions present in the fluid form a cloud of ions around the negatively charged clay particles (double layer theory). The result can be either attraction or repulsion depending on the relative strengths (depending on number of positive ions) and the distance between the particles.

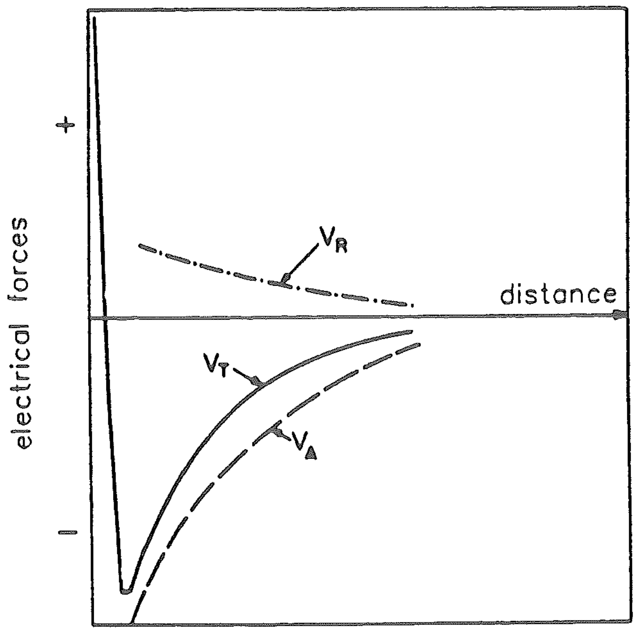


Figure 11.3.1 Electrical forces as a function of distance between particles

In fresh water suspensions (few positive ions) the repulsive forces between the negatively charged particles dominate and the particles will repel each other. In saline water the attractive forces dominate due to the (abundant) presence of positive sodium-ions forming a cloud of positive ions (cations) around the negatively charged clay particles resulting in the formation of flocs (aggregates), as shown in Fig. 11.3.2. Other binding forces are chemical forces (hydrogen bonds, cementation, coatings of organic materials).

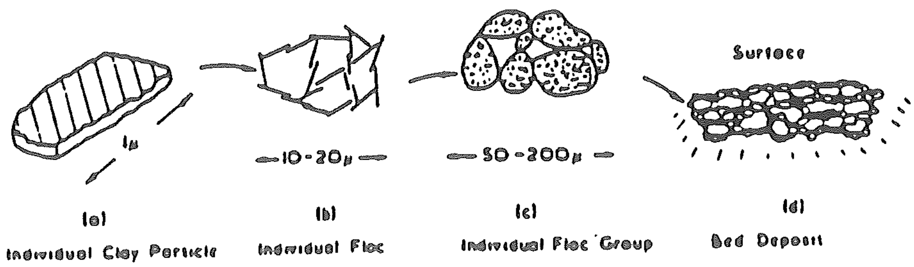


Figure 11.3.2 Sizes of individual clay particles, flocs and floc groups

Flocculation requires particle collisions. The three collision mechanisms are:

- the Brownian motions of the particles (< 4 μm) due to the random bombardment by the thermally agigated water molecules; the number of collisions is linearly proportional to the concentration,
- turbulent mixing due to the presence of velocity gradients in the fluid and
- differential settling velocities because the larger particles have larger settling velocities and may therefore "fall" on the smaller particles.

Other factors affecting flocculation are: size, concentration of particles, salinity, temperature and organic material.

A small particle size in combination with a large concentration greatly intensifies the flocculation process because these two factors yield a small relative distance between the particles.
Experimental research (Krone, 1962) has shown that flocculation quickly reaches an equilibrium situation at a salinity of about 5 to 10 promille which is small compared to that (35 promille) of sea water (Fig. 11.4.1).

A high temperature also enhances the flocculation process because the double layer repulsive energy decreases in magnitude and this leads to a decreased repulsion.
Organic materials in and on the flocs significantly intensify the flocculation process, because of the binding properties of the organic materials. The binding forces become larger due to the presence of organic material (biogenic forces) and the flocs are becoming larger.

Break up of the flocs is caused by large shearing forces in the fluid when these forces are larger than the strength of the flocs, the flocs are broken into smaller flocs or particles. Large shearing forces exist close to the bottom where the velocity gradients are largest. Large shearing forces also exist in small-scale eddies everywhere in the fluid. Under the influence of turbulent forces there is a continuous process of flocculation and break-up resulting in a dynamic equilibrium of the flocs (size, density and strength).

In still water (no turbulence) the flocs may grow to larger sizes due to differential settling collisions. However, as the flocs get larger they fall faster until the fluid shear on the flocs becomes greater than the floc strength resulting in break-up.
Analysis of under-water photographs shows the presence of macroflocs with sizes in the range of 100 to 1000 μm, miniflocs with sizes in the range of 10 to 100 μm and single mineral particles smaller than about 10 μm.

When the flocs grow larger, the floc size increases but the density of the flocs (consisting of sediment, fluid, organic materials) becomes smaller. Figure 11.3.3 shows the excess floc density as a function of floc diameter based on experimental research. Individual clay particles will have an excess density of about 1600 kg/m^3 (ρ_s = 2600 kg/m^3, ρ = 1000 kg/m^3). Large flocs of about 1000 μm may have a density in the range of 1 to 10 kg/m^3 in excess of the fluid density, because most of the floc consists of (pore) fluid.

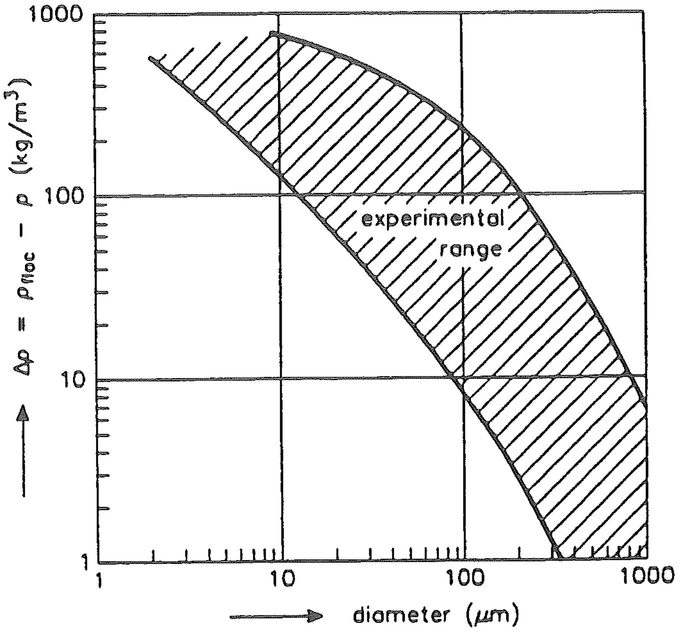


Figure 11.3.3 Differential density (floc density minus fluid density) as a function of floc diameter

11.4 Settling

An important parameter in sedimentation studies of cohesive materials is the settling velocity of the flocs. Analysis of laboratory and field data has shown that the settling velocity of the flocs is strongly related to the salinity, the sediment concentration (c), the water depth, the flow velocity and the type of measuring instrument.

11.4.1 Influence of salinity

Experimental laboratory research shows a clear effect of the salinity on the settling velocity for salinities upto 10 ppt when the sediment concentration is smaller than 1000 mg/l (Krone, 1962, see Fig. 11.4.1). When the sediment concentration is larger than 1000 mg/l, an almost linear increase of the settling velocity with the salinity can be observed (Owen, 1970; Allersma, 1967; see Fig. 11.4.1).

11.4.2 Influence of concentration

In saline suspensions with sediment concentrations upto about 1000 mg/l an increase of the settling velocity with concentration has been observed as a result of the flocculation effect both in laboratory and in field conditions (Fig. 11.4.2).
When the sediment concentrations are larger than approximately 10000 mg/l, the settling velocity decreases with increasing concentrations due to the hindered settling effect (Fig. 11.4.2). Hindered settling is the effect that the settling velocity of the flocs is reduced due to an upward flow of fluid displaced by the flocs. At very large concentrations the vertical fluid flow can be so strong that the upward fluid drag forces on the flocs become equal to the

downward gravity forces resulting in a temporary state of dynamic equilibrium with no net vertical movement of the flocs. This state which occurs close to bed, generally is called fluid mud. In the laboratory the hindered settling velocity can be quite accurately determined from consolidation tests by measuring the subsidence of the sediment-fluid interface.
The settling velocity in the two ranges can be expressed as:

$$w_{s,m} = k\, c^{m} \quad \text{in flocculating suspensions (10-10000 mg/l)} \tag{11.4.1}$$

$$w_{s,m} = w_s\, (1 - \alpha\, c)^{\beta} \quad \text{in hindered-settling suspensions (> 10000 mg/l)} \tag{11.4.2}$$

in which:
$w_{s,m}$ = settling velocity of flocs in fluid-sediment mixture
w_s = settling velocity of individual particles
c = volume concentration
m = coefficient (= 1 to 2)
k = coefficient
α = coefficient
β = coefficient (= 3 to 5)

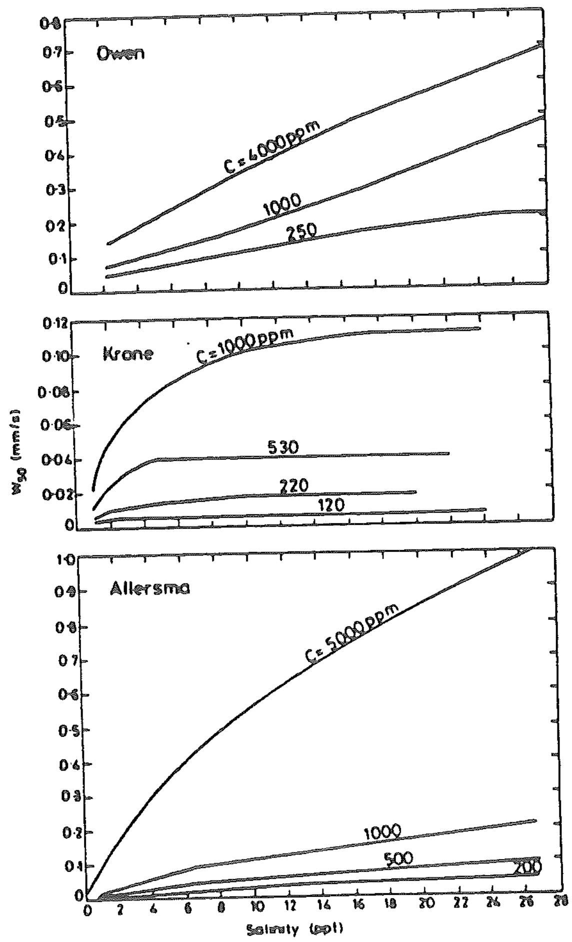


Figure 11.4.1 The influence of salinity on the settling velocity (Burt, 1984)

Settling velocities as a function of concentration in saline conditions from all over the world are shown in Fig. 11.4.2 (Severn, Avonmouth, Thames, Mersey in England; Western Scheldt in The Netherlands; River Scheldt in Belgium; Brisbane in Australia; Chao Phya in Thailand, Demerara in South America).

The settling velocities in flocculating suspensions were mostly determined in still water by analyzing fluid-sediment samples taken from the original suspension and may not represent the actual settling velocity in a natural turbulent flow.

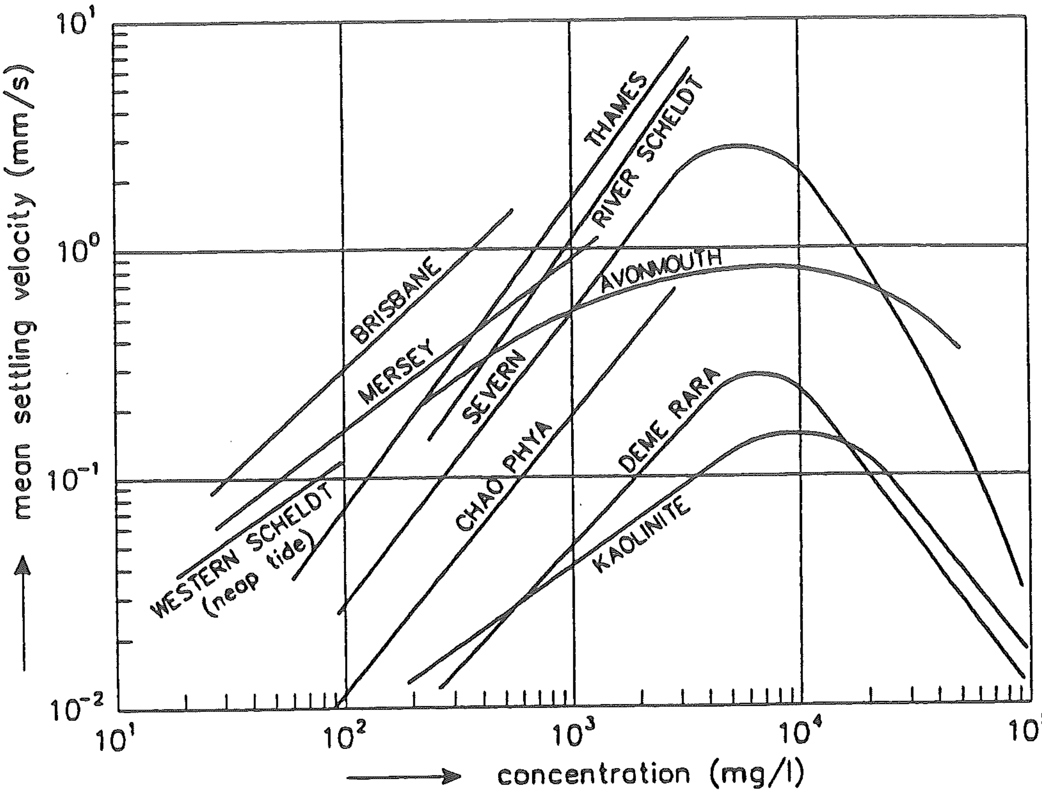


Figure 11.4.2 The influence of sediment concentration on the settling velocity

11.4.3 Influence of water depth and flow velocity

Analysis of settling velocities in fluids of different water depths shows a marked influence of the water depth, which is caused by the differential settling effect. Owen (1970) found that a minimum height of 2 m was required to achieve maximum flocculation and settling velocities (by differential settling). Differential settling is the effect that the larger flocs have larger settling velocities and may therefore "fall" on the smaller flocs forming new larger flocs. Consequently, in natural conditions the settling velocities in the lower layers will be larger than those in the upper layers, especially during the slack water period of the tide, when the disruptive turbulent fluid forces are absent. Figure 11.4.3 shows an example of concentration and settling velocity profiles at different times in a laboratory settling column with a height of 4 m (Cornelisse et al, 1990). Low-level turbulence was generated by an oscillating grid (f = 0.025 Hz) giving an effective mixing coefficient of 3.10^{-5} m^2/s. The settling velocities were computed from the (vertical) mass-balance equation using the measured concentration profiles as input data. As can be observed, the settling velocities are largest close to the bottom. The settling velocity is maximum after about 1600 s, which is the time scale to obtain maximum flocculation.

The settling process in the laboratory column is representative for the settling process in the slack water period (low velocities) of a tidal cycle in an estuary. At higher flow velocities the settling velocities near the bed may be significantly reduced due to the presence of disruptive shear forces (velocity gradients) in the boundary layer, as shown schematically in Fig. 11.4.4. Larger flocs will be broken down in smaller flocs, the latter being resuspended in the flow.

Burt (1984) found no influence of the tidal range (neap-spring cycle) on the settling velocity based on the analysis results of about 200 samples collected during a period of several years in the Thames estuary (England). He concluded that the flocs are mainly affected by eddy scales comparable to the floc size scales. These smaller eddy scales will be present during nearly all tidal conditions with exception of the slack tide periods.

11.4.4 Influence of measuring instrument

Figure 11.4.2 clearly shows that general relationships to determine the settling velocity of the flocs at a particular location are not available. The best approach is to do field measurements when there is no information available. Since, the settling and deposition process is most effective during the slack water period of the tidal flow, the measurements should be concentrated in this period. Sampling positions should be located near the water surface, at middepth and near the bed to determine differential settling effects (Fig. 11.4.4). These type of measurements are usually carried out with an in-situ settling tube, which is lowered horizontally to the sampling position to trap a fluid-sediment sample (by closing two valves on both ends of the tube). The sample container is rotated into a vertical position and the settling process starts (in still water). The settling velocity can be determined by measuring the sediment concentration in the container as a function of time. The length of the settling tube should be small ($\approx$ 0.3 m) to avoid differential settling in the tube.

Van Leussen and Cornelisse (1991) used an in-situ video camera system and an in-situ settling tube to determine floc sizes and settling velocities in the Ems estuary in The Netherlands. The in-situ video system consists of a small vertical tube with a closed end at the bottom in which particles are settling down in still water. Two small windows are present in the tube for enlightning (light beam) and for videorecordings (camera). The instrument was connected by a signal cable to the survey vessel which floated with the flow during the sampling period. Floc sizes and settling velocities were obtained from the recordings by computer analysis. Figure 11.4.5 shows settling velocities based on the in-situ video system and the in-situ settling tube. The video recordings showed the presence of a large amount of relatively large flocs with large settling velocities during the maximum flow period. During the slack tide period much smaller flocs and settling velocities were observed.

The large settling velocities around maximum flow were not found from the settling tube results.
Boere (1987) determined the settling velocities in the Western Scheldt Estuary by analyzing sediment concentration profiles measured during the slack water period.
Neglecting horizontal diffusive transport ($\varepsilon_s\, \partial^2 c/\partial x^2 \approx 0$) and vertical convective transport ($w = 0$), the fall velocity can be determined from integration of the two-dimensional vertical mass balance equation, yielding:

$$w_{s,z} = \frac{1}{c}\left[-\int_z^h (\partial c/\partial t)dz - \int_z^h u(\partial c/\partial x)dz - \varepsilon_s \partial c/\partial z \right] \tag{11.4.3}$$

in which: $w_{s,z}$ = fall velocity at height z, c = concentration at height z, t = time, u = velocity at height z, ϵ_s = mixing coefficient at height z.
The settling velocities based on this method were found to be considerably larger (factor 10) than those derived from settling tube measurements. Based on the above-given results, it may be concluded that the use of an in-situ settling tube may result in relatively small settling velocities because of floc destruction during the sampling procedure. The video camera system may offer promising results, but it should be realized that the larger flocs are better observed by the camera than the smaller flocs. The in-situ settling tube represents the total distribution of particles, but the larger flocs may be destroyed during closing of the valves to trap the fluid-sediment sample.

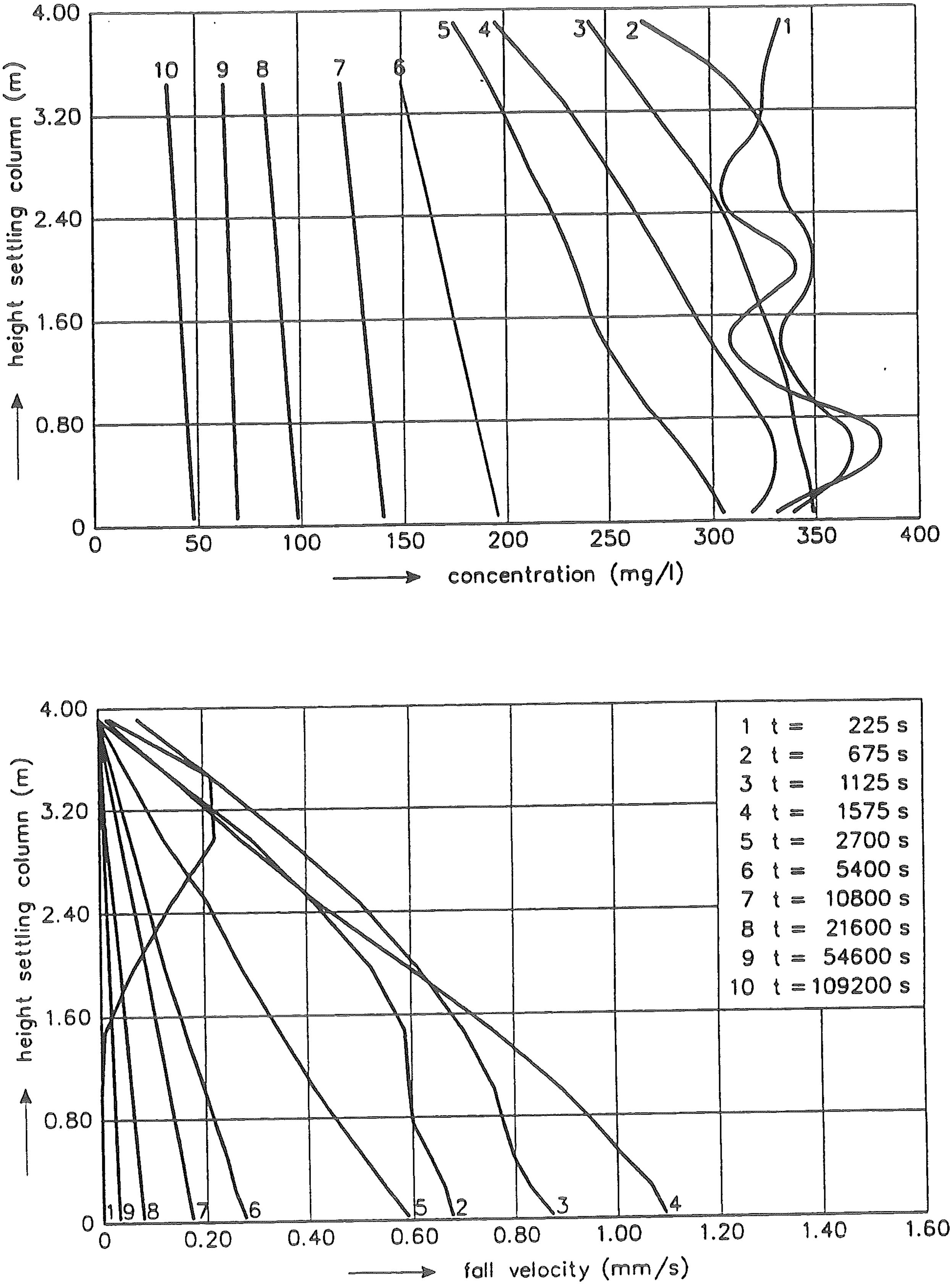


Figure 11.4.3 Concentration and settling velocity profiles at various times Mud Ketelmeer (lake), c_o = 0.5 kg/m^3, fresh water, f = 0.025 Hz

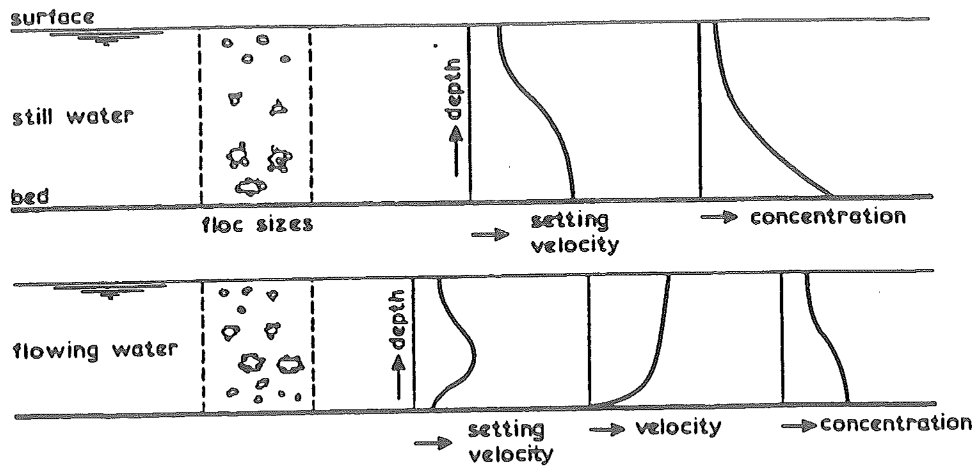


Figure 11.4.4 Settling and flocculation in still and flowing water

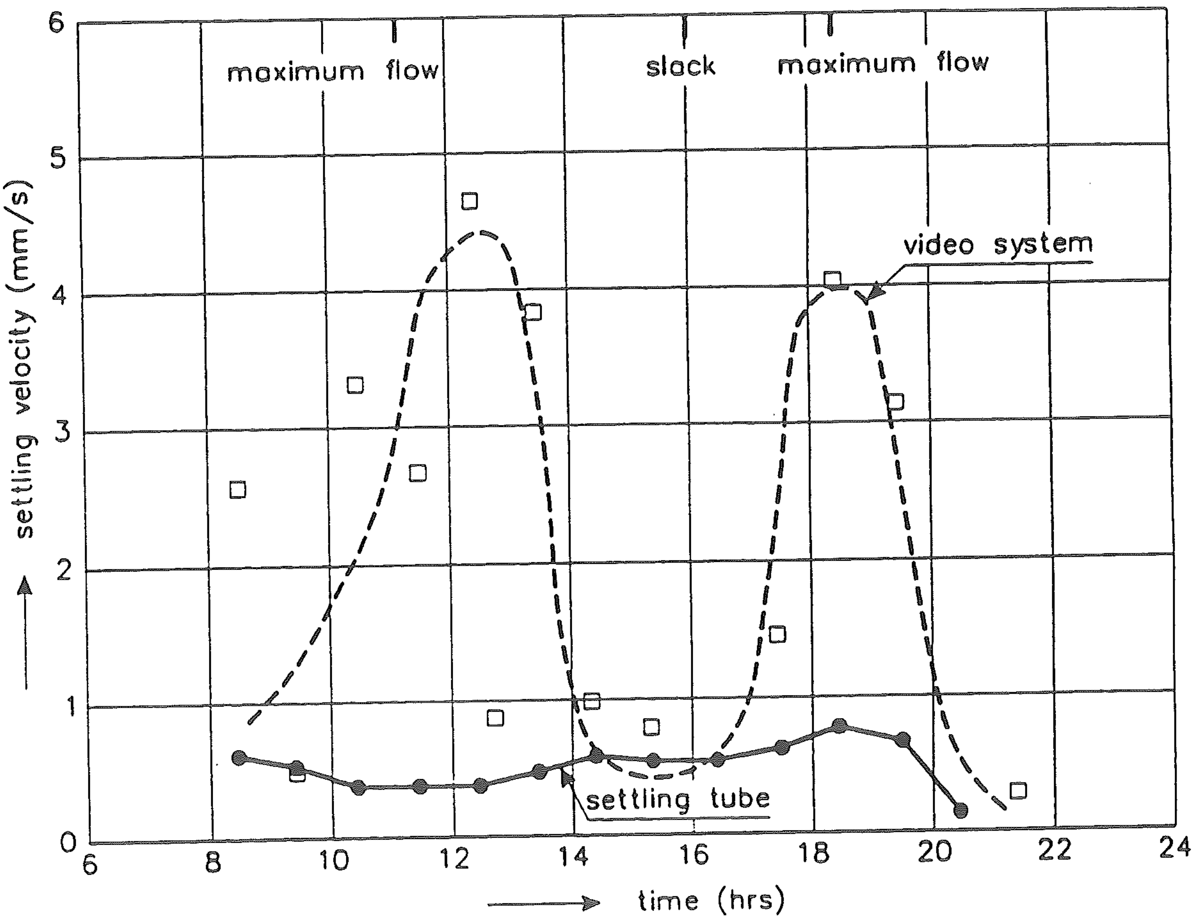


Figure 11.4.5 Settling velocities at 2.8 m below the surface over a tidal cycle in the Ems-estuary

11.5 Deposition

11.5.1 Introduction

Deposition is predominant when the bed-shear stress (τ_b) falls below a critical value for deposition (τ_d), as shown by Krone (1962). He studied the deposition process of cohesive material by circulating natural mud at various flow velocities in a laboratory flume with saline water. The material (mud from San Francisco Bay) had a particle size distribution with 60% smaller than 2 μm and 50% smaller than 1 μm.
Figure 11.5.1 shows the deposition processes in case of an initial mud concentration of 20 kg/m³ and a flow velocity of about 0.1 m/s. Three distinct periods depending on the mud concentration can be observed:

- a period of about 10 hours with relatively (slow) deposition (due to the hindered settling effect) and (suspension) concentrations decreasing to about 10 kg/m³,
- a period of about 100 hours with more rapid deposition (due to the flocculation effect) and (suspension) concentrations decreasing from about 10 kg/m³ to about 0.3 kg/m³,
- a final period of long duration with a very slow deposition of concentrations smaller than 0.3 kg/m³. Deposition in this stage could be increased significantly by placing a grid in the flow (to intensify flocculation).

11.5.2 Concentrations larger than 10 kg/m³

A fluid mud of sediment flocs was found to exist at concentrations above 10 kg/m³ in which the flocs are partially supported by the escaping fluid (known as the hindered settling effect) and partially supported by interfloc contacts (Fig. 11.5.1).
In nature where the water depths are large, there will be a situation with concentrations increasing towards the bed. The settling velocity reduces for increasing concentrations (hindered settling). Thus, the settling velocity will be relatively small near the bed and relatively large at the upper layers resulting in the formation of a near-bed fluid mud layer (especially in neap-tide conditions) with increasing sediment concentrations in the range of 10 to 300 kg/m³. The thickness of the fluid mud layer will increase as long as the deposition rate at the upper side of the layer is larger than the consolidation rate at the bottom side. Values up to several meters have been observed. The fluid mud layer can be transported horizontally as a turbidity current due to:

- gravity forces on a sloping bottom,
- pressure gradient forces due to horizontal differences in sediment concentrations and fluid densities,
- shear forces at the interface generated by the overflowing water.

Turbulent mixing at the interface (lutocline) between the fluid mud layer and the overlying water generally is small because of the stabilizing effect of the heavier sediment material which has to be moved against gravity. Kirby and Parker (1980, 1983) have observed fluid mud layers with concentrations > 15 kg/m³ transported with velocities of about 0.3 m/s in the Severn Estuary in England (see also Section 11.8.2).

11.5.3 Concentrations from 0.3 to 10 kg/m³

The deposition process in this concentration range is dominated by flocculation effects, as shown by the increase of the deposition rate in the flume experiment of Krone, 1962 (see Fig. 11.5.1).
The decrease in time of the concentration was, as follows:

$$\log(c/c_o) = -K \log(t/t_o) \qquad (11.5.1)$$

in which:

c_o = initial concentration (at $t = t_o$)

t = time

K = coefficient

Krone conducted his deposition experiments in a straight flume under steady flows with bed-shear stresses smaller than the critical bed-shear stress for full deposition.

Mehta and Partheniades (1975) performed experiments with cohesive materials (initial concentrations of 1 to 10 kg/m^3) under steady flow conditions with bed-shear stresses smaller and larger than the critical bed-shear stress for full deposition (see Figs. 11.5.4, 11.5.5 and 11.5.6).

The experiments were performed in a straight flume and in a circular flume (see Fig. 11.1.1). The circular flume was covered by a circular ring in contact with the water surface. A simultaneous rotation of the two components in opposite directions generated an uniform turbulent flow with minimum rotation-induced secondary currents. Figure 11.5.5 shows some experimental results. After a short period of rapid deposition an equilibrium concentration (c_{eq}) is obtained. Flocs with a low shear resistance are broken down in the near-bed layer with large velocity gradients whereas the smaller flocs and particles are resuspended. Flocs with a high shear resistance (strong flocs) can be deposited. The equilibrium concentration was found to be dependent on the applied bed-shear stress (τ_b), the type of sediment material and the initial concentration (c_o). Mehta and Partheniades also found a constant ratio c_{eq}/c_o at a constant bed-shear stress (τ_b) for various initial concentrations (c_o). The ratio c_{eq}/c_o which is dependent on the τ_b-value is shown in Fig. 11.5.6. The c_{eq}/c_o-ratio and the variation in time of the concentration c from c_o to c_{eq} can both be represented by logarithmic-normal relationships (Mehta and Partheniades, 1975).

The experimental results can be interpreted by assuming that there are two groups of flocs. The first have sufficiently strong bonds to resist the disruptive near-bed shear stresses and will be able to reach the bottom and form strong bonds with deposited flocs. The second does not have sufficiently strong bonds and will be broken down before reaching the bed resulting in resuspension or are eroded very quickly after being deposited because the bond between the floc and the bed was relatively weak. The ratio c_{eq}/c_o represents the percentage of the flocs that remains in suspension and that have a floc strength smaller than the bed-shear stress.

The deposited flocs develop bonds with the bed flocs. The bed-shear stress to disrupt these bonds and to overcome the submerged floc weight has to be larger than that at which the floc is deposited. This larger value is called the critical bed-shear stress for erosion (τ_e).

In one experiment Mehta and Partheniades slowly replaced the fluid-sediment mixture (having the equilibrium concentration c_{eq}) by clear water and found no measurable sediment concentrations in the replaced water. This indicated that there was no erosion of the deposited strong flocs. Thus, there was no process of simultaneous deposition and erosion (equilibrium) for the group of strong flocs. This does not rule out the possibility that there may be simultaneous deposition and erosion for the group of weak flocs or individual particles.

Based on the experimental results of Mehta and Partheniades, the following processes can be distinguished (see also Fig. 11.5.6):

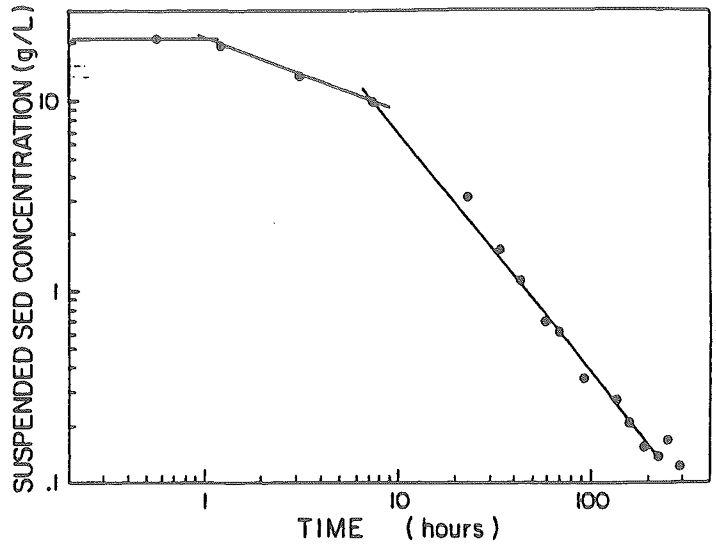


Figure 11.5.1 *Deposition experiments of Krone (1962)*
$c_o = 20\ kg/m^3$, $\tau_b < \tau_{d,full}$ *(full deposition)*

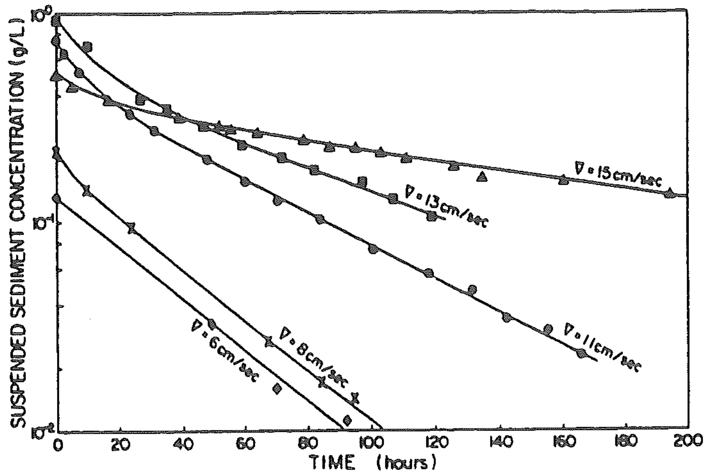


Figure 11.5.2 *Deposition experiments of Krone (1962)*
$c_o = 0.1\text{-}1\ kg/m^3$, $\tau_b < \tau_{d,full}$ *(full deposition)*

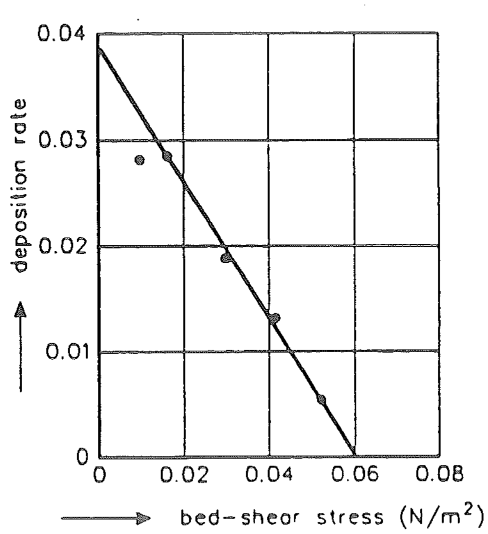


Figure 11.5.3 *Deposition rates according to Krone (1962)*
$c_o = 0.1\text{-}1\ kg/m^3$, $\tau_b < \tau_{d,full}$ *(full deposition)*

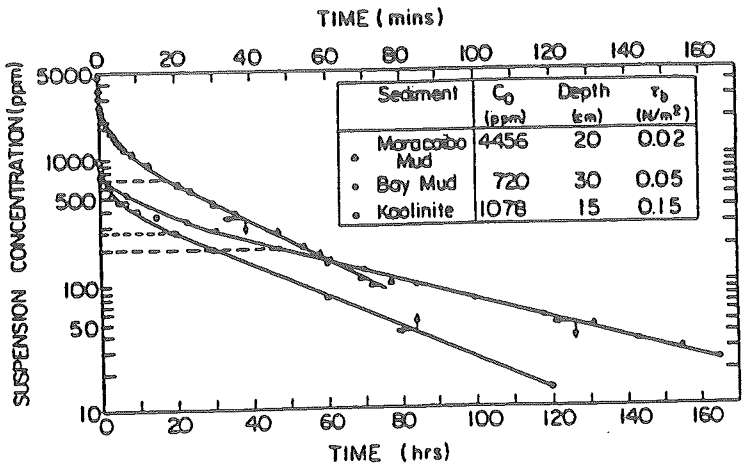


Figure 11.5.4 Deposition experiments of Mehta (1984), straight flume c_o = 1-5 kg/m^3, $\tau_b < \tau_{d,full}$ (full deposition)

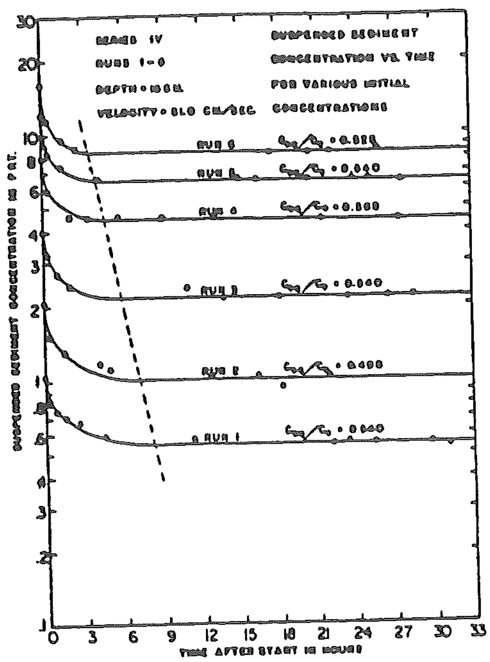


Figure 11.5.5 Deposition experiments of Mehta and Partheniades (1975) c_o = 1-10 kg/m^3, $\tau_b > \tau_{d,full}$ (partial deposition)

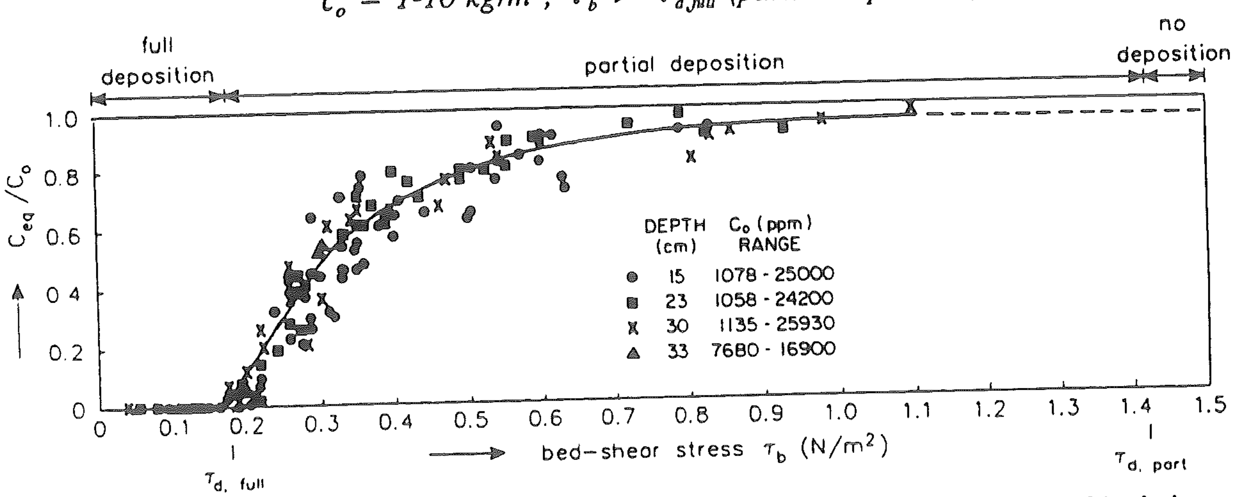


Figure 11.5.6 Ratio of equilibrium and initial concentration as a function of bed-shear stress, Mehta and Partheniades (1975)

1. Full deposition

All sediment particles and flocs are deposited when the bed-shear stress (τ_b) is smaller than the bed-shear stress for full deposition ($\tau_{d,full}$).
The deposition rate (D) is:

$$D = c\ \alpha\ w_{s,m} \qquad \text{for} \quad \tau_b < \tau_{d,full} \tag{11.5.2}$$

in which:
c = sediment concentration
$w_{s,m}$ = (concentration-dependent) settling velocity distribution in still water
α = coefficient (≤ 1)

The deposition rate D of Eq. (11.5.2) should be read as a quantity obtained by summation over size fractions based on the settling velocity distribution.

Equation (11.5.2) is valid for uniform concentrations over the depth, otherwise the near-bed concentration should be used.
The α-coefficient can be interpreted as a reduction factor of the settling velocity (in still water) due to the presence of turbulence and small concentration gradients yielding an upward sediment flux ($\epsilon_s\ dc/dz$). Consequently, the α-coefficient will be dependent on the bed-shear stress: $\alpha = f(\tau_b)$ with $\alpha = 1$ for $\tau_b = 0\ N/m^2$ and $\alpha < 1$ for $\tau_b > 0\ N/m^2$.
Mehta and Partheniades (1975) found $\tau_{d,full} = 0.15\ N/m^2$ for kaolinite in distilled water.

2. Hindered or partial deposition

The group of relatively strong flocs is deposited, whereas the group of relatively weak flocs with a shear strength smaller or equal to the applied bed-shear stress τ_b (see Fig. 11.5.6) remains in suspension. The relative magnitude of both groups depends on the bed-shear stress. The deposition rate can be represented as:

$$D = (c - c_{eq})\ \alpha w_{s,m} \qquad \text{for} \quad \tau_{d,full} < \tau_b < \tau_{d,part} \tag{11.5.3}$$

3. No deposition

No sediment flocs are deposited when the bed-shear stress is larger than the maximum bed-shear stress for deposition ($\tau_{d,part}$). Thus, no flocs are present with a shear strength larger than $\tau_{d,part}$. The deposition rate is equal to zero:

$$D = 0 \qquad \text{for} \quad \tau_b > \tau_{d,part} \tag{11.5.4}$$

11.5.4 Concentrations smaller than 0.3 kg/m³

Krone (1962) performed a series of deposition experiments in a flume at low concentrations. The flocculation effect appeared to be of minor importance. In each experiment a constant concentration (c_o) was established in the flume while the water was recirculating. Then, the flow velocity was decreased to start the deposition process with decreasing concentrations, as shown in Fig. 11.5.2.

Assuming a uniform concentration profile, the deposition rate can be expressed as:

$$D_{net} = -\frac{d(hc)}{dt} = c\ \alpha\ w_{s,m} \tag{11.5.5}$$

in which:

c = concentration at time t
h = water depth
$w_{s,m}$ = settling velocity of particles or flocs at $\tau_b = 0$ (zero flow)
t = time
α = coefficient

The quantity (hc) represents the volume (or mass) in suspension per unit area.

Integration of Eq. (11.5.5) yields the decrease of the concentration in time (assuming constant settling velocity):

$$c = c_o\, e^{-(\alpha\, w_{s,m}/h)\, t} \tag{11.5.6}$$

in which:
c_0 = initial concentration at t = 0.

Substitution of Eq. (11.5.6) in Eq. (11.5.5) yields:

$$D_{net} = -\frac{d(hc)}{dt} = \alpha\, w_{s,m}\, c_0\, e^{-(\alpha\, w_{s,m}/h)\, t} \tag{11.5.7}$$

The deposition rates measured by Krone (1962) are shown in Fig. 11.5.3 as a function of the applied bed shear stress (τ_b).
Extrapolation of the line indicates:

- no net deposition at $\tau_b > 0.06$ N/m²
- maximum deposition rate at $\tau_b = 0$ yielding a settling velocity $w_{s,m} = 6.6\ 10^{-6}$ m/s which is equivalent to a Stokes diameter of 2.5 μm.

According to Krone (1962), the settling velocity in Eq. (11.5.5) is the value measured at $\tau_b = 0$ (zero flow). The value of $w_{s,m} = 6.6\ 10^{-6}$ m/s (equivalent to a stokes diameter of 2.5 μm) indicates that flocculation is of minor importance (Stokes diameter is about equal to that of the individual particles). Flocculation increased significantly when a grid was placed in the flume to create extra turbulence and hence flocculation (more particle collisions).

The α-coefficient of Eq. (11.5.5) was found to be

$$\alpha = (1 - \tau_b/\tau_{d,full}) \tag{11.5.8}$$

with $\alpha = 0$ for $\tau_b = \tau_{d,full}$ and $\alpha = 1$ for $\tau_b = 0$

in which:
τ_b = applied bed-shear stress
$\tau_{d,full}$ = critical bed-shear for full deposition

The term $\alpha\ w_{s,m}$ represents the apparent settling velocity in flowing water (see Eq. (11.5.2). Based on the experimental results of Krone (1962) for low concentrations ($c < 0.3$ kg/m³), the following processes can be distinguished:

no deposition : $D = 0$ for $\tau_b \geq \tau_{d,full}$ (11.5.9)

full deposition : $D = \left(1 - \dfrac{\tau_b}{\tau_{d,full}}\right) c\ w_{s,m}$ for $\tau_b < \tau_{d,full}$ (11.5.10)

According to Eq. (11.5.7), all sediment particles will eventually be deposited yielding full deposition. The range of hindered or partial deposition was not observed by Krone. Probably for low concentrations there is a sharper transition from full deposition to no deposition because the floc sizes will be small (infrequent particle collisions) yielding a relatively uniform floc size and floc strength distribution. In a high-concentration suspension the floc size and floc strength distributions will be rather wide resulting in partial deposition in a certain bed-shear stress range.

11.5.5 Critical bed-shear stresses for deposition

Based on the experimental work of Mehta and Partheniades (1975), two critical bed-shear stresses for deposition can be distinguished: $\tau_{d,full}$ and $\tau_{d,part}$.
The minimum bed-shear stress for full deposition ($\tau_{d,full}$) is defined as the bed-shear stress below which full deposition of the sediment flocs will occur. The maximum bed-shear stress for deposition ($\tau_{d,part}$) is defined as the bed-shear stress above which no deposition of the sediment flocs will occur. For a bed-shear stress in the range between $\tau_{d,full}$ and $\tau_{d,part}$ partial deposition may occur (see Fig. 11.5.6).
Experimental values of ($\tau_{d,full}$) and ($\tau_{d,part}$) have been presented by Mehta (1984) and by Winterwerp et al. (1991).
Mehta (1984) used three sediments: a mud from Maracaibo Estuary (Venezuela) and San Francisco Bay mud (USA), both in 35 ppt salt water and kaolinite in distilled water. The muds initially suspended at high velocities in a flume were allowed to deposit at a lower velocity. The initial concentrations were in the range of 1 to 5 kg/m^3 (see Fig. 11.5.4).
Winterwerp et al (1991) tested various natural muds from lakes, rivers and estuaries in The Netherlands in a circular flume using native fluids. The initial concentrations were in the range of 0.1 to 1 kg/m^3. The method of Mehta and Partheniades (1975) was used to determine the ($\tau_{d,full}$)-values.
The results of Mehta (1984) and Winterwerp et al. (1991) are given in Table 11.3.

Mud type		Sand (%)	Organics (%)	Critical deposition bed-shear stress (N/m^2)	
				$\tau_{d,full}$	$\tau_{d,part}$
kaolinite	(saline)	-	-	0.05	-
kaolinite	(distilled)	-	-	0.15	1.4
Hollands Diep 1	(lake)	9	10	0.10	-
Hollands Diep 2	(lake)	23	9	0.08	-
Ketelmeer	(lake)	7	12	0.08	-
Biesbosch	(lake)	8	8	0.04	-
Maas	(river)	36	8	0.06	-
Breskens Harbour	(estuary)	27	5	0.06	-
Delfzijl Harbour	(estuary)	60	2	0.03	-
Loswal Noord	(sea)	69	2	0.08	-
San Francisco Bay		-	-	0.10	1.7
Maracaibo estuary		-	-	0.08	1.6

Table 11.3 Critical bed-shear stresses for deposition

11.5.6 Deposition rates

The maximum deposition rate ($D_{max} = c\ w_{s,m}$) will occur at low flow conditions as present during slack tide. Figure 11.5.7 shows the (maximum) deposition rate as a function of concentration, based on a settling velocity in the range of 10^{-6} to 10^{-3} m/s. As can be observed, the deposition rate is largest for a concentration of about 10 kg/m³. The variation of the deposition rate is largest in the flocculation range and smallest in the hindered settling range.
At higher (tidal) flow velocities the deposition rates will be reduced due to the effect of turbulence resulting in a smaller apparent settling velocity ($w^{*}_{s,m} = \alpha\ w_{s,m}$ with $\alpha \le 1$).

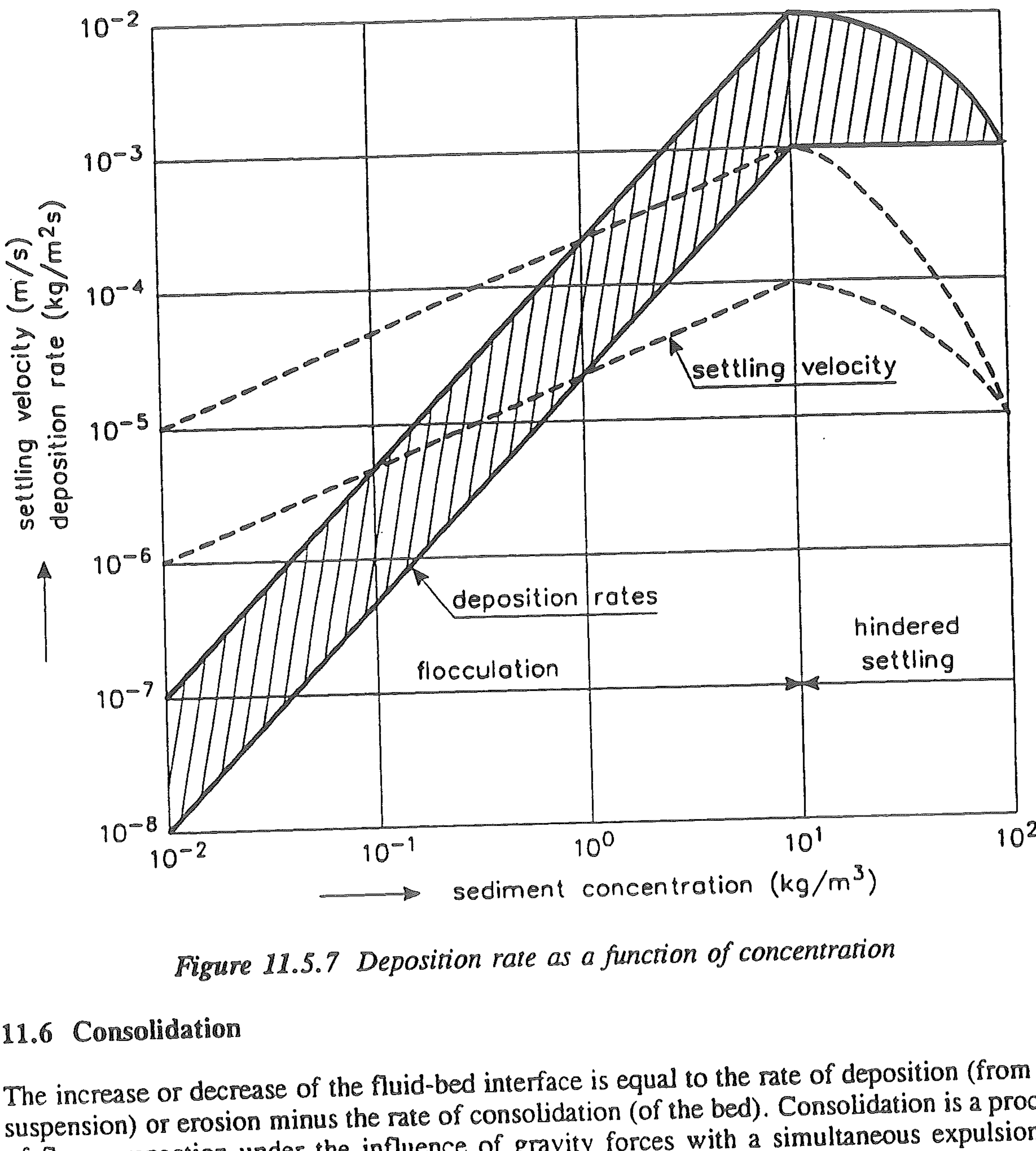


Figure 11.5.7 Deposition rate as a function of concentration

11.6 Consolidation

The increase or decrease of the fluid-bed interface is equal to the rate of deposition (from the suspension) or erosion minus the rate of consolidation (of the bed). Consolidation is a process of floc compaction under the influence of gravity forces with a simultaneous expulsion of pore water and a gain in strength of the bed material.

Generally, three consolidation stages are distinguished (see also Fig. 11.6.1):

- initial stage (days) : The process consists of hindered settling and consolidation. The flocs in a freshly deposited layer are grouped in an open structure with a large pore volume. The weakest bonds are broken down first and the network gradually collapses. The bed surface sinks with time t.
- secondary stage (weeks) : The pore volume between the flocs is further reduced. Small thin vertical pipes (drains) are formed allowing the pore water to escape. The bed surface sinks with $t^{0.5}$ or log(t).
- final stage (years) : The pore volume inside the flocs is further reduced and the flocs are broken down. The bed surface sinks with log(t).

The consolidation process is strongly affected by the:

- initial thickness of the mud layer (h_o)
- initial concentration of the mud layer (c_o),
- permeability (k) of the mud layer (sediment composition and size, content of organic material, salinity, water temperature).

Gibson et al (1967) formulated an equation to describe the consolidation process. The consolidation equation expresses the void ratio (e) as a function of the vertical coordinate (z) and time (t). The void ratio (e) is the ratio of the void volume and the solid volume. The consolidation equation represents: the vertical equilibrium of the soil, the flow (Law of Darcy) and continuity of the pore fluid. The soil skeleton is assumed to be homogeneous. A review has been given by Kuijper (1992).

Based on experimental results, the functional relationship of the consolidation time scale (T_c) was found to be:

$$T_c = f\left(h_o^2, \frac{1}{k}\right) \tag{11.6.1}$$

Figure 11.6.3 shows the consolidation process in a layer of pure kaolinite material in saline water (31‰). The initial height of the uniform suspension was h_o = 0.1 m and h_o = 0.3 m. The initial sediment concentrations (or dry densities) were 25, 50, 100, 175, 250 and 500 kg/m³ (Van Rijn, 1985). The tests consisted of measuring the height (h) of the kaolinite surface above the bottom of the consolidation column as a function of time. The various consolidation phases can be clearly observed. A thin layer (h_o = 0.1 m) of mud consolidates faster than a thick layer (h_o = 0.3 m) with the same initial concentration (c_o) because the pore water must travel over a longer distance to the mud surface. In a thick layer the kaolinite material is however more compacted (higher density) after the same consolidation period (7 days) due to the larger overburden (weight), as shown in Figure 11.6.3. Owen (1975) found opposite results for natural mud. A thicker final bed layer resulted in a lower dry density, probably due to the presence of organic materials.
A higher initial concentration will of course give a larger final bed thickness and thus a larger consolidation time. The final density also increases with increasing initial concentrations, probably because more dense flocs are formed (see Figure 11.6.3).

The consolidation behaviour of a natural mud is different (depending on the size ranges and the content of organic material) from that of pure kaolinite material.
Figure 11.6.2 shows the consolidation process of a very fine natural mud (without sand) from the Bangkok Bar Channel in saline water (Nedeco, 1985). Different initial suspension heights (h_o) and wet densities (ρ_o) were tested.
Similar to the consolidation of pure kaolinite, the consolidation process of the Bangkok mud proceeds relatively fast in a thin layer and relatively slow in a thick layer. The mud samples with a low initial density tend to have a somewhat lower final density than the samples with the higher initial density (similar to the pure kaolinite case).

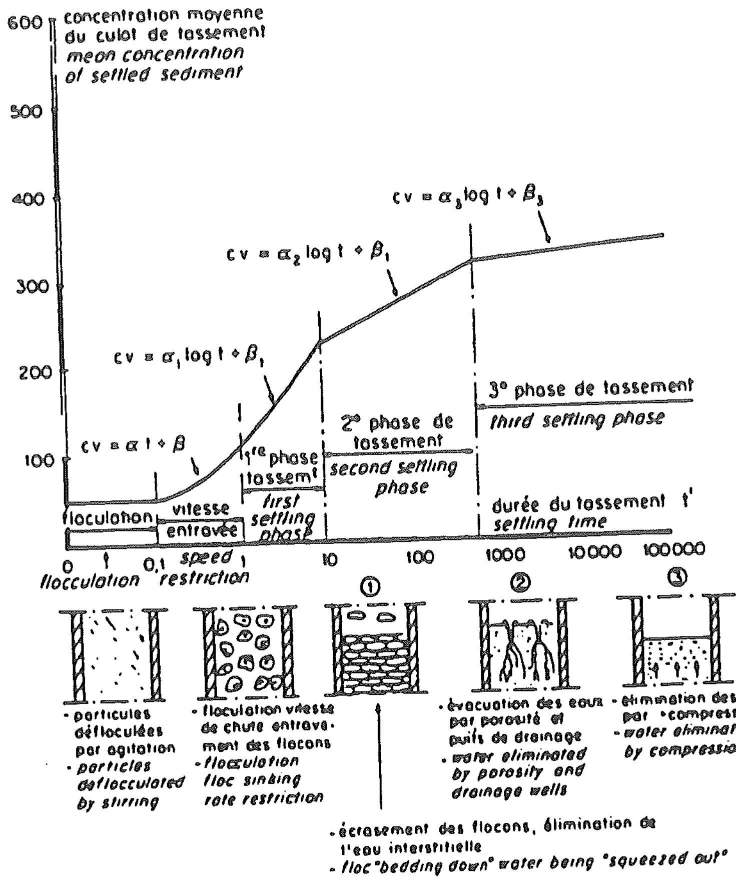


Figure 11.6.1 Consolidation phases (Migniot, 1989)

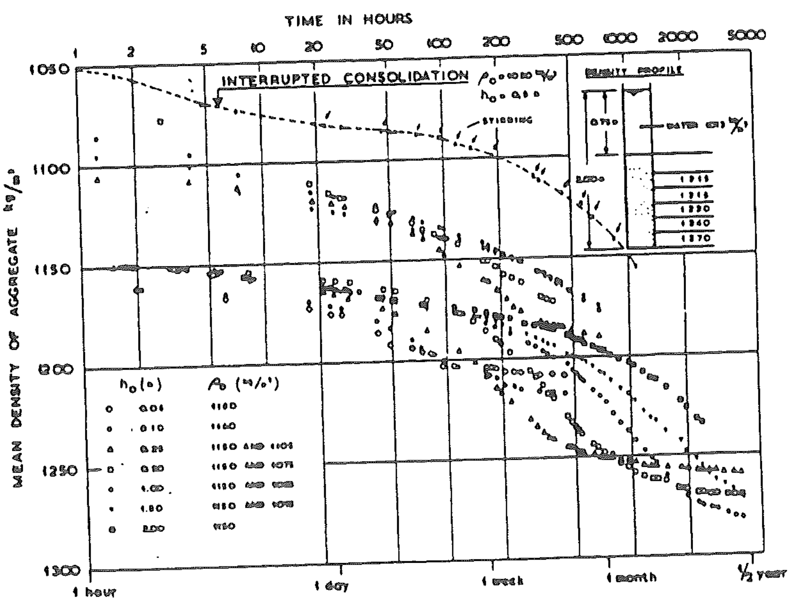
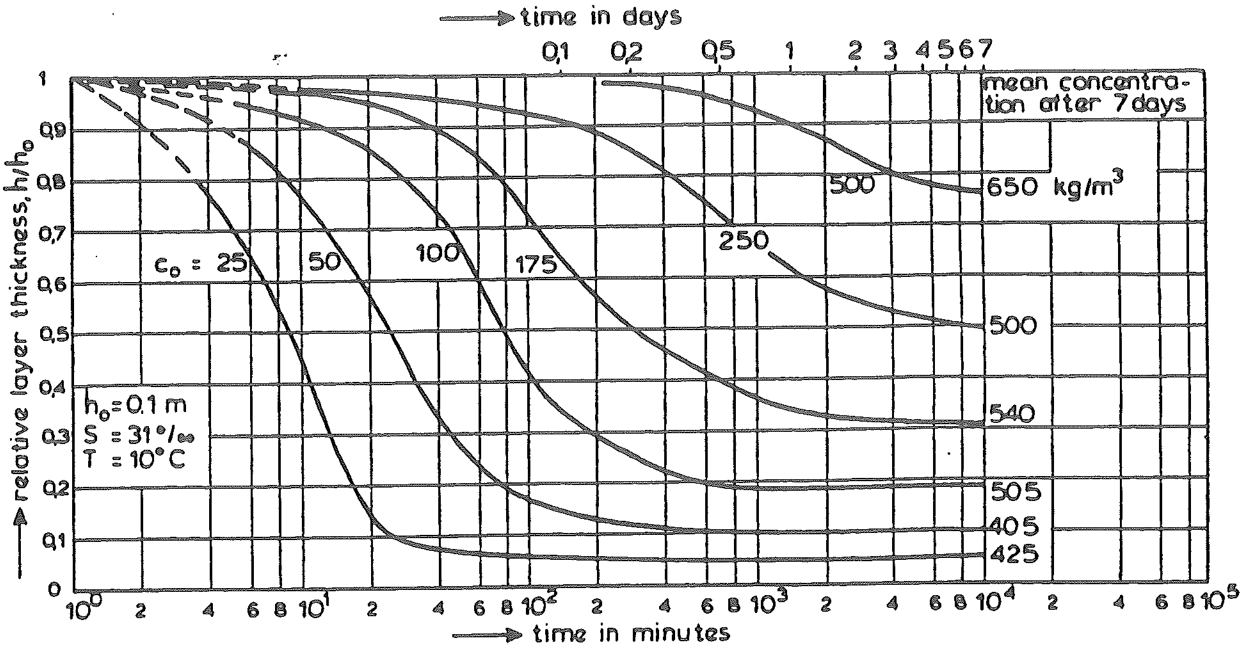
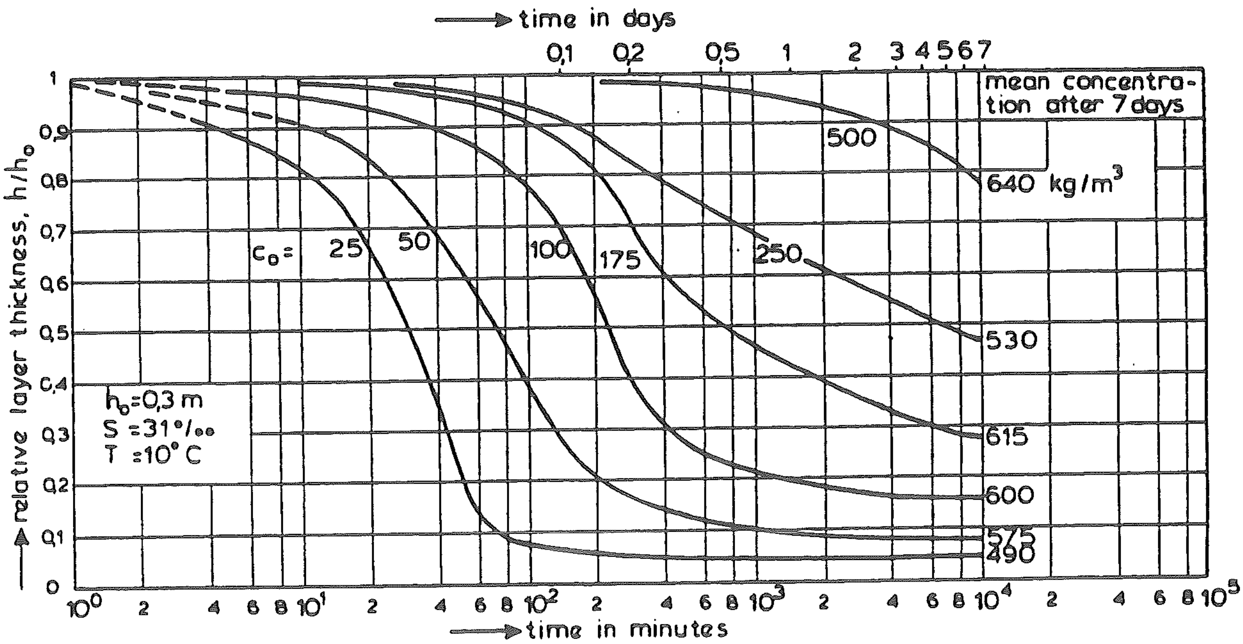


Figure 11.6.2 Wet sediment density as a function of time Bangkok mud (Nedeco, 1985)



A. CONSOLIDATION TEST, h_0=0.1m



c_0= initial concentration in kg/m³

B. CONSOLIDATION TEST, h_0=0.3m

Figure 11.6.3 Mud layer thickness as a function of time Kaolinite in saline water (Van Rijn, 1985)

In one experiment the consolidated material was stirred at regular intervals to simulate wave-induced and current-induced motions as present in nature, yielding a retardation of the consolidation process. The consolidation time scale of the Bangkok mud is much larger than that of pure kaolinite. A kaolinite layer with a thickness of 0.1 m obtains its final density after about 7 days, whereas the Bangkok mud shows a time scale of about 1 month. This latter behaviour is probably related to the presence of organic materials (with a dry density of about 1200 to 1500 kg/m³) which cause biochemical reactions resulting in gas production. Gas bubbles may escape during the initial consolidation phase when the material strength is low, thereby retarding the consolidation process. During later stages (or deeper layers) with increasing material strength, the gas bubbles may remain in the bed and affect the consolidation process, depending on the permeability of the bed material.

The influence of the sediment size and composition of natural muds on the consolidation process can be clearly observed from Figure 11.6.4, showing consolidation curves of natural muds (with different sand percentages) from lakes and estuaries in The Netherlands. The initial layer thickness and concentration were about the same for all tests ($h_o = 0.3$ m, $c_o = 50$ kg/m³). The consolidation curve of pure kaolinite in saline water is also shown. During the initial phase the consolidation of the natural muds proceeds much faster than that of pure kaolinite material due to the presence of high percentages of sand particles. During the final stage the consolidation of the natural muds proceeds considerably slower probably due to the presence of organic materials. The two mud samples with the highest content of organic material have the highest final bed thickness.

Information of the vertical distribution of the sediment concentration (dry density) of consolidated mud layers is given in Figure 11.6.5 based on data of Owen (1970), Thorn and Parson (1980) and Winterwerp et al (1991). The latter used an initial consolidation height of 0.3 m, whereas the former used values of 4.5 to 10 m. Thin mud layers show higher densities in the upper layers ($z > 0.5$ h) than thick mud layers. The relative densities in the top surface layer are about the same for all muds. Salinity had no marked influence on the density distribution (Owen, 1970).

Absolute values of the dry sediment density in top and bottom layer are given in Table 11.4. The top surface layer is defined as the surface layer with a thickness smaller than about 10 mm. The samples with a very high percentage of sand (> 50%) showed a top layer of mud on a lower layer of sand due to differential settling velocities. The dry densities in the top layer are in the range of 50 to 250 kg/m³ depending on the sediment composition and consolidation time. The dry densities in the bottom layer are in the range of 400 to 1500 kg/m³ depending on the percentage of sand.

The density profile of natural mud can be represented by

$$c_z = \alpha\, \bar{c} \left(\frac{z}{h}\right)^{-\beta} \quad \text{for} \quad 0.1 \le z/h \le 0.9 \tag{11.6.2}$$

in which:
c_z = dry sediment density at height z above "hard" bottom
$\bar{c}$ = average dry sediment density of layer
h = thickness of mud layer
α,β = coefficients ($\alpha + \beta = 1$)

The β coefficient is about 0.3 to 0.6 for natural muds with low sand percentages (< 5%). Equation (11.6.2) is not accurate in the upper and lower layers.

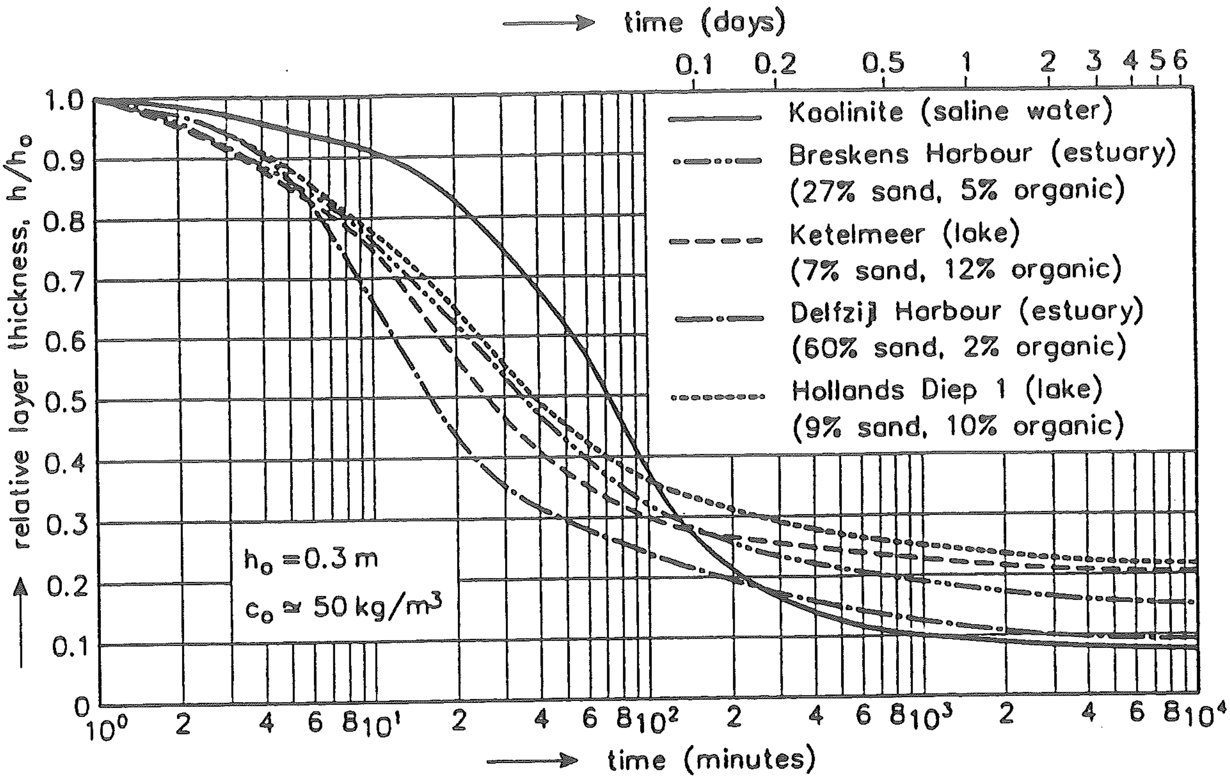


Figure 11.6.4 *Mud layer thickness as a function of time*
Kaolinite and natural muds (Winterwerp et al, 1991)

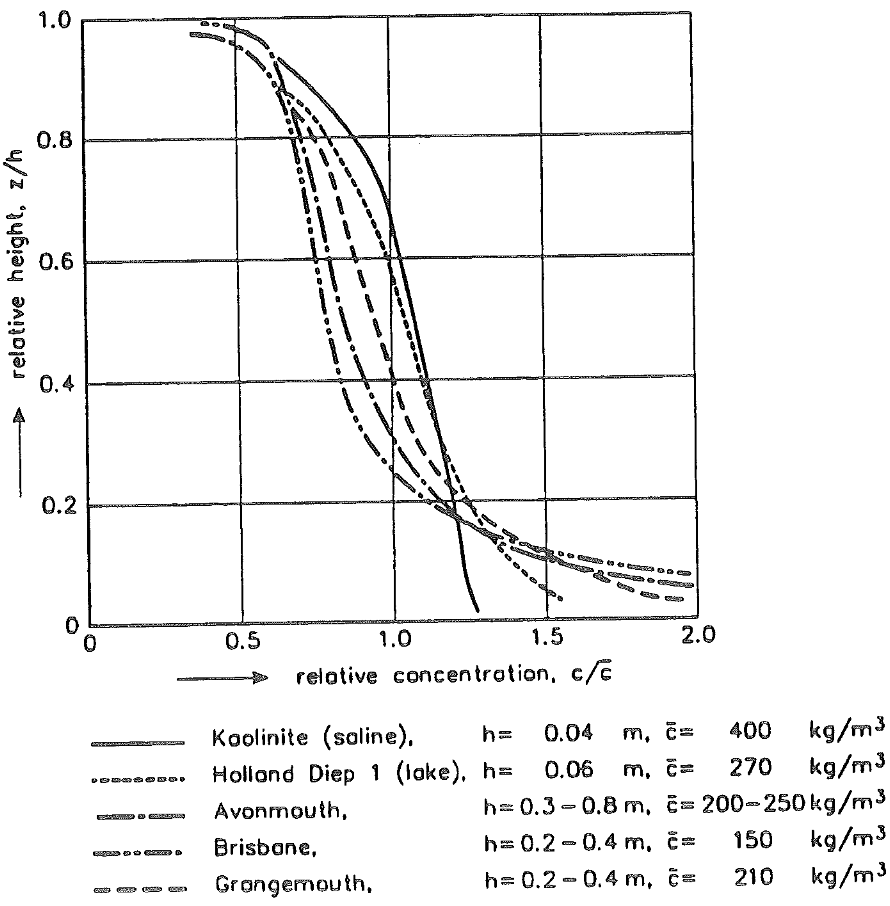


Figure 11.6.5 *Vertical profile of dry sediment density*
Kaolinite and natural muds

Finally, values of the wet and dry sediment densities ($\rho_{wet} = \rho + \rho_{dry} (\rho_s - \rho)/\rho_s$) in successive consolidation stages in estuarine conditions are given in Table 11.5. Various consolidation stages are distinguished. Mud with a wet density of about 1200 kg/m³ is still sailable for ships. The nautical depth in the harbour of Rotterdam is defined at a wet density of 1200 kg/m³. The transition from the fluid to the solid phase occurs at a wet density of about 1300 kg/m³.

Mud type		Sand	Organic	Thickness bed layer	Thickness top layer	Dry sediment density (kg/m³)			
						Top layer		Bottom layer	
		(%)	(%)	(m)	(mm)	1 day	≥ 7 days	1 day	≥ 7 days
kaolinite	(saline)	0	0	0.05	3-5	110-150	150-200	450	500
kaolinite	(distilled)	0	0	0.05	3-5	-	150-200	-	-
Hollands Diep 1	(lake)	9	10	0.05	3-5	100-200	150-250	350	400
Hollands Diep 2	(lake)	23	9	0.05	3-5	100-200	150-250	350	350
Ketelmeer	(lake)	7	12	0.05	2-3	100-150	150-200	450	450
Biesbosch	(lake)	8	8	0.05	3-5	50-100	100-150	350	400
Maas	(river)	36	8	0.05	2-3	100-150	100-150	500	600
Breskens Harbour	(estuary)	27	5	0.05	3-5	100-150	-	650	-
Delfzijl Harbour	(estuary)	60	2	0.05	5-10	100-150	150-200	900	1100
Loswal Noord	(sea)	69	2	0.05	5-10	100-150	150-200	1300	1500
Avonmouth	(estuary)	-	-	0.3-0.8	-	-	80-150	-	350-450
Brisbane	(estuary)	-	-	0.3-0.4	-	-	50-100	-	350-500
Grangemouth	(estuary)	-	-	0.3-0.4	-	-	50-100	-	350-500
Belawan	(estuary)	-	-	0.3-0.4	-	-	50-100	-	350-500

Table 11.4 Densities of consolidated muds

Consolidation stage	Rheological behaviour	Wet sediment density (kg/m³)	Dry sediment density(kg/m³)
freshly consolidated (1 day)	dilute fluid mud	1000-1050	0-100
weakly consolidated (1 week)	fluid mud (Bingham)	1050-1150	100-250
medium consolidated (1 month)	dense fluid mud (Bingham)	1150-1250	250-400
highly consolidated (1 year)	fluid-solid	1250-1350	400-550
stiff mud (10 years)	solid	1350-1400	550-650
hard mud (100 years)	solid	> 1400	> 650

Table 11.5 Density ranges of consolidated mud

11.7 Erosion

11.7.1 Introduction

Sediment particles, flocs or lumps of the bed surface (including fluid mud layers) will be eroded when the applied current-induced or wave-induced bed-shear stress (τ_b) exceeds a critical value for erosion (τ_e), which depends on the bed material characteristics (mineral composition, organic materials, salinity, density etc.) and bed structure. Experimental results

show that the critical bed-shear stress for erosion is strongly dependent on the deposition and consolidation history.
The critical bed-shear stress for erosion was (by many researchers) found to be larger than the critical bed-shear stress for full deposition ($\tau_e > \tau_{d,full}$). Mehta and Partheniades (1975) observed partial deposition (and no erosion) of mud material for (high) bed-shear stresses up to 1.5 N/m² in steady flows, because the bed consisted of strong dense flocs deposited during high-shear conditions in which the weak flocs could not be deposited. Thus, τ_e may be very large when the top layer of the bed consists of strong flocs deposited at relatively high velocities ($\tau_e > \tau_{d,part}$).
In tidal conditions with decreasing velocities (near slack tide) the weak flocs will be deposited on top of the strong flocs deposited earlier at higher velocities. At increasing velocities of the next tidal cycle, these weak flocs can be eroded rather easily at low velocities.
Summarizing, erosion will occur, when:

$$\tau_e > \tau_{d,full} \qquad (11.7.1)$$

$$\tau_e \leq \tau_{d,part} \text{ or } > \tau_{d,part} \qquad \text{depending on deposition history} \qquad (11.7.2)$$

Various types of erosion are distinguished in the literature: 1. particle or floc erosion (surface erosion) which is the one by one removal of particles and/or aggregates, 2. mass erosion which is the erosion of clusters or lumps of aggregates due to failure within the bed. Many attempts have been made to relate the τ_e-values to basic parameters as plasticity index, voids ratio, water content, yield stress and others.
Generally accepted relationships, however, are not available. Determination of the τ_e-value must, therefore, be based on laboratory tests using natural mud or on in-situ field tests. Young and Southard (1978) used an in-situ flume to determine τ_e-values. The observed values were found to be considerably smaller than those for corresponding laboratory flumes using the same material. The difference was ascribed to a change in sediment cohesion caused by natural bioturbation (biologically aggregated sediments) yielding lower densities.

11.7.2 Consolidated hard deposits

All information is based on the analysis of scour data of small natural channels. Lane (1955) proposed the following threshold velocities for erosion (Table 11.6):

Material	Loose	Moderate compacted	Compact
sandy clay	0.45 m/s	0.9 m/s	1.3 m/s
clay	0.35 m/s	0.8 m/s	1.2 m/s
lean clayey soil	0.3 m/s	0.7 m/s	1.0 m/s

Table 11.6 Critical depth-averaged erosion velocities

Hughes (1980) examined the scouring process in small natural channels (at 150 locations) in the southwestern USA by measuring and statistically analyzing velocities associated with initiation of scour. Three types of soils were distinguished: sandy silt, silty clay (firm compacted) and clay (compact). The results are shown in Fig. 11.7.1. The 1% threshold velocity is the maximum velocity which will not give channel erosion in 99 cases out of 100 and for practical purposes is equivalent to the maximum non-eroding velocity.
Comparison with other data showed good agreement when the 1% threshold value was used. Kamphuis (1982) found that a fluid containing sand particles will cause erosion of a consolidated cohesive bed at much lower fluid velocities (or shear stresses) than if the fluid was free of sand. The average erosion rates were of the order of 2.10^{-4} m/hour for velocities in the range of 1 to 2 m/s.

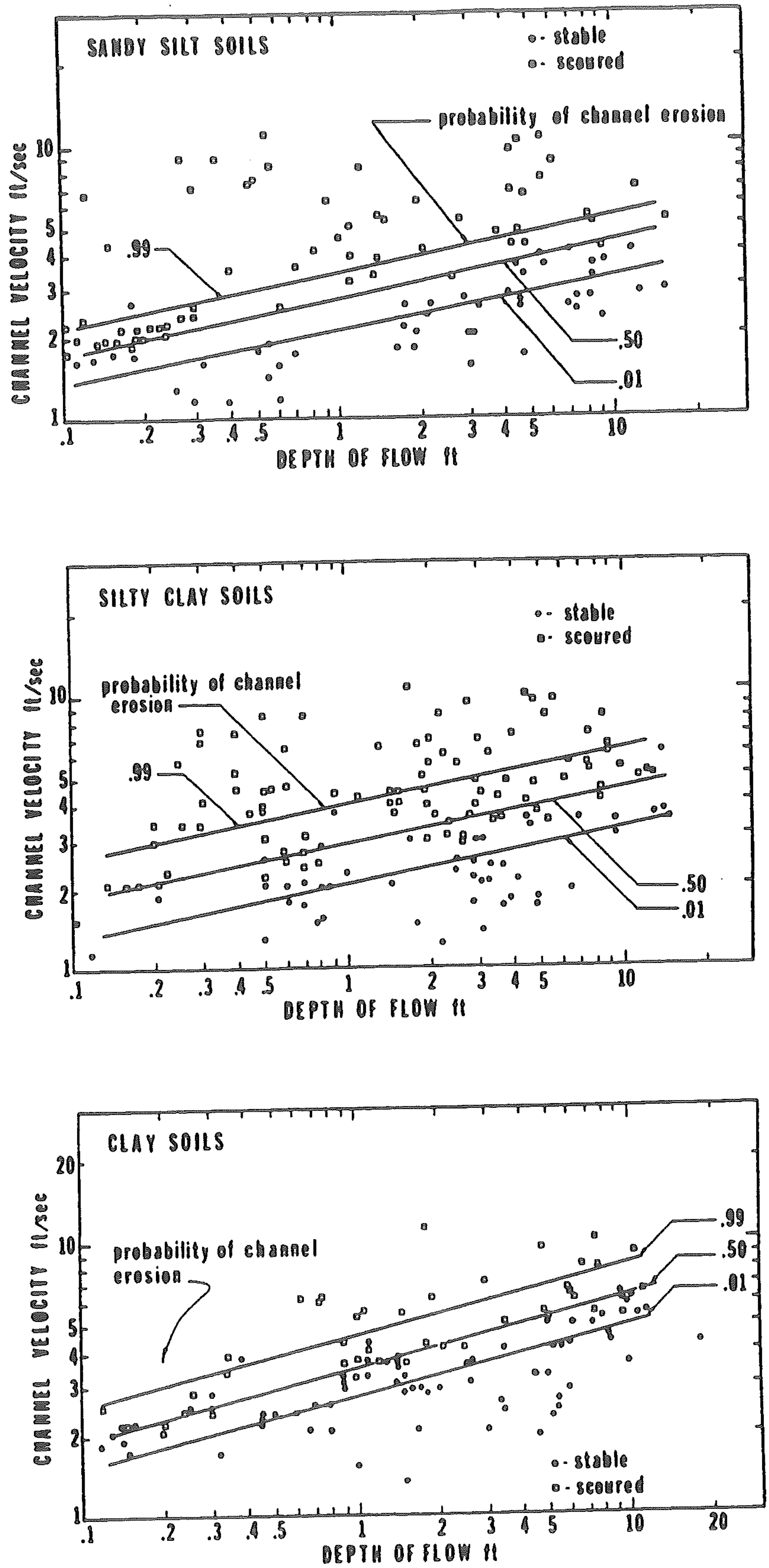


Figure 11.7.1 Critical erosion velocities of consolidated hard deposits, Hughes (1980)

11.7.3 Consolidated soft deposits

Migniot (1968), Cormault (1971), Thorn (1981), Parchure and Mehta (1985), Winterwerp et al (1991) and others have observed that the critical bed-shear stress for erosion (τ_e) is related to the sediment concentration (dry density).

The annular flume experiments of Parchure and Mehta (1985) clearly demonstrate that the τ_e-value increases with depth in the bed layer, because the dry sediment density increases with depth. For a given constant current-related bed-shear stress an equilibrium concentration is established after some time (Fig. 11.7.2). A further increase of the current velocity and hence the bed-shear stress (τ_b) will result (after some time) in a new but larger equilibrium concentration (Fig. 11.7.2).
This sequence can be interpreted as erosion of the top layer down to a level at which the bed strength (τ_e) is equal to the applied bed-shear stress (τ_b). Then, the erosion is stopped. This interpretation is confirmed by the results of a special experiment in which the fluid-sediment suspension above the bed was slowly replaced by clear water showing no further erosion (replaced fluid remained clear). Thus, the establishment of an equilibrium concentration is caused by an erosion stop ($\tau_b = \tau_e$) and not by saturation of the suspension. Parchure and Mehta (1985) determined τ_e-values as a function of the dry sediment density. The tests were performed in an annular flume. Suspension concentrations were measured. Samples were taken from the bed by using a metal corer (after freezing). The samples were divided in segments to determine the dry densities. The thickness of the eroded bed layers at each stage (with constant equilibrium concentration) was determined from the increase of the suspension concentration. Since, the applied current-induced bed-shear stress (τ_b) is always known, it follows that $\tau_e = \tau_b$ when there is no more erosion, while the dry sediment density of the top bed surface can be determined from the observed increase of the suspension concentration (corresponding to an eroded layer thickness). Figure 11.7.2 shows the results for kaolinite in saline water. Other experiments of Parchure and Mehta showed a clear increase of the τ_e-value with increasing salinity (see Fig. 11.7.3). The effects were largest for salinities in the range of 0 to 2‰. Further increase of the salinity only had a marginal effect. A larger consolidation time resulted in a larger τ_e-value. No further increase was observed after a consolidation period of 10 days. The τ_e-value at the top bed surface (z=0) was not dependent on consolidation time.

The results of Winterwerp et al (1991) for natural muds from The Netherlands give information of τ_e-values related to surface erosion (particle or floc erosion) and τ_e-values related to mass erosion (bed failure), see Table 11.7.
Two types of mud had a high percentage of sand particles, which were deposited in the bottom layers. In most tests a marked top layer consisting of mud with a thickness of about 5 mm was observed, which was eroded by surface erosion at bed-shear stresses in the range of about 0.15-0.30 N/m². Thereafter, the lower layers were eroded, first by surface erosion and after that by mass erosion (bed failure) at bed-shear stresses larger than about 0.5 N/m².

The critical bed-shear stress for erosion can be related to the dry sediment density (ρ_{dry} in kg/m³), as follows:

$$\tau_e = \alpha \, (\rho_{dry})^{\beta} \tag{11.7.3}$$

The β-coefficient was found to be in the range from 1 to 2.5. Thorn and Parsons (1980) found $\beta = 2.3$ for mud from the Brisbane river (Australia), Grangemouth estuary (Scotland) and Belawan (Indonesia). Burt (1990) found $\beta = 1.5$ for mud from Cardiff Bay (England). The muds from The Netherlands suggest $\beta = 1.5$ (see Table 11.8).

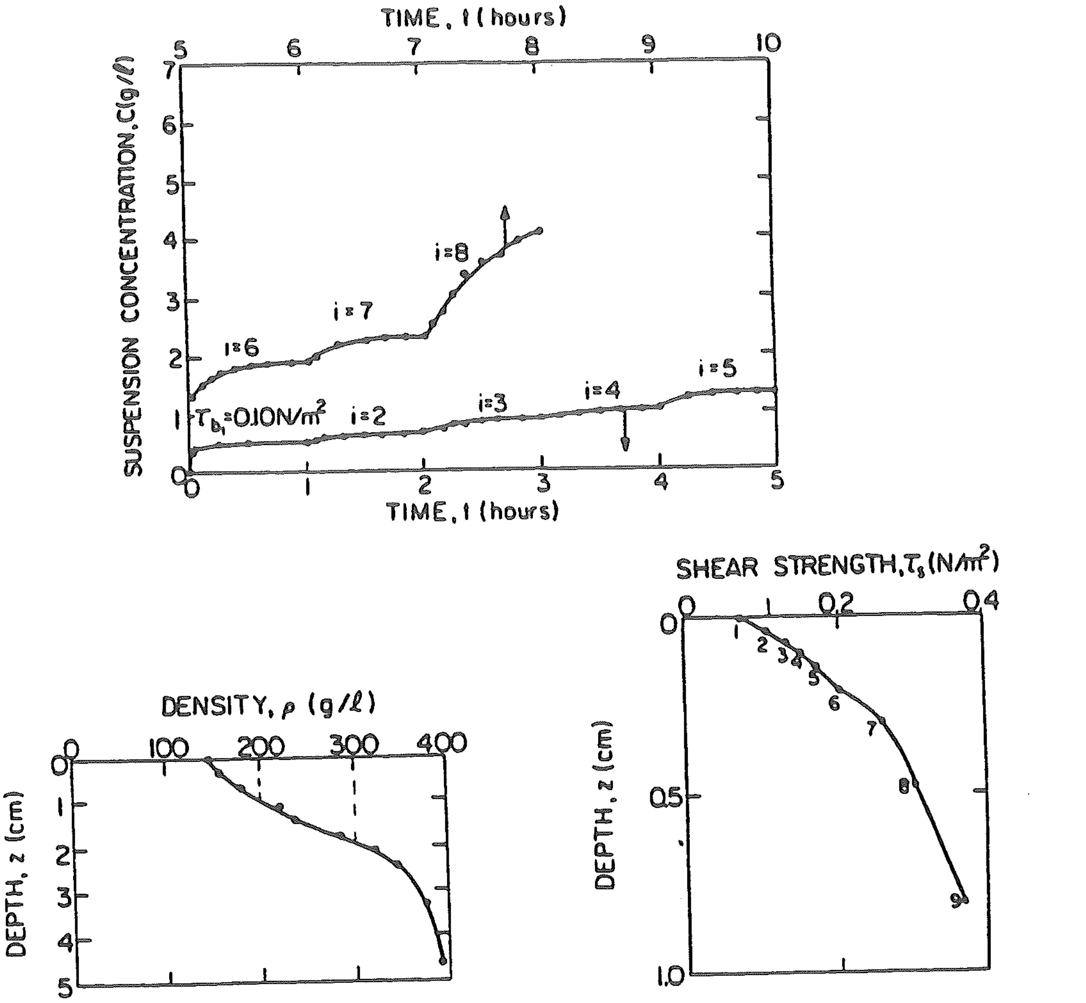


Figure 11.7.2 Erosion tests of kaolinite (saline water) after Parchure and Mehta (1985)

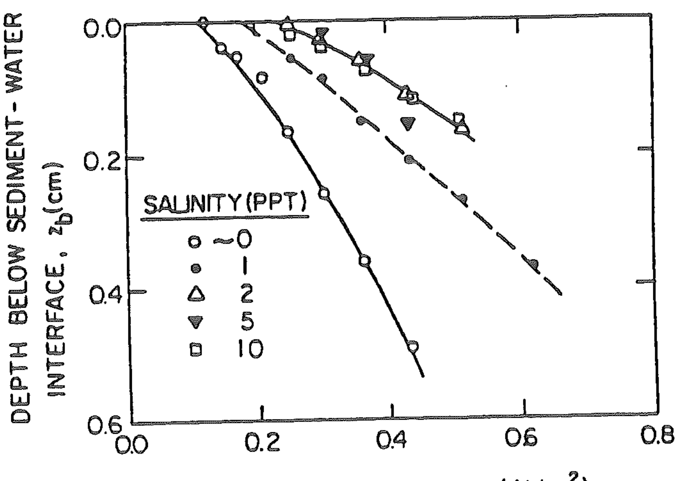


Figure 11.7.3 Influence of salinity on critical bed-shear stress for erosion Lake Francis mud, Parchure and Mehta (1985)

Table 11.8 shows values of the critical bed-shear stresses for kaolinite and natural muds from The Netherlands, Australia, Scotland, Indonesia, France and England.

Migniot (1968) and Otsubo-Maraoko (1988) studied the relationship between the critical bed-shear stress for erosion (τ_e) and the yield stress (τ_y).
Migniot (1968) found $\tau_e = 2.5\ \tau_y^{0.5}$ for $\tau_y < 1.5$ N/m^2 and $\tau_e = 2\ \tau_y$ for $\tau_y > 1.5$ N/m^2.
Otsubo and Muraoko (1988) performed many tests to determine the critical shear stresses of natural muds. Based on their experimental results, the critical shear stress for surface erosion ($\tau_{e,1}$) and mass erosion ($\tau_{e,2}$) was related to the yield stress (τ_y), as follows: $\tau_{e,1} = 0.27\tau_y^{0.56}$ and $\tau_{e,2} = 0.79\tau_y^{0.94}$.
A roto viscometer was used to determine the τ_y-value. The $\tau_{e,1}$-values were in the range of 0.05-0.5 N/m^2, and $\tau_{e,2}$ in the range of 0.1-1 N/m^2.

Mud type		Sand	Organic	τ_e = critical bed-shear stress erosion (N/m^2)					
				Surface erosion of top layer		Erosion of lower layers		Mass erosion of lower layers (bed failure)	
		(%)	(%)	1 day	7 days	1 day	≥ 7 days	1 day	≥ 7 days
kaolinite	(saline water)	0	0	0.08	0.08	0.15	0.15	0.25	0.25
Hollands Diep 1	(lake)	9	10	0.20	0.35	0.45	0.55	0.65	≥ 0.65
Hollands Diep 2	(lake)	23	9	0.20	0.30	0.45	0.50	0.75	0.75
Ketelmeer	(lake)	7	12	0.15	0.20	0.30	0.35	0.55	≥ 0.55
Biesbosch	(lake)	8	8	0.15	0.25	0.30	0.40	0.60	0.60
Maas	(river)	36	8	0.20	0.35	0.45	0.55	≥ 0.55	≥ 0.65
Breskens Harbour	(estuary)	27	5	0.20	-	0.40	-	≥ 0.60	-
Delfzijl Harbour	(estuary)	60	2	0.10	0.20	0.20	0.40	0.40	≥ 0.50
Loswal Noord	(sea)	69	2	0.25	0.30	0.40	0.45	0.60	0.60

Table 11.7 Critical bed-shear stress for surface and mass erosion for consolidation periods of 1 and 7 days

Mud type		Sand	Organic	τ_e = critical bed-shear stress erosion (N/m^2)				
		(%)	(%)	c = 100 kg/m^3	c = 150 kg/m^3	c = 200 kg/m^3	c = 250 kg/m^3	c = 300 kg/m^3
kaolinite	(saline water)	0	0	-	0.05-0.10	0.30-0.40	-	-
kaolinite	(distilled water)	0	0	-	0.05-0.10	0.15-0.20	0.20-0.25	0.25-0.30
Hollands Diep 1	(lake)	9	10	0.15-0.25	0.30-0.40	0.40-0.50	0.60-0.80	-
Hollands Diep 2	(lake)	23	9	0.15-0.25	0.30-0.40	0.40-0.50	0.80-1.00	-
Ketelmeer	(lake)	7	12	0.10-0.20	0.20-0.25	0.25-0.35	0.50-0.70	-
Biesbosch	(lake)	8	8	0.20-0.25	0.25-0.30	0.30-0.35	0.50-0.70	-
Maas	(river)	36	8	0.15-0.30	0.30-0.40	0.40-0.50	0.80-1.00	-
Breskens Harbour	(estuary)	27	5	0.15-0.25	0.25-0.35	0.35-0.45	0.60-0.80	-
Delfzijl Harbour	(estuary)	60	2	0.05-0.15	0.15-0.20	0.20-0.25	0.40-0.60	-
Loswal Noord	(sea)	69	2	0.20-0.30	0.30-0.35	0.35-0.45	0.60-0.80	-
Brisbane, Grangemouth, Belawan		0	-	0.20.0.30	0.40-0.60	0.80-1.00	-	-
Loire		-	-	0.10-0.15	0.15-0.20	0.20-0.30	0.30-0.40	0.80-1.20
Cardiff Bay		-	-	0.20-0.30	0.40-0.50	0.60-0.70	0.70-0.90	-

Table 11.8 Critical bed-shear stress for erosion for different sediment concentrations (dry density)

Finally, the influence of the various physical-chemical parameters is summarized. Experimental results (see Winterwerp, 1989) indicate that the critical bed-shear stress τ_e:

- decreases for increasing temperature (weaker bonds),
- decreases for increasing pH-values (weaker bonds),
- decreases for increasing sand concentration (weaker bonds),
- increases for increasing clay concentration (stronger bonds),
- increases for increasing organic content (stronger bonds),
- increases for increasing salinity (stronger bonds),
- increases for increasing SAR (stronger bonds),
- increases for increasing CEC (stronger bonds).

11.7.4 Erosion rates

The erosion rates for a bed of constant density (τ_e = constant) is, generally, expressed as (Partheniades, 1963):

$$E = \frac{dm}{dt} = M\left[\frac{\tau_b - \tau_e}{\tau_e}\right] \qquad \text{for } \tau_b > \tau_e \qquad (11.7.4)$$

The M-coefficient (in mass per unit area and time) is a material "constant" depending on mineral composition, organic material, salinity etc. Reported values are in the range of M = 0.00001-0.0005 kg/sm^2 for soft natural muds.
Parchure and Mehta (1985) found for a bed of increasing density (τ_e = not constant) a relationship of the form:

$$E = E_o \exp\left[\alpha(\tau_b - \tau_e)^{0.5}\right] \qquad \text{for } \tau_b > \tau_e(z) \qquad (11.7.5)$$

The E_o-value (in kg/sm^2) is defined as the value for $\tau_b = \tau_e$ (at the surface z = 0) and can be determined by extrapolation from a plot of E against $\tau_b - \tau_e$. The E_o-values are in the range of 0.00001 to 0.001 (kg/sm^2) for kaolinite and natural mud in saline water. The α-coefficient is in the range of 5 to 30 (m/N$^{0.5}$).

11.7.5 Bed forms and roughness

When the critical shear stress for erosion is exceeded ($\tau_b > \tau_e$), various types of bed forms may be generated, as follows:

- longitudinal grooves and ridges with a typical spacing in the range of 0.01 m to 1 m, which are probably generated by spiral fluid motions. Meandering grooves have also been observed.
- flute marks which are spoon-shaped depressions in the bed surface probably generated by the highest velocities of turbulent eddies.
- small-scale isolated ripples with a length of 0.1 m and a height of 0.005 m have been observed in flumes (velocity = 0.3 m/s, τ_b = 0.2 N/m^2).

The micro topography of a muddy bed in low-energy regions may also be controlled by biological activity.
Although locally, small-scale bed forms are generated, the flow over a cohesive bed surface generally is found to be hydraulically smooth with relatively large Chézy-coefficients in the range of 60 to 100 m$^{0.5}$/s. The effective bed roughness height is in the range of 0.1 to 1 mm.

11.8 Transport of mud by currents

11.8.1 (Quasi) steady flow

1. Wash load

The wash load of a river consists of the finest-grained portion of the total sediment load. The wash load shows an almost uniform concentration profile (small vertical gradients). The wash load typically is controlled by the upstream supply rate and not by the local river bed characteristics. Generally, the wash load bears no relationship to the discharge of the river. Concentrations tend to be large when there is a ready source of fine sediments in the drainage basin (soil erosion). Extremely large concentrations (hyperconcentrations up to 50% sediment by weight in the fluid) are a common occurrence during flash floods in semi-arid environments. Generally, this may occur in small rivers. The Yellow river is a dramatic exception with hyperconcentrations in the range of 500 to 1000 kg/m³ due to presence of extensive loess deposits in the upper river basin. About 10% of this material consists of clay (< 5 μm). The remaining material consists of silt and fine sand (25 to 50 μm), (Ning et al, 1988). Hyperconcentrations may enhance the total bed material transport because the suspended sand particles have a reduced settling velocity and remain longer in suspension before they are deposited again on the bed.

2. Density currents in reservoir

A density current is defined as the movement of a fluid-sediment mixture by gravity under, through or over another mixture fluid with a lower density. Density currents consisting of a high fraction of fine silt and clay material have been observed in reservoirs built in heavy silt-laden rivers in China (Jiahua, 1986, 1991) and the USA.

A density current will be generated at the upstream end of a reservoir when the internal density-related Froude number exceeds 0.8 (Jiahua, 1986). Thus:

$$\frac{\bar{u}_o}{\left(\left(\frac{\Delta\rho}{\rho}\right) g\ h_o\right)^{0.5}} \geq 0.8 \qquad (11.8.1)$$

in which:

$\bar{u}_o$ = mean velocity at initial location
h_o = depth at initial location
$\Delta\rho$ = density difference
ρ = density of incoming fluid-sediment mixture
$\Delta\rho$ = $\rho - \rho_o$ = density difference
ρ_o = density of surrounding fluid (clear water)

Equation (11.8.1) is valid for concentrations of the incoming mixture smaller than 100 kg/m³.

An indication for the generation of a density current at the upstream end of a reservoir is the plunging of the muddy river flow into the deeper reservoir. At the plunge point reversed flow and vortices have been observed at the water surface. Figure 11.8.1 shows the behaviour of a density current during the period of 16 to 18 August 1961 in the Sanmenxia Reservoir in China. Velocity and concentration profiles were measured along the reservoir. The density current had a thickness of approximately 3 to 5 m. A clear interface can be observed. Velocities inside the density current were in the range of 0.4 to 0.6 m/s. The density current traveled over the full length of the reservoir. Part of the density current material was deposited on the bed.

A density current can be reflected (partly) against an obstacle resulting in an increase of the layer thickness and a decrease of the velocity. This will enhance deposition.

The generation of a density current into a reservoir usually is a short period phenomenon (days), because the incoming flood discharge in arid regions increases and decreases abruptly. During a flood the silt and clay concentration (due to land surface erosion) increases enormously.

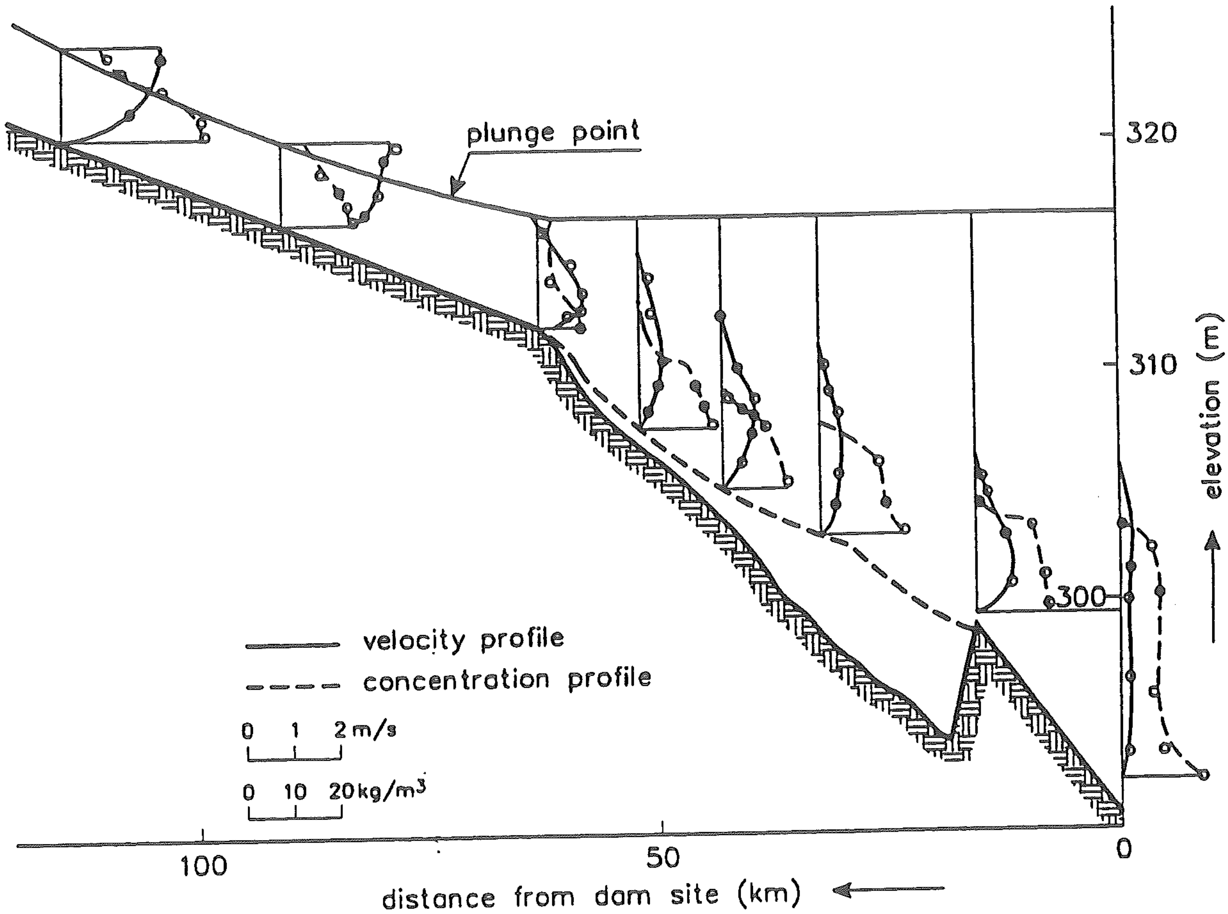


Figure 11.8.1 Density current in Sanmenxia Reservoir, China

11.8.2 Non-steady (tidal) flow

Field observations in tidal flow show the presence of a cyclic process of erosion, transport, settling, deposition and consolidation.

Generally, a three-layer system can be distinguished in vertical direction (see Fig. 11.8.2), as follows:

- consolidated mud layer at the bottom with concentrations larger than about 300 kg/m^3. The flocs and particles are supported by the internal floc framework.
  The mud interface is detectable by echosounding instruments (30 kHz).
- fluid mud suspension layer with concentrations in the range of 10 to 300 kg/m^3. The layer thickness is of the order of 0.1 to 1 m. The flocs and particles are supported by fluid drag forces exerted by the escaping fluid (hindered settling effect). A clear interface (lutocline) can be observed from echosounder recordings or from nuclear density recordings. Transport of fluid mud may take place by tide-induced and wave-induced forces and gravity forces (sloping bottom). Figure 11.8.3 shows a fluid mud layer in the Banjarmasin Channel (Kalimantan, Indonesia).

- dilute mud suspension with concentrations in the range of 0 to 10 kg/m³, which are detectable by optical methods and mechanical sampling (see Fig. 11.8.2). Flocculation is dominant. The flocs and particles are supported by turbulence-induced fluid forces and transported by tide-driven and wind-driven currents.

Vertical layers of different densities are influenced by gravity processes which oppose the mixing processes. The stability of the system can be characterized by the gradient Richardson number (R_i). For R_i larger than unity (based on experimental data) a stable system will be present (interfacial instabilities will die out). Winterwerp et al (1991) found R_i-values upto 0.5 for the fluid mud layer in an annular flume experiment expressing the presence of interfacial instabilities (generation of internal waves).

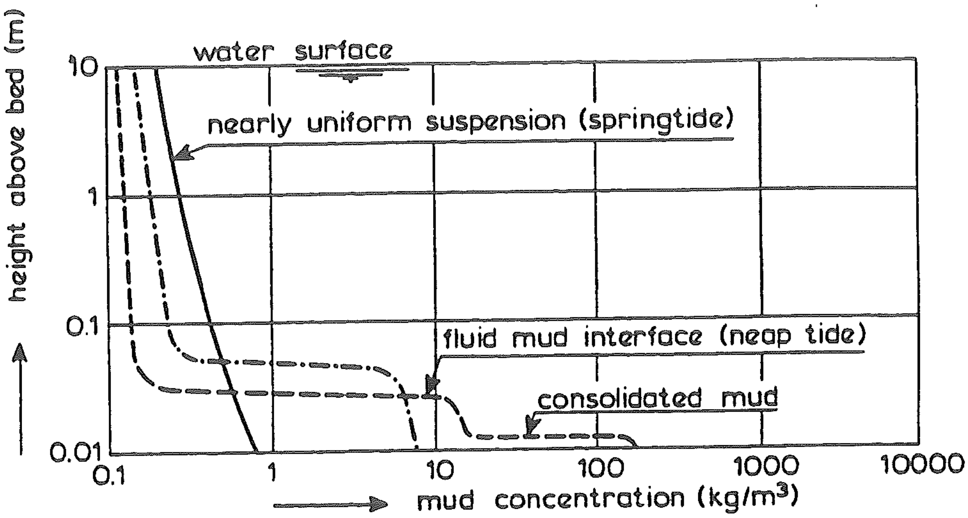


Figure 11.8.2 Vertical distribution of mud concentrations

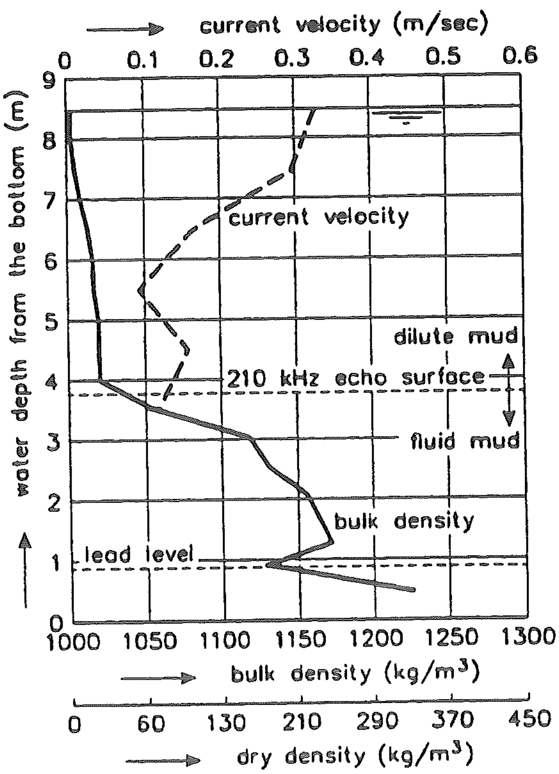


Figure 11.8.3 Fluid mud layer, Kalimantan, Indonesia

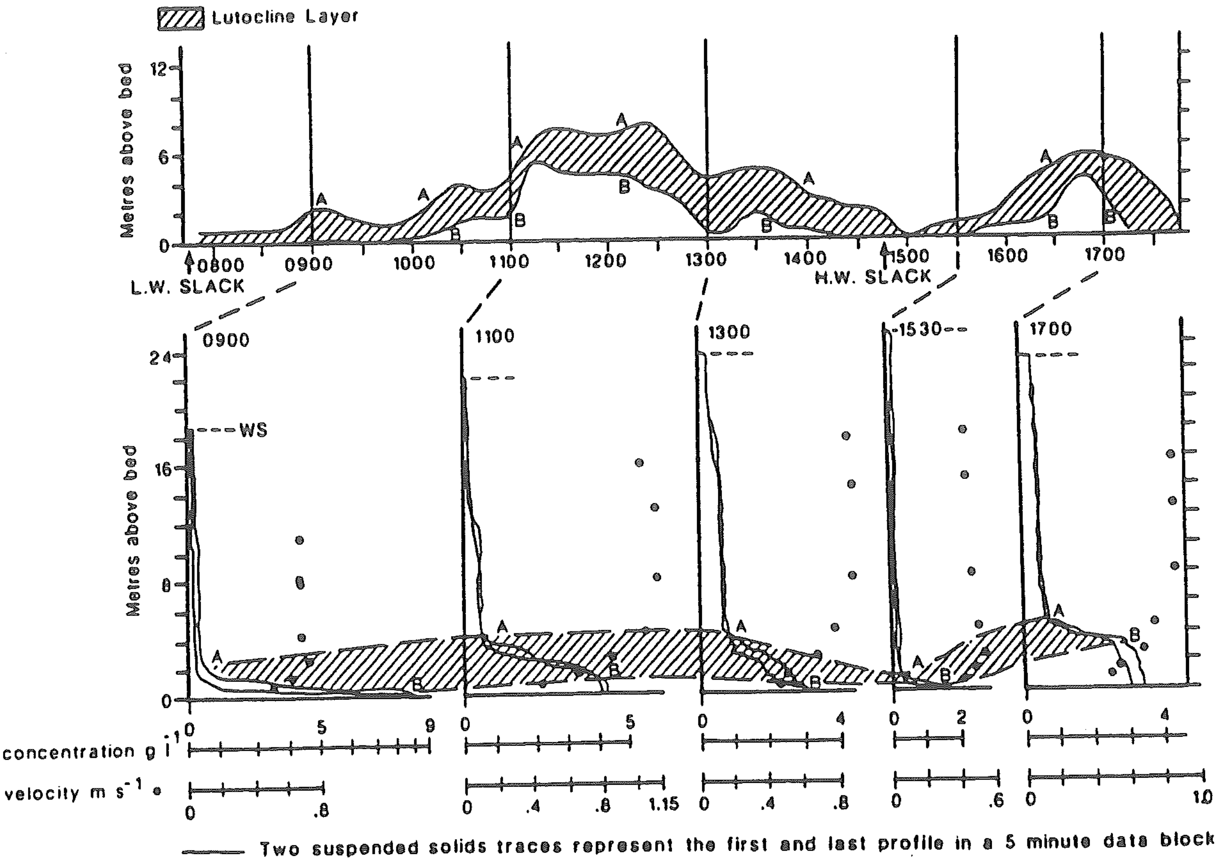


Figure 11.8.4 Lutocline layer, Severn estuary, England (Kirby and Parker, 1980, 1983)

Erosion at the consolidated mud interface or at the fluid mud interface will occur in accelerating flow when the applied bed-shear stress exceeds the critical value for erosion ($\tau_b > \tau_e$), see Fig. 11.8.5. Similarly, deposition will take place in decelerating flow when the applied bed-shear stress falls below the critical value for deposition ($\tau_b < \tau_{d,part}$). Deposition is maximum around slack water because the floc destruction due to turbulent shear stresses is minimum and floc growth due to differential settling is maximum (large flocs "fall" on small flocs yielding new larger flocs). As a result of these processes the mud concentrations vary in time and space, see Fig. 11.8.5. The concentrations decrease near the slack water period (deposition) and increase towards maximum flow (erosion). Maximum concentrations generally occur at a certain (lag) period after maximum flow because it takes time to transport the particles or flocs to the upper layers, see Fig. 11.8.5. The lag period is relatively large near the water surface and relatively small near the bottom. The small settling velocities of the individual particles prevent settling of all particles during slack water. Thus, the concentration remains always larger than zero (background concentration). Figure 11.8.6 shows (natural) mud concentrations as a function of bed-shear stress for a tidal flow experiment in a circular flume (Winterwerp et al, 1991). The tidal flow introduces a lag effect (hysteresis effect).

Figure 11.8.4 shows typical velocity and mud concentration profiles from low water slack to high water slack in the Severn estuary, England (mean spring range = 12.3 m, mean neap range = 6.5 m, maximum spring flood velocity = 2 m/s). The mobile suspension system consists of three layers. The transition (middle) layer is called the lutocline layer. As the velocity increases towards maximum flow, the mud is mixed upwards by turbulence and the lutocline layer moves upwards. As the velocity decreases towards slack water, the sediments begin to settle and the lutocline layer moves downwards.

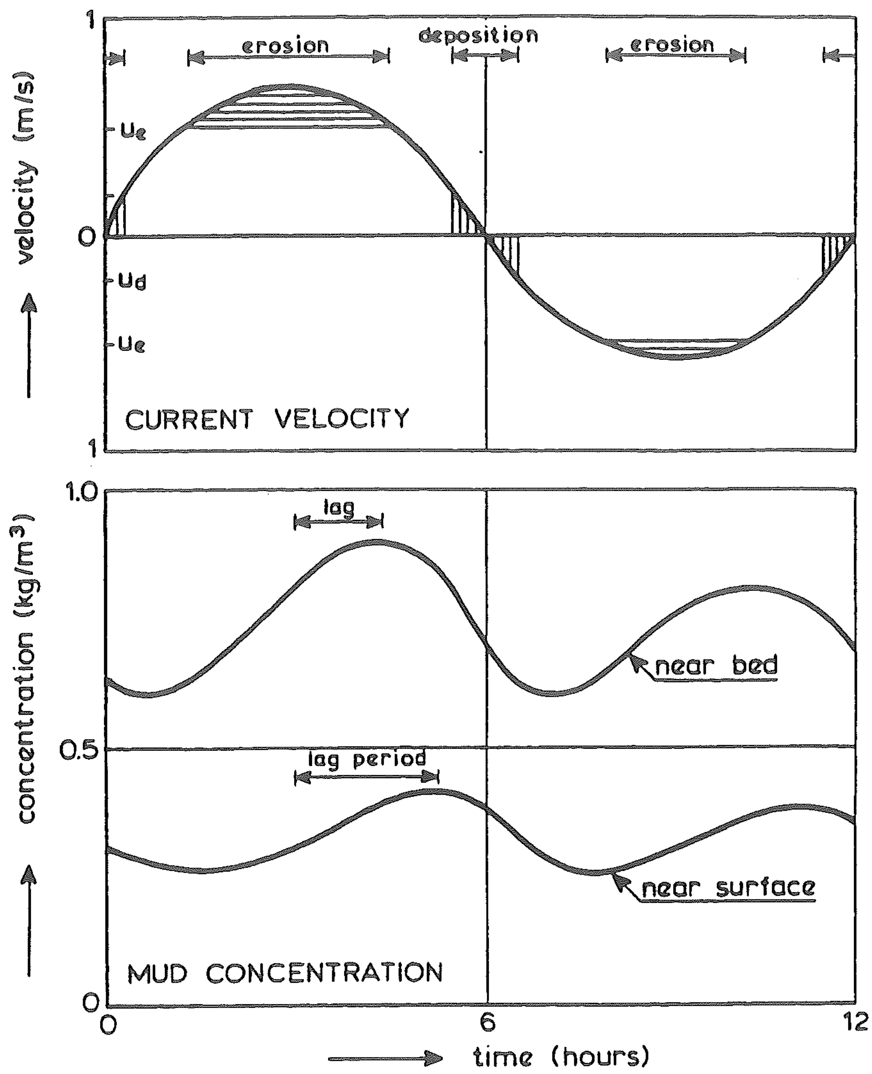


Figure 11.8.5 Variation of mud concentration in tidal flow

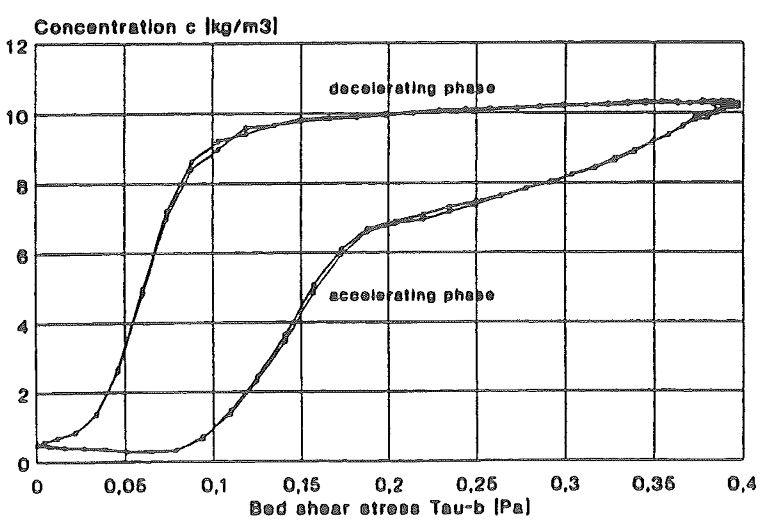


Figure 11.8.6 Variation of (natural) mud concentrations as a function of bed-shear stress in tidal flow (Winterwerp et al, 1991)

As the tidal energy decays towards neap tides, there is insufficient turbulence available resulting in a strong reduction of the thickness of the lutocline layer. Deposition and consolidation prevails and the fluid mud layer remains stable over several tidal periods.

Thus, the spring-neap cycle may have a marked influence on the vertical structure of the mud suspension:

springtide: nearly uniform (well-mixed) concentration distribution at maximum flood and ebb velocities, formation of fluid mud interfaces (lutoclines) near the bottom during slack water, lutocline is moving downward, re-entrainment of mud during the next tide,

neaptide : tidal velocities are generally too small to cause erosion at the near-bed fluid mud layer, the fluid mud layer may survive several tidal cycles as a stationary layer.

Mathematically, the transport of cohesive sediments in a well-mixed estuary (no vertical salinity stratification) can be described by the convection-diffusion equation. Since the vertical concentration distribution above the fluid mud layer is nearly uniform, the convection-diffusion equation can be averaged over the depth, yielding:

$$\frac{\partial \bar{c}}{\partial t} + \bar{u}\frac{\partial \bar{c}}{\partial x} + \bar{v}\frac{\partial \bar{c}}{\partial y} - \frac{1}{h}\frac{\partial}{\partial x}\left[hK_x \frac{\partial \bar{c}}{\partial x}\right] - \frac{1}{h}\frac{\partial}{\partial y}\left[hK_y \frac{\partial \bar{c}}{\partial y}\right] - \frac{S}{h} = 0 \qquad (11.8.2)$$

in which:

$\bar{c}$ = depth-averaged concentration

$\bar{u},\bar{v}$ = depth-averaged current velocities in x, y directions

K_x, K_y = dispersion coefficient in x, y directions

h = water depth

S = source-sink term representing erosion and deposition depending on the hydraulic conditions, $S = E$ for $\tau_b > \tau_e$ and $S = D$ for $\tau_b < \tau_d$ and $S = 0$ for $\tau_d \le \tau_b \le \tau_e$.

The boundary conditions, erosion and deposition functions, are given in sections 11.5 and 11.7.

The bed layer should be represented as a number of sublayers, each with its own thickness, density, shear strength and consolidation stage. When deposition occurs, the thickness of the bed layer increases. When erosion occurs, the bed layer thickness decreases. Consolidation causes a continuous decrease of the bed-layer thickness.

In (partially) stratified estuaries the maximum silt and mud concentrations (turbidity maximum) are usually found in the area where the salt wedge is migrating during the tidal cycle. Figure 11.8.7 schematically shows the ebb tide flow of water and mud. Heavy sedimentation will occur in the salt wedge area resulting in the formation of soft mud layers (fluid mud) on the channel bottom.

During the ebb tide the river flow erodes the landward end of the mud layer. Near the toe of the wedge the fresh river water is lifted from the bottom and flows seaward over the heavy saline water. Intensive mixing will occur at the interface between the fresh and saline water. In the salt wedge the bottom current is landward. This saline water meets the fresh water near the toe of the wedge where it is carried upwards.

The mud carried by the river enters the area of the salt wedge together with the mud eroded from the landward end of the mud layer. This mud is mixed (near the interface) with mud already suspended in the saline water resulting in additional flocculation and increased settling velocities. Settling will occur seaward of the sharp salt wedge interface where the mixing is reduced. The mud particles fall towards the bottom where they are transported landward again by relatively strong bottom currents of saline water (see Fig. 11.8.7). Summarizing, there are

relatively strong landward bottom-currents (Fig. 11.8.8) with high mud concentrations during flood tide and relatively weak seaward bottom-currents (Fig. 11.8.8) with low mud concentrations during ebb tide resulting in a net transport of mud in landward direction.
Mathematically, the transport of silt and mud in a stratified estuary can be described by the time-dependent three-dimensional convection-diffusion equation. The erosion of bed material is represented by an erosion function specifying the erosion rate at the bed. The settling effect is represented by the settling velocity.

The convection-diffusion equation reads, as:

$$\frac{\partial c}{\partial t}+\frac{\partial}{\partial x}(uc)+\frac{\partial}{\partial y}(vc)+\frac{\partial}{\partial z}(w-w_s)c-\frac{\partial}{\partial x}(\epsilon_{s,x}\frac{\partial c}{\partial x})-\frac{\partial}{\partial y}(\epsilon_{s,y}\frac{\partial c}{\partial y})-\frac{\partial}{\partial z}(\epsilon_{s,z}\frac{\partial c}{\partial z}) = 0 \qquad (11.8.3)$$

in which:

c	= mud concentration
u,v,w	= flow velocities in x,y,z directions
w_s	= settling velocity
$\epsilon_{s,x}$, $\epsilon_{s,y}$, $\epsilon_{s,z}$	= sediment mixing coefficients in x,y,z directions

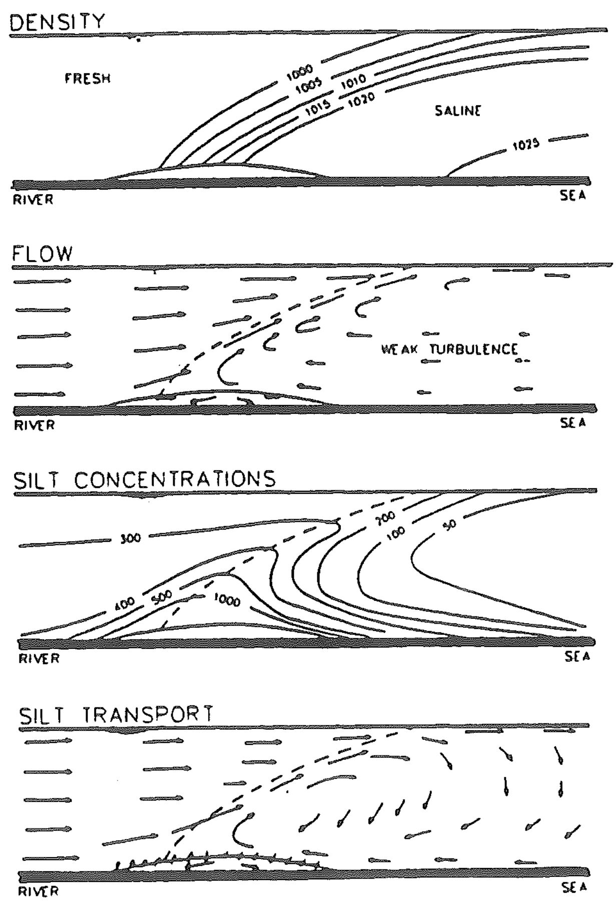


Figure 11.8.7 Flow of water and silt/mud near the salt wedge in a stratified estuary

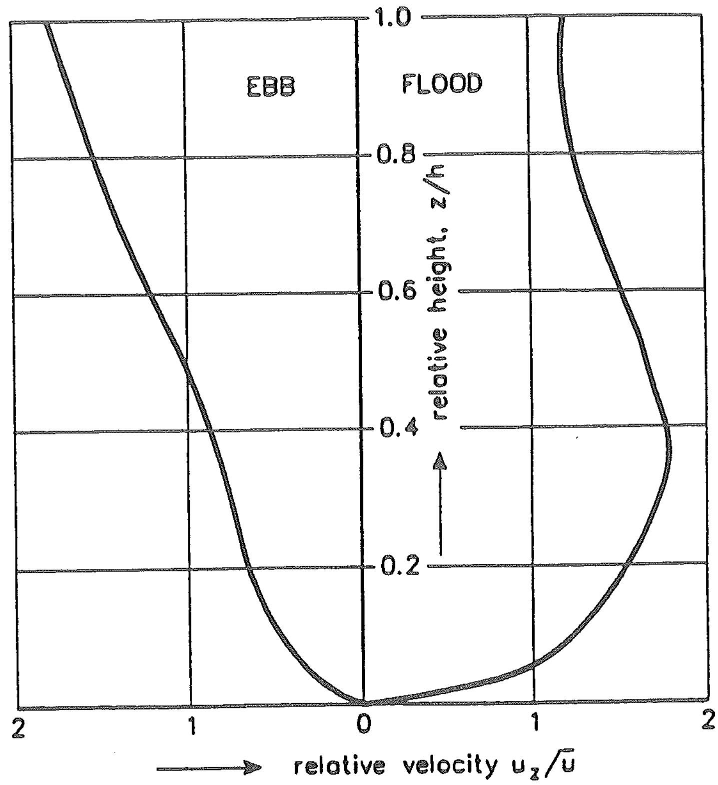


Figure 11.8.8 *Characteristic velocity profiles during ebb and flood tide in case of a horizontal landward salinity (density) gradient*

11.9 Transport of mud by waves

Field observations have shown a significant increase of the mud concentrations near the bed during and after periods with large waves (Van Rijn and Louisse, 1987; Kendrick and Clifford, 1981). Flume experiments with waves over a consolidated mud bed have shown that large waves can easily fluidize the top layer of the mud bed and generate a thin fluid mud layer with concentrations larger than 100 kg/m³ (Van Rijn and Louisse, 1987; Maa and Mehta, 1986). Fluidization of the top mud layer is initiated by wave-induced pressure variations at the bed, which lead to an increase of the water pressure in the pores and hence to a reduction of the internal soil shear strength. Various types of sediment mixtures in saline water were tested by Van Rijn and Louisse (1985, 1987). In four experiments the bed consisted of pure (100%) kaolinite with different densities. In two other experiments the bed consisted of a mixture of fine sand (= 100 μm) and kaolinite. Each experiment consisted of executing a sequence of tests with increasing wave heights. At each constant wave height, an equilibrium concentration was established after a few hours.

Optically and mechanically-collected samples were used to measure the sediment concentrations. The results of test T4 with a bed of kaolinite (ρ_{dry} = 640 kg/m³), a wave period of 1.5 s and a water depth of 0.25 m are shown in Fig. 11.9.1. The following features were observed:

wave height = 0.022 m :	general sediment movement, generation of a thin suspension layer (= 0.01 m) with concentrations of about 0.1 kg/m³.
wave height = 0.03 m :	suspension layer with a thickness of 0.05 m and concentrations upto 0.5 kg/m³ (Fig. 11.9.1A), a vague interface was visible.

wave height = 0.047 m : initially, the concentrations were reduced due to orbital mixing with clear water from higher levels, after 30 minutes the concentrations increased rapidly. Equilibrium concentrations ranging from 2.5 kg/m^3 near the bed to 0.5 kg/m^3 near the surface were established after 200 minutes (Fig. 11.9.1B). After this period, the wave generator was stopped and the suspension was allowed to settle and consolidate for 43 hours resulting in a clear fluid with locally small scour holes (depth = 0.02 m) on the bed.

wave height = 0.07 m : generation of a fluid mud layer (= 0.025 m) with concentrations of 100 kg/m^3, the concentrations above the fluid mud layer ranged from 20 kg/m^3 near the bed to 4 kg/m^3 near the surface (Fig. 11.9.1C). Dye injections showed that the fluid mud layer was slowly (= 0.02 m/s) moving in the wave direction by wave induced velocities (drift). The generation of a fluid mud layer has a stabilizing effect on the erosion process because velocity gradients and hence bed-shear stresses remain relatively small.

Experiments with mixtures of fine sand (= 100 μm) and kaolinite showed that a consolidated bed of 75% kaolinite and 25% fine sand had a similar erosional behaviour as a pure (100%) kaolinite bed. A bed consisting of 25% kaolinite and 75% fine sand had a completely different behaviour. Fluidization of the top layer did not occur. The kaolinite concentrations were not larger than about 0.3 kg/m^3, because only the top layer of the bed was washed out. The sand concentrations were also quite small (factor 30 smaller than in case of 100% sand bed) due to a strong suppression of sand ripples (height = 0.002 m).

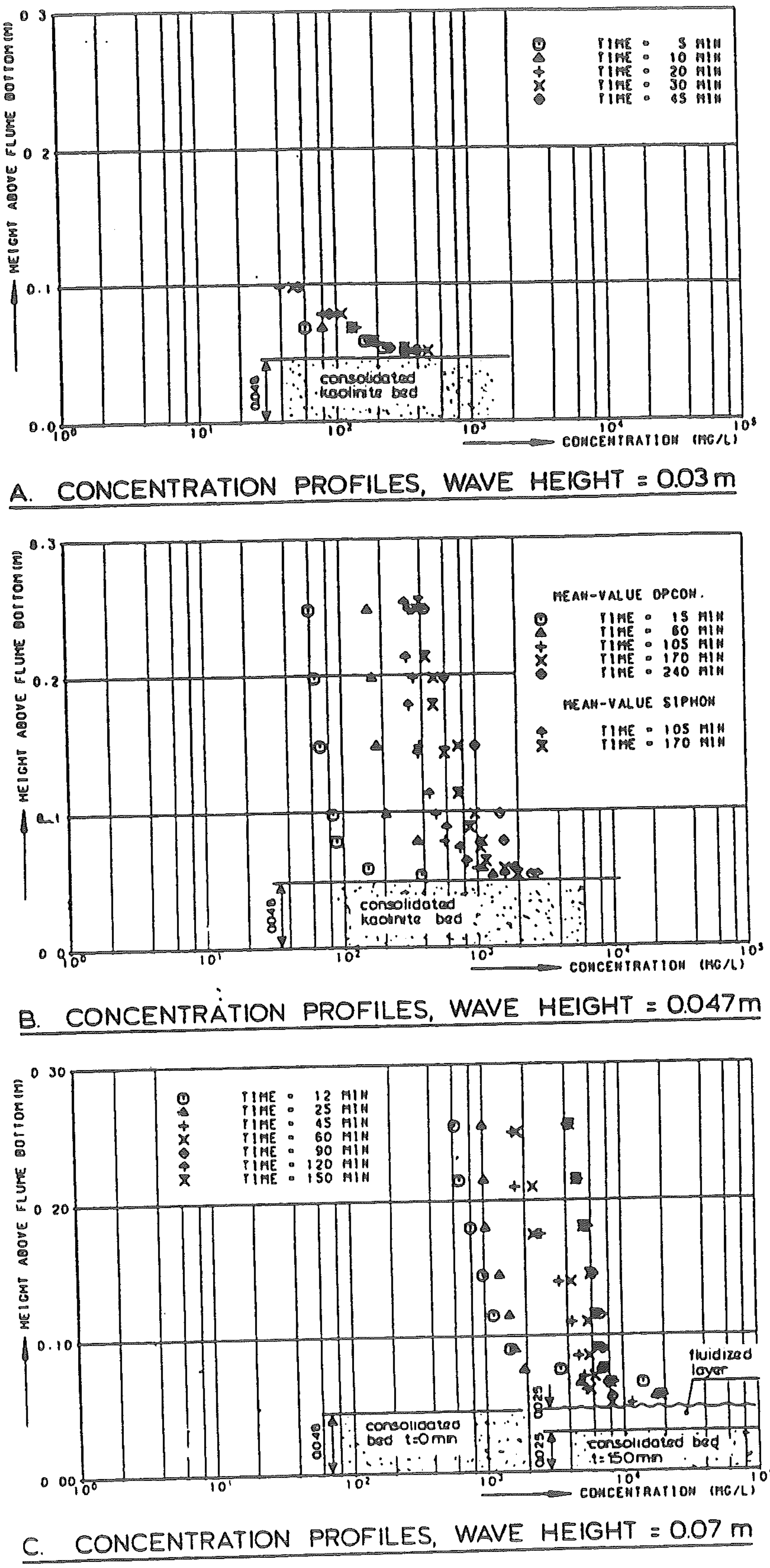


Figure 11.9.1 *Concentration profiles in waves Kaolinite bed, Van Rijn (1985)*

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5 *User functions*

5.1 *Interests and activities*

Description of most important economic and social activities in Indonesian coastal zones which can possibly be of importance for coastal engineering, e.g. agriculture, fishery, housing, transport, mining etc.

5.2 *Interaction with the natural system*

Consequences of activities for the coast, e.g. land reclamation, coral mining, dredging etc.

5.3 *Demand for infrastructure*

From the activities and/or the interaction with the natural system, a demand for infrastructure can be the case, e.g. coastal protection, harbour moles etc.

6 Infrastructure

6.1 Coastal defence structures

An overview of possible measures against coastal erosion.

6.2 Land reclamation

Techniques of land reclamation.

6.3 Harbours

An overview of relevant harbour structures, from a coastal engineering point of view (dredged channels, breakwaters etc., no design of harbour lay-out from a logistics point of view)

7 DESIGN PROCESS

7.1 Introduction

7.1.1 Phases in a coastal project

The realization process of a coastal engineering project (this can be a structure along the coast, but also coastline management project) should proceed in stages. These form a sequence in which the intensity of the investigations and the degree of detail of the plans should increase progressively whilst the overall level of generalization decreases. Increasingly more refined and optimized plans should be made through a process of successive approximations. The following phases can be distinguished in the life cycle of a project:

1. Programme Estimating
2. Programme Identification (Reconnaissance or Pre-feasibility)
3. Programme Preparation (Feasibility)
4. Final or Detailed Engineering Design
5. Preparation of Tender Documents and Tendering
6. Construction or Implementation
7. Operation and Maintenance
8. Monitoring and Evaluation
9. Extension, Upgrading, Improvement, etc.
10. Abandon or demolishing of the works.

The major and most important part of the design takes place in stage (2), (3) and (4). During stage (2) and (3) the concept of the project should be decided and elaborated upon, the selected concept should be presented for communication with the Principal, authorities, etc. Design during these two stages are at conceptual level. During stage (4), detailed engineering design of the project concept which was decided upon in stage (2) and (3) are carried out to establish the final dimensions of the project components.

7.1.2 The design process itself

Design is a creative process, and all new creations of the mind are the results of trial and error. Therefore, one of the main characteristic of the design process is that "It contains feedback". It is a <u>Cyclic</u> or <u>Iterative</u> process. It contains continuous evaluation and it is common for decision-making to involve repeated trials or iterations. Example:

Hypothesis
Testing against previous defined criteria
Adjust criteria
New hypothesis etc.

The need to go back and try again **<u>should not be considered as a personal failure or weakness!</u>** The design engineer will have to acquire a high tolerance for failure and the determination to work the problem out one way or another. Further, the cyclic nature of design provides an opportunity to improve the design based on the preceding outcome. This may lead to the search for the best possible technical solution, e.g. maximum performance at minimum cost.

Many optimization techniques have been developed lately. Although these methods are intellectually pleasing and technically interesting, they often have limited application in a complex situation, in which there are too many variables to include them in the optimization. For example, how to include non-technical considerations like available time, legal or social constraints? In the majority of these cases trade-offs must be made so that the parameters ultimately selected for design are not the best for each individual constraint, but the best for the combination of all constraints. The optimum is the best that can be achieved within the total constraints of the situation or system.
Generally, any Design Process (this is step 4 in the life-cycle of a project, se previous section) consists of the following steps:

a. Need Assessment
b. Problem Definition or Formulation
c. Data and Information Collection
d. Development of Alternatives & Conceptual Design
e. Evaluation
f. Communicate the Design.

7.2 *Problem Analysis and identification*

7.2.1 *Need assessment*

In many cases the design process starts with very broad requirements or needs, which will be defined in more detail during the successive phases of the process, and if possible quantified. Sometimes these initial requirements are vaguely defined, especially when the user does not know exactly what he would like to have. The first task of a design engineer is to define the requirements or needs as accurate as possible. Every project must starts with a Need Assessment, in order to be able to formulate requirements or compromises. A need usually arise from dissatisfaction with the existing situation (dry room in example 2.1), and it is an answer to the question "What is Required"? They may be to reduce expenses, increase safety, increase reliability or performance. Needs may come from inputs of operating and maintenance personnel, or are generated by government agencies or by attitudes or decisions of the general public.
Example of an initial requirement:

> *"The acceptable probability of failure of a river-flood protection system should be smaller than 10-2 a year".*

After going through the feasibility and engineering design phases, this initial requirement is ultimately formulated in terms of more detailed or specific requirements, e.g. required length of bed protection required minimum freeboard required crest width compaction requirements of flood embankments required concrete quality of water retaining structures required tolerances during construction of the works etc.

The needs should be considered as factors requiring to start with a design or to proceed with a design in more detail. All these needs should be:

(1) valid
(2) known (in fact an important effort during the design process is devoted to search for the needs)
(3) given priorities.

Needs can be classified in the following categories:

(a) Basic needs.

What does the user wants? (therefore these needs are often also called user needs).

(b) Functional needs.

What is required for the functioning or performance of a structure or system?

(c) Design & Construction needs.

What is possible with the available know-how, equipment, material, etc.

(d) External needs.

What are the laws, regulations, habits, constraints such as environment, social. etc.

Apart from these categories, the needs have another dimension, that is Time. The life cycle of the project should be taken into consideration during defining the needs. Generally, hydraulic engineering structures have the following life cycle after the design process:

(1) Construction and installation
(2) Testing
(3) Initial functioning
(4) Operation and maintenance
(5) Extension
(6) Demolishing.

By combining the needs (a) to (d) with the stages (1) to (6), a list of the various needs and requirements can be developed.

7.2.2 *Basic elements*

Using the basic needs, first of all the Basic Elements of the problem should be identified. What are the basic elements of the problem?

Example 7. 1:

What is the main objective of a coastal protection project ?

(1) To prevent that land is permanently lost to the sea
(2) To prevent flooding of the land by storms at sea
(3) Both

In Indonesia in most cases only objective (1) is relevant, while in the Netherlands objective (2) is usually more relevant. This will lead to different basic elements. Lets concentrate on the Indonesian case. The essential components are dry land and stable underwater slope. In order to prevent loss of land, the stable under water slope has to remain stable. This means that the material of the underwater slope should not be moved away. So, the basic elements are dry land area and sediment balance of under water slope.

Example 7.2: An access channel towards a harbour has insufficient water depth for navigation purposes. What dominates the problem? Draft of the ship and the water depth! The water depth depends on the Water Level (WL) in the sea and the Bed Level (BL) of the channel.

The basic elements of the problem are.

(1) Draft of the ship
(2) Water Level at sea
(3) Bed Level of the access channel.

7.2.3 *The primary functional needs*

The Primary Functional Needs (PFN) or Basic Needs are those needs that must be met before meeting the other (= secondary) needs. The criteria for the assessment of the PFN are: (a) knowledge of all real needs, and (b) knowledge of priorities.

A useful tool to assess the PFN and other needs is the so-called "Six questions rule".

The six question rule

1. Needs :	What? - Goal - Shape - Size - Capacity - Elevation	4. Location :	Where? - Area - Envrionment - Situation - Relation
2. Reason :	Why? - Purpose - Efficiency - Consequences - Source	5. Time :	When ? - Day/night - Frequency - Duration - Sequence - Season
3. Method :	How? - Principles - Concepts - Priorities - Engineering - Non-engineering	6. Means :	With what ? - Equipment - Material - Money - Regulations

In practice it appears that after asking these questions, a more detailed analysis of the problem is required, as with each of the above 6 questions, additional questions would emerge. It is clear that the collection of these critical factors requires a lot of work and effort. Apart from the collection of these factors, an analysis of their inter-relations is required. These 6 questions are sometimes referred to as the "6 questions cube", with each question as a cube side. To a certain extent, the engineer may control the dimension of the cube by changing one of them. By combining each basic element of the problem with these 6 questions, one follows the so-called "Dynamic Approach", which is very much encouraged whenever possible to solve coastal engineering problems.

Example 7.3:
In a lowland area near a large tidal river a canal connects the hinterland with the tidal river. The hinterland should be prevented from being inundated during high tides. In addition existing navigation in the canal to and from the river should be maintained. The assignment for the engineer is to design "Something" that meets both requirements. The Primary Functional Needs of the problem can be defined as follows: "Provide a Mean that is able to Retain horizontal Water Pressure, that must be Watertight and should Allow Passage of Ships".

In many cases the way of defining the assignment or task of the engineer may determine the outcome of the design. By using "Provide a Mean" in the above example, the engineer has all the freedom to explore all the possibilities to arrive at his solution. On the other hand, by asking the engineer "To design a sluice gate", the engineers' creativity is focused on the sluice gate only, while there may be other solutions to the problem.

Example 7.4:
The Primary Functional Needs of an overhead crane can be defined as follows: "Provide a Mean of Lifting W kN through a Height of H m, and Transporting it within a rectangular area of B x L m2 at max. Speed of v m/s ".

The PFN in the above example can be divided into a number of sub-PFN. For example: Provide a Mean of Travelling L m; Provide a Mean of Supporting. W kN over a span of B m: Provide a Mean of Travelling B m; Provide a Mean of Hoisting H m; etc.

The next example illustrates the Dynamic Approach to analyze an Inland Navigation Problem.

Example 7.5:
Along a certain river reach, navigation is sometimes hampered between the Sea (point A) and an inland Harbour (point B). Assess the need for improvement of the situation!
*First of all we need to formulate the **Need for Improvement.** We should ask a number of questions and try to find an answer for it. What is navigation in this problem? Ships travelling along the river between point A and B. Why is this necessary? To bring cargo to Point B. It is the cargo only or should the ships go to point B as well? Should the ships return to point A empty or loaded? The answer to these questions may be acquired from the owner or principal.*
The Primary Functional Needs may be initially defined as follows: "Ships and shipload should go from A to B, and back". This initial defined PFN need to be redefined. refined or fine-tuned after going through the Problem analysis.
What are the Basic Elements of the problem?
(1) Inland Navigation
(2) Navigation Route
(3) Water-movement

The best way to solve this type of problem is to apply a dynamic approach using various entries, e.g. What? - Size
When?- Frequency
Etc.

These entries can be combined with the basic elements of the problem as follows:

Inland Navigation:	*What? - Size*
	When?- Frequency
	Etc.
Water Movement:	*What? - Discharge*
	When?- Season
	Etc.
Navigation Route:	*What? - Thalweg*
	When?- Season
	Etc.

The dynamic approach ultimately yields:

Inland Navigation:

<u>*What?*</u> - <u>*Size*</u>

* *Size ships*
* *Type ships*
* *Consequences of various types*
* *Interference between ships*
* *Disturbing factors for sip-movements*
* *Type of shipload*
* *Alternative transportation means*
* *Alternative transportation route*

Etc.

A list of requirements and critical factors is the ultimate result of the above exercise. An example of such a list is given below. Note that the list is not complete!

<u>*Inland Navigation:*</u>

* *Peak requirement*
* *Type & package of shipload*
* *Allowable waiting time*
* *Sailing time from A to B and back*
* *Type of ships*
* *In which direction loaded*
* *Navigation during construction*

<u>*Navigation Route:*</u>

* *Typical max. and min. cross section*
* *Minimum width*
* *location thalweg*
* *influence river bends*
* *Built up areas along the river*
* *Existing structures*
* *Alternative route during improvement works*

<u>*Water Movement:*</u>

* *River discharges as function of time*
* *When and how long low water levels*
* *Hydrographs*
* *Water velocities*
* *Other water users*
* *Water supply for or from other areas*

7.3 Problem definition or formulation

Probably the <u>most critical</u> step on the design process is the <u>Definition of the Problem</u>. The final design can differ greatly depending upon how the problem is defined at the very beginning. It is advantageous to define the problem as broadly as possible. Unusual or unconventional solutions will be less likely to be overlooked if the problem definition is broad. It should be realized however, that the degree to which a broad problem formulation can be pursued toward a final design will depend on factors often outside the control of the design engineer.

One approach that should not be taken is simply to improve the existing "solution" to the problem to be the problem itself. That approach immediately submerges us and we will find ourself generating solutions to a problem that we have failed to define. For example, *at this moment transport across a river is by ferry. However, the capacity of the ferry is not enough and long queues appear on both sides of the river. In that case one should not immediately think of more ferries. Perhaps a bridge is a better solution.*

The aim of the Problem Definition or Formulation is to define the problem as accurate as possible. For example:

(a) Main functions
(b) Behaviour of the structure
(c) Operational requirements
(d) Boundary conditions:
  * Nature
  * Environment
  * User

By using the 6 questions cube during the previous stage, questions were generated to acquire knowledge of the factors influencing the problem and to ultimately define the problem to be solved.

The definition of a problem should include writing down a formal **Problem Statement** which should express as specifically as possible what the design is intended to accomplish to achieve the Objectives. It should include objectives and goals, definitions of any special technical terms, the constraints placed on the design, and the criteria that will be used to evaluate the design. Obtaining a satisfactory definition of the problem is a task as big as, if not bigger than, solving the problem itself. Time spent in defining the problem properly and then writing a complete problem statement invariably pays off in efficient problem solving. The worst case is to arrive at a problem solution only to realize that the solution obtained is not for the real problem, in other words "The right Answer to the Wrong Question".

Problem definition is based on identifying the real needs of the user and formulating them in a set of goals for the problem solution. Design specifications are a major component of the Problem Statement. The Problem Statement defines the initially poor or vague identified Problem area as sharply as possible. Its essential elements are:

(a) A Need Statement;
(b) Goals, Aims and Objectives;
(c) Constraints and trade-offs;
(d) Definitions of terms or conditions;
(e) Criteria for evaluating the Design.

User needs should be categorized as to Performance, Time and Cost. Performance deals with what the design should do when it is completed and in operation. The Time dimension of need includes all time aspects of the design, especially the schedule for the project. Cost pertains to all monetary aspects of the design. Generally, the Performance goal will dominate over the time and cost goals at the start of the design process. However, as the process proceeds, it is common to find that the time and/or cost goals become dominant.

The goals of the project should be expressed in as complete detail as possible at the outset.

It is a common mistake "to plunge headlong into the problem solution before setting the goals". Then the tendency is to substitute the solution to represent the goal. By setting out the HOW rather than the WHAT, we increase the possibilities for creative design solutions. A very well defined set of design goals is called Design Specifications or Need Criteria. For a very routine design problem most of the problem statement will consist of standard design criteria. the Problem Statement of any design project should be reviewed almost continuously. As the design process proceed further, the goals become more detailed. Development of criteria to measure the achievement of those goals are needed. At the beginning of the design process, these criteria are called Target Specifications. By the end of the process they have solidified into specifications for construction, operation and maintenance.
The real purpose of specifications is to screen out unsatisfactory design solutions. Designs that pass all of the tests in the specifications should meet all of the goals set for a satisfactory solution. To be effective, a criterion must be discriminate, that is the answer to it must be a definite yes or no. Unfortunately, not all design criteria can be established with mathematical precision. For example, how do we establish specifications for an attractive appearance of a structure? Further, the degree of objectivity is also determined by who judges the criteria. Increased objectivity and credibility results can be achieved when the decision of the design engineer is replaced by a panel of colleagues engineers, or even better, by a panel of users.

Example 7.6: *Problem statement of the Navigation Problem in example 7.5*

Primary Functional Needs:
"Ships and shipload should travel between A and B in both directions".

Problem definition:
A system should be provided that is able to meet the Primary Functional Needs with the following requirements:

(a) Max. draft of the ships : .. . m;
(b) Max. Width of the ships : .. . m;
(c) Max. length of the ships : .. . m;
(d) Waiting time for handling per ship : less than .... hours;
(e) Downtime allowed max. .... % of time
(f) Etc.

under the following conditions:
(1) two full loaded ships should pass each other with a clearance of minimal .. m;
(2) ships carrying non-dangerous cargo should be able to travel throughout the year, during day and night;
(3) ships carrying dangerous cargo should be able to travel throughout the year, during day time only;
(4) Etc.

Other requirements:
(5) when river discharges reaches .... m3/s or higher, no navigation is allowed in the river;
(6) when the wind speed exceeds .... m/s, no navigation is allowed in the river;
(7) existing navigation should not be hampered or disturbed during river improvement or

construction activities and during maintenance works;

(8) *currents due to ship movements should not cause erosion of the river bank and river bed;*

(9) *Etc.*

Need Criteria:

(1) *Min. width of the river navigation channel should be ..... m;*

(2) *Min. clearance between river bed and bottom ships under all flow conditions should be .. m;*

(3) *Min. capacity of the handling facilities at B should be .. ships per day;*

(4) *Min. curvature of the river bands should be .. m;*

(5) *Etc.*

7.4 Constraints or boundary conditions

After going through the process of Problem Analysis, the problem can be formulated more accurately. the requirements which should be met by the solutions have been formulated and the conditions defined. Further, the relations between the characteristics of the solutions have been investigated. In many cases during this stage, one may conclude that the real problem is quite different from what has been assumed initially. Before looking for alternative solutions it is essential to look into Constraints or Boundary Conditions. Sometimes a distinction is made between Constraints (requirements following from society) and Boundary Conditions (requirements following from nature). For practical purposes this distict is not very relevant. Generally, Constraints are imposed by:

- Limited resources, e.g. financial, time, material, manpower, equipment, etc.
- Limitations due to material characteristics, e.g. strength, etc.
- natural conditions such as water levels, tides, etc.
- laws and regulations.

Every design problem will have certain boundaries or Constraints within which the solution must be found. The most basic constraints is that the design must not violate the Law of Nature. Legal constraints on engineering design are becoming increasingly important. Examples are regulations related to public health and safety. Many of these legal constraints grew out of social and cultural pressures. Other constraints pertaining environmental pollution, energy consumption, social, culture, etc. Another aspect that should be considered is sustainability of the final solution. Finally, the user may impose constraints on the design that are peculiar to his own desire. If the customer is willing to pay for them, these man-imposed constraints become part of the problem definition.
When defining the constraints and boundary conditions, it is important to realize that some of these conditions may change in time due to development in science and technology. Also activities of man may change the boundary conditions, for example water levels and discharges along a river may change due to the construction of embankments, polders, river engineering works, etc. Sometimes it is attractive to change or control the boundary conditions in order to arrive at a solution which initially does not meet the boundary conditions or to arrive at a more efficient solution.

Examples:

Constraints: *(following from society)*

- limited water	*- skilled labour*	*- poor soil condition*
- poor infra structure	*- poor O&M*	*- poor institutions*
- salt water intrusion	*- sediment in the water*	*- manpower*
- financial resources	*- environment*	*- laws & regulations*
- energy	*- social*	*- cultural*
- construction material	*- limited equipment*	*- topography*

Boundary Conditions: *(following from nature*

- Q river	*- topography*	*- climate*
- WL river	*- tides*	*- geotechnical condition*
- wave height	*- bathymetry*	*- sediment*
- geological conditions	*- tidal effects*	*- flood*

7.5 Generation of alternatives

An alternative can be described as a combination of measures which solve the problem sufficiently. An alternative can of course also consist of only one measure. Variants can indicate several variations within one alternative.
The development of alternatives is not only a technical matter; a hard analytical method cannot be presented. Important points of attention in the process of thinking about alternative solutions are, besides the problem itself:

* already indicated solutions
* experience in similar cases
* the causes of the problem
* the urgency of the problem.

In spite of this it is possible to give a few general applicable methods to come in a more or less systematic way to a set of reasonable alternatives:

* the zero-option
* the zero-plus option
* analog solutions
* symptom combatment
* brainstorming

7.5.1 preselection

It is very well possible that we have in this phase of the project study with a big number of alternatives, a too big number of alternatives. It is impractical and very costly to work out all alternatives, to present them and to include them in the final decision making process.
In those cases it is necessary to make a preselection. The big number of alternatives is reduced to a limited number of promising solutions. This selection is usually done without a clear definition of criteria. Important criteria are:

* costs
* potential realization

* <u>attractiveness</u>
* <u>dominance</u>
* <u>reliability</u>
* <u>discrimination</u>

7.5.2 *Determination of the effect*

The computation, or with other words, the prediction of the effect of the alternatives is one of the most difficult aspects of the project study.
First has to be determined <u>which effects</u> have to be taken into account for judgement of the various alternatives. <u>Criteria</u> for this choice might be:

* the <u>magnitude</u> of the effect
* the <u>discrimination</u> with respect to the alternatives
* <u>recursability</u>
* <u>time/duration</u>
* <u>frequency</u>

In the analysis of coastal problems, the costs are usually one of the most important criteria. It is important to make the number of effects to be taken into account not too big. On one hand it cost a lot of time and money to perform all required studies, and on the other hand the comparison of the alternatives only becomes more difficult. Clustering of the effects in order to come to a smaller number of judgement-parameters may solve this problem. Practically the total number of judgement-parameters should be limited to six to ten.
A very apparent way of clustering is put all effects action on one specific function, together in one group.

We can in this phase of the project-study distinguish two types of effects: <u>aimed</u>-effects and <u>side</u>-effects.
Aimed effects are simply those effects written down in the problem analysis, such as: desired beach width, prevention of erosion. These effects are the effects which have to be fulfilled by the design, by the alternative.
Side-effects, often also called undesired effects, are effects which occur, but do not contribute to the aim of the project. These effects might have a positive or a negative effect. For example if one performs a beach nourishment in order to combat chronic erosion (=aimed effect), one gets a wider beach. This is very pleasant for the beach recreation (=positive side-effect), but also more sand is blown inland in the agricultural area behind the coast (=negative side-effect).
The determination of these effects is not easy; in some cases the required knowledge is not available, in other cases it may cost a lot of time and money to perform the required studies. Very often these side-effects are in areas of interest which are not in the direct field of a ministry of public works. But mostly civil servants of this ministry determine wether these side-effects are included in the study or not. It is clear that this cannot be done without consultation of the parties involved (like farmers, fishermen, nature conservancy agencies, etc.)

For the determination of the effect the following <u>methods</u> are available:

* <u>computational models</u>
* <u>statistical analysis</u>

* dose-effect relations
* process-description
* expert judgement (Delphi method)
* expert systems

The determination of the effects always involves an uncertainty. The models used are in principle an approximation of the reality, and also the reality will often develop different than expected. A number of methods to decrease the uncertainties are:

* collection of more data
* do more research
* flexibility in the design
* perform a sensitivity analysis in order to get an impression of the robustness of the computational results.

In any case the uncertainties have to be mentioned in the project memorandum and be explained.

7.6 Comparison of the alternatives

The data collected have to be structured and presented in a clear way. This can be done by means of an effect-overview. In this effect-overview information can be both quantitatively as well as qualitatively. Quantitatively means that the effects are judged with respect to a given reference situation or that the effects of the various alternatives were compared with each other.
Whether the zero-option has to be included in this overview is sometimes under discussion. However it is advisable always to include this option, also when you are sure that this option will not be selected. In this way always a good reference is possible.
The zero-option is not always really do-nothing. The zero-option is what happens when no special decision is made and works are carried out acc. to "business as usual".

The effect-overview may be the last item of the project study. But depending on the kind of the problems some more elaboration of the data might be necessary. A number of evaluation techniques are available to do this. However not all techniques can be used in all cases. Sometimes a separate study is necessary to determine the most applicable evaluation technique. Some techniques are automatised and can handle very complex problems. These techniques are in general not very appropriate for coastal problems.
In coastal zone management the methods usually used are:

* aggregate approach
  This includes monetary methods, like the cost-benefit method.
* disaggregate approach
  This include the score-card method.

Once the impacts of the alternatives have been assessed, a major difficulty still remains: synthesizing the numerous and diverse impacts of each alternative and presenting them to the decisionmakers for comparison of alternatives. In an **aggregate approach** to synthesis, each impact is weighed by its relative importance and combined into some single, commensurate unit, such as money, worth, or utility.
One can try to aggregate all impacts (e.g. by expressing all impacts in monetary units). Then a decision is very simple.

The aggregate-technique has several major disadvantages. First the aggregation process loses considerable information: For example, it suppresses the fact that alternative A has environmental problems, whereas alternative B has financial problems.
Second, any single measure of worth depends very strongly on the weights given to de different impacts when they were combined and the assumptions used to get them into commensurate units. Unfortunately, these crucial weights and assumptions are often implicit or highly speculative. They may impose on the decisionmakers a value scheme bearing little relation to their concerns. For example, cost-benefit analysis implicitly assumes that a dollar's worth of one kind of benefit has the same value as a dollar's worth of another; yet in many public decisions, monetarily equivalent but otherwise dissimilar benefits would be valued differently by society. Also, in converting disparate impacts to monetary values, cost-benefit analysis must sometimes make speculative assumptions, such as: How much money is one square meter of coral reef worth ? And is the value of ten thousand square meters of coral reef the value of ten thousand times the value of one square meter ?
Third, the aggregate techniques are intended to help an individual decisionmaker choosing the preferred alternative, the one that best reflects his values (importance weights). Serious theoretical and practical problems arise when there are multiple decisionmakers: Whose values get used (the issue of inter-personal comparison of values), and what relative weight does the group give to the preferences of different individuals (the issue of equity) ?
It has been proved that there is no rational procedure for combining individual rankings into a group ranking which does not explicitly include inter-personal comparison of preferences. To make this comparison and to address the issue of equity, full consideration of the original impacts appears essential.
Finally, to be theoretically valid, the aggregate technique (others than cost-benefit analysis) require that the importance (value) of each impact be independent of the size of all other impacts. But in the real world, this condition is not always satisfied. Each impact that violates this condition must be suppressed, either by eliminating it or by treating is at the next level of aggregation.

Therefore it is better to choose a **disaggregate approach** that presents a column of impacts for each alternative, with each impact expressed in natural units. In comparing the alternatives, the decisionmaker can assign whatever weight he deems appropriate to each impact. Explicit consideration of weighing thus becomes an essential part of the decision process itself, as it should be. Prior analysis can consider the full range of possible effects, using the most natural description for each effect. Sometimes effects can be described in monetary terms and others in physical units; some are assigned with quantitative estimates (e.g. "100 jobs would be created), others with qualitative comparisons (e.g. "recreation opportunities would increase slightly"), and still others with statements of non-ordinal facts ("an attractive tourist site would be destroyed"). A disadvantage of this approach is that the amount of detail makes it difficult for the decisionmaker to see patters or to draw conclusions.

To aid the decisionmaker in recognizing patters and trading off disparate impact one can use **score-cards**, a technique developed by the Rand-corporation in the USA in 1971). Impact values are summarized (in natural units) in a table, each row representing one impact and each column representing an alternative. The score-card takes the table of impacts. Colours can be added to indicate each alternative **ranking** for a particular impact, for example Blue for the best value, Yellow for the worst and Grey for the intermediate values.

Example:
In a coastal area, erosion is a problem. Four alternatives have been investigated:

- *the use of a set-back system;*
- *the construction of a beach wall;*
- *to do beach nourishment;*
- *to build a groyne-system and to do some additional nourishment.*

Several impacts can be distinguished. Some of these impacts are quantified, some cannot, like social acceptance. Because colour-printing is not possible, different shades are used in the table. Of course this is an example. In a real case, the relevant impacts can be different, and also the alternatives considered will be different. The scorecard is given in Figure 7.1
From the score-card it is not obvious which is the best solution. If you look to government expenses on the long run, set-back is the cheapest, followed by a beach wall. However, the

alternative	set-back system	beachwall	nourishment only	groyne system
direct costs	0	10,000	3,000	12,000
capitalized maintenance costs	1,000	2,000	12,000	3,000
capit. cost for third parties	10,000	0	500	500
people to be relocated during lifetime	5000	0	0	0
social acceptance	--	+	+	+
sustainability of the solution	+	-	+ +	+
Flexibility of the solution	+	-	+	-
Ecological effects	o	-	+	+

Legend:

best option
intermediate option
worst option

Figure 7.1: Example of a score-card

set-back will cause social problems.

With the use of score-card the decision maker can evaluate the alternatives and make his decision. In fact the scorecard is a siple decision support system (DSS). And like every DSS, also a scorecare helps in taking a decision, it does not take the decision.

7.7 *References*

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8 INTERACTIONS BETWEEN COASTAL STRUCTURES AND NATURAL SYSTEM

8.1 Structures and currents

In coastal engineering ocean currents are not very important, as have already been stated in chapter 4. However, tidal currents can be of great importance. Some coastal engineering structures may have interactions with currents. One can distinguish two types of structures:

- Structures intended to stop or modify the current;
- Structures with an other aim, but with a secondary effect that they change the current patters.

Examples of the first type of structures are a closure dam in an estuary (current becomes zero, for example the Duriangkan reservoir dam at Batam island) or a causeway connecting two islands (the planned dam between Batam island and Tanjung Sauh Island to create a quiet harbour basin).
Examples of the second type of structures are groynes (intended to reduce the longshore sediment transport) but also causing an increase of tidal current at the tip of the groyne. This increase of current may cause an erosion-hole, unless bottom protection is provided.
In a river mouth a structure is sometimes provided to increase the velocity with the purpose of decreasing sedimentation. This will be worked out in section 8.3.

As a general conclusion one may state that in general the interaction between structures and currents is of minor importance for coastal engineers in Indonesia; this topic will therefore not be worked out in detail in this manual

8.2 Structures and waves

Especially along the ocean coasts of Indonesia, the impacts of waves on the coast is quite relevant. Along sandy coast waves are usually the most important driving force for morphological processes. For controlling these processes, wave impact has to be controlled. This can be done with several types of structures.
A general distinct can be made between floating structures, rubble structures and vertical wall structures. The general experience with floating structures is that they are rather vulnerable to wave action. Especially at ocean coast a floating structure has a high risk of getting damaged by the waves. Because of these drawbacks floating structures are, as a general rule, not recommended.
Vertical wall structures have the problem that they reflect waves. This is a rather negative drawback, consequently one should try to avoid vertical structures in coastal engineering. However, in some cases there is a strong need for special functions, which can only be provided in a convenient way by a vertical structure. An example of this are mooring facilities at quays. On other important example is the construction of a walkway near to the water (a boulevard). In those cases one has to select a vertical wall structure in spite of the fact that it has disadvantages with respect to wave impacts.
An other reason to select a vertical structure is in some cases that the amount of material to be used in construction can be significantly less in comparison with a sloping structure. This is, for example, the case with a breakwater in deep water.

A harbour breakwater can be designed as a rubble-mound or as a caisson type breakwater. In case there is a shortage of rock, caissons can be commercially quite attractive, because the material to be handled is much less. For the design of both the rubble-mound breakwater and the caisson breakwater is referred to chapter 9.

8.3 Structures and mud transport

In Indonesia, mainly in the Java Sea, the sediment mainly consists of fine particles, $d < 4$ μm. These particles have cohesive properties, because electro-statical forces comparable to or larger than the gravity forces are acting between the particles. Consequently the particles do not behave as individual particles, but tend to stick together and form aggregates known as flocs.
However, also because of the cohesion, sediment connected to the bottom will not easily go into suspension.
The erosion behaviour of such sediment is completely different from the settling behaviour. Because also the settling velocity is very low, there is usually no relation between the local active parameters (like waves and currents) and the erosion and deposition of material.

In general one can say that material is brought into suspension by strong currents and wave-action only. The threshold value is quite large. During low currents or weak wave action, no particles will be picked up from the bottom.

Usually the mixing is quite intensive. This means that at the end of the erosion process, the concentration of material in suspension is everywhere more or less the same.

After the strong currents and heavy wave action disappears, the material starts to settle down. However, the velocity where settling starts is much lower than the velocity where erosion starts.

The consequence is that in a rather turbulent environment (for example at sea during a windy day) everywhere at sea sediment is picked up, especially where waves are breaking or where currents are strong (this means that shallow areas, banks, etc. will be eroded first, because there the wave action at the bottom is stronger). The mixing takes care that the concentration in the water becomes constant everywhere. After the windy period, when the water is calm again, the sediment starts to settle down. Because the concentration is everywhere more or less the same, the amount of siltation in deeper sections is more than in shallow sections. Consequently deeper parts, like channels, sedimentate more than shallow parts.

The effect of this whole process is that the bottom topography becomes less and less extreme, it becomes flatter. Banks disappear and channels are filled up.

From the above follows that the only way to stop the siltation at unwanted locations is to prevent that silt laden water can come there or that the area becomes calm. Both options are extremely difficult to realize. Because of the tidal differences, there will be always flow in and out a certain area. The inflow will always bring silt-laden material, what will settle down.
The other option is to prevent that the water becomes quiet enough for settlement. This can be achieved by building structures creating more turbulence. Some groynes in rivers or river mouths can sometimes do so. But the effect is rather local, and not very effective.

In the mouth of a river, where there is a constant supply of water, one may use such a system, but along a coast itself, with only tidal currents (which implies that there is always a period of slack water) this is usually impossible.

The reversed problem, i.e. to increase the amount of siltation (e.g. to make a reclamation) is very well possible. This can be done by make it easy for the water to flow into a certain compartment, and allow it only to flow out in a very quiet and controlled way. Small groynes perpendicular to the coastline, combined with small ditches dug out of the mud can be very effective. In the north of the Netherlands and Germany, vast areas have been reclaimed using this method.
The groynes and small dams can be made of bamboo or other soft wood, or even using sods of vegetation. The height of these dams is in the order of 30 - 50 cm. When the dams are made higher there is the risk that they induce high currents, causing erosion parallel to the dams or erosion holes by the water flowing over the dams. A grid of dams is made with intermediate distances of 200 - 400 m. At the seaward end, the groynes are connected with low coast-parallel dams. These parallel dams do not completely close of the compartment, but in the middle a wide opening is left. Through this opening the water can enter, but especially leave the compartment slowly. Inside the grid, ditches are made, perpendicular to the coastline. They have a distance in the order of 6 - 10 meters. At a few places ditches are made parallel to the coastline. In this way strong currents on the siltation fields are prevented, and only limited silt will be transported back to the sea during falling water. When the ditches are silted up, they are cleaned sufficiently, and the outcoming mud is placed over the fields ore used to make a small levee at one side of the ditch. Sunshine and wind will stiffen the mud. When the fields are above low water, mangroves can be planted to stimulate further the siltation. When the fields are above high water, salt resistant plants can be used, which will cultivate the area. After some time the soil will become salt-free, and the area can be used as paddies. Before starting the rice cultivation, a sea-dike has to be constructed, to prevent that occasional high water will flow into the paddies.

The conclusion is that structures to control the mud transport are not easy to design, structures to prevent sedimentation by mud are not very effective, while structures increasing sedimentation can be rather effective.

8.3.1 Sedimentation in basins

In contradiction to sand, transport, erosion and accretion due to fine particles, like mud, does not depend very much on the local parameters. This is due to the fact that it takes quite a long time before the fine particles settle down. Because large parts of the Indonesian waters are along muddy coasts, this is a rather important point. In general one may state that sea-water contains a certain concentration of mud. This concentration depends on the supply of silt-laden wter from the rivers and the ability of the waves to keep this silt into suspension. Therefore, as long as there is wave action, only a small fraction of the fine particles will start to settle downs.
However, as soon as this water comes into a protected area (for example in a harbour basin), there are no waves any more to keep the sand in suspension and the settling process will start. In order to make an estimate of the amount of siltation in a harbour basin, one can calculate the total amount of "fresh" silt-laden water coming into the harbour every tide (= the tidal prism). Multiplying this quantity of water with the concentration of silt in the sea

in front of the harbour gives the amount of silt which enters every tide into the harbour. However, the quantity of water which had entered the harbour also has to leave the harbour. But the concentration of sediment in the water leaving the harbour is much lower. Multiplying the amount of water leaving the harbour with the concentration in "clean" water gives the total amount of sediment leaving the harbour. The difference stays in the harbour and causes the harbour siltation.
This process can be illustrated using the following example:

Given a harbour:

lenght	L (m)
width	W (m)
depth	d (m)

At sea, there is a tide with a tidal difference of Td (m) and a tidal period of Tp hrs (for example 12hrs, 25 min)

Density

seawater	ρ_w (kg/m^3)
sediment	ρ_s (kg/m^3)
bottom mud	ρ_m (kg/m^3)

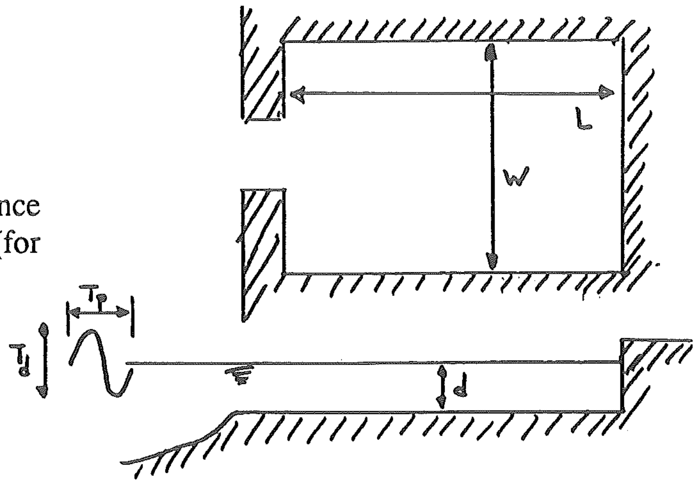

Figure 8.1: Definitions

Concentration of sediment in:

outside water	c_{sea} (g/l = kg/m^3)
"clear" water	c_{clean} (g/l = kg/m^3)

Each tide Td * L * w (m^3) of water comes in and goes out of the harbour. The concentration of the incoming water is c_{sea}, of the outgoing water is c_{clean}. Consequently eacht tide:

$$S_0 = [c_{sea} - c_{clean}] * Td * L * d \qquad [kg] \tag{8.1}$$

stays in the harbour.
One m^3 of mud weights:

$$1 * \rho_{mud} = v_v * \rho_w + (1 - v_v) * \rho_s \tag{8.2}$$

Therefore the void ratio is:

$$v_v = \frac{\rho_s - \rho_w}{\rho_s - \rho_m} \tag{8.3}$$

So, 1 m^3 o mud contains $(1-v_v)*\rho_s$ kg of sediment.
Therefor, 1 kg of sediment gives:

$$\frac{1}{(1-v_v)\rho_s} \qquad m^3 \quad of\ mud \tag{8.4}$$

Multiply (1) with (4). This is the amount of m^3 that settles each tide in the harbour, expressin in m^3/tide. We call this value s1:
This is:
This is:

$$S_1 = S_0 \frac{1}{(1-v_v)\rho_s} \tag{8.5}$$

$$S_2 = S_1 * \frac{24}{T_p} * 365 \qquad [m^3/year] \tag{8.6}$$

$$S_3 = \frac{S_2}{(L*W)} \tag{8.7}$$

The parameters, usually unknown in this computation are c_{sea}, c_{clean} and ρ_{mud}.

For Rotterdam c_{sea} is 50 - 100 mg/l
c_{clean} is 10 mg/l
ρ_{mud} is 1200 kg/m^3

For the north coast of Java, during dry season c_{sea} is 75 - 100 mg/l, for the rainy season c_{sea} is 300 - 400 mg/l.

8.3.2 Sedimentation in mangrove areas

In mangrove areas wave action is decreased significantly by the extra resistance of the stilt roots and the pneumatophores. Also strong currents are decreased, but the amount of water flowing into a mangrove area is usually not decreased very much. For detailed information regarding waves and currents in mangrove areas is referred to SCHIERECK & BOOIJ [1994]. This means that during rising water (due to the tide) much silt-laden water can flow into the mangrove area. During slack water, because of the absence of wave action, the sediment in the water can easily settle down. This sedimentation will cause that the bottom level will rise. At a certain moment the layer will become that high, that mangroves will not grow any more, and that by natural succession, they will be replaced by other species. However, because the gradient of the bottom in muddy areas (and consequently also in mangrove areas) is extremely small, mangrove forest can become extremely wide, especially in those cases where the tidal range is significant. On the front side of the mangrove forest, new mangroves can settle, provided there is enough sediment to allow the bottom to raise above the low water level. Mangroves will not grow below the low-water line.
In order to achieve a faster sedimentation below low-water and the stimulate sedimentation to a level where mangroves can be planted, one may use a groyne-system, as has been indicated earlier in this section.

8.4 Structures and sand transport

8.4.1 General aspect

In this chapter the effect of structures on longshore sediment transport is discussed. One has to stress that this chapter is valid for sandy coasts, and not for mud coasts. It is also important to realise that all sediment transport formulas only give capacity to transport sand. Of course, when no sand is available, there will be no sediment transport. This occurs quite often along rocky coastlines.

When coastal erosion occurs it is sometimes necessary to combat this erosion. In principle this can be done in two ways:

- protecting the coastline, so that the erosion at the endangered spot decreases or becomes zero;
- supplying sand, so that the discharge of sand by the longshore current is no longer a problem for the endangered spot;

In general the first solution is called the "hard" solution, the second one the "soft" solution.
Often supplying sand is compared with normal maintenance of a structure, like the painting of a bridge or the resoling of your shoes.
There are in the coastal engineering profession dedicated advocates for the hard solutions, as well as dedicated advocates of the soft solutions. As usual, it will never be possible to tell what is the "best" solution. The optimum depends usually very much on the availability of money on the short and the long term, on the importance of the coastal stretch, and -last but not least- on the importance of the adjacent coastal stretches.
In this manual, an introduction will firstly be given on the natural stable form of the crenulate bay (or zeta-bay), which leads to the protection using artificial headlands. Usually these are the larger-sized structures.
On the medium scale groynes and offshore breakwaters (detached breakwaters) are often used.
Beach walls and revetments are a very special type of coastal protection, because they do not have much influence on the coastal erosion, but give a certain (usually temporary) safety against "falling into the sea" for the buildings just landward of the wall. For example at a coast with an erosion rate of 1 m/year, a beach wall may stop the landward movement of the coastline for e.g. 10 years. However erosion continues, and is deeping the seabed in front of the wall. After 10 years it has become so deep that the wall collapses, and immediately there is 10 m erosion. So beach walls may change an erosion rate of 1 m/year to 10 m per 10 years.
The soft solutions (artificial beach nourishment) will be discussed in the next chapter, dealing with coastal maintenance.

When there is an obstruction perpendicular on the beach, extending through the whole breaker line, sediment cannot be transported at this point. A classic example of such an obstruction is a groyne. The effect of the construction of a groyne is that accretion will occur at the upstream side of the groyne, while erosion will take place at the leeward. Accretion will continue until either:

a) the coastline is moved seaward so much that sand may move around the tip of the groyne
b) the new coastline is turned so much that it is now parallel to the incoming waves.

With a groyne-system one can decrease or stop the longshore transport in a part of the coast. In this way one may be able to stabilise an eroding coastline. However, one should realise that there will be always leeside erosion at the end of a groyne system. This means that groyne-systems do not solve erosion problems, but only move the problem to another location.

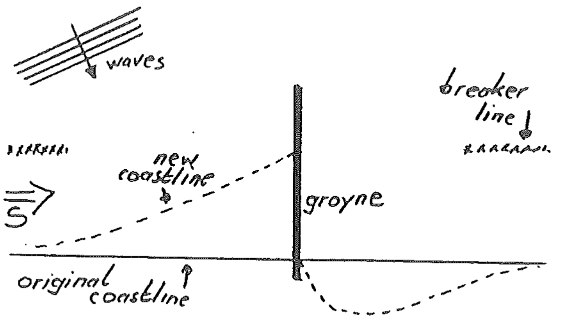

Figure 8.1: Effect of a single groyne

The construction of harbour-breakwater usually induce an identical problem. At the upstream side of the breakwaters there is an accretion zone. When this accretion zone reaches the tip of the breakwater the sand moves into the access channel. Consequently the channel has to be dredged to maintain the required depth. At the leeward side of the port, beaches are starving from sand. There is erosion. This problem can be solved by sand bypassing. This means that sand is removed from the upstream beach and transported to the leeside beach. There it is dumped and will be transported further by the longshore current. A clear example of this phenomenon can be found at the harbour of Lagos, Nigeria.

In some cases longshore transport is not stopped by an obstruction but by the fact that there is no propulsing force any more. This happens behind a detached breakwater (or offshore breakwater).

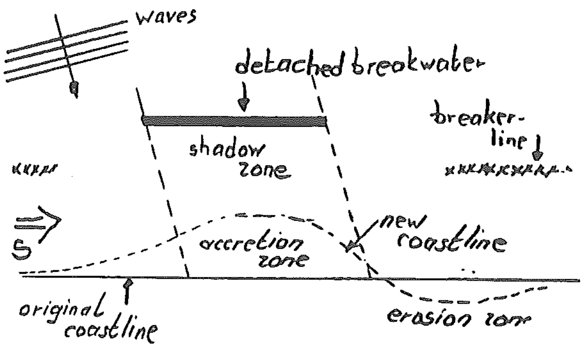

Figure 8.2: Effect of offshore breakwaters

In the shadow zone behind the breakwater there are hardly any waves, so also no breaking waves, and consequently no long-shore current.

The sand, coming from the upstream side can not be transported further and will be deposited. An accretion bulge will appear behind the breakwater, which we call a salient. Leeward of the breakwater there will be erosion.

If the breakwater is long enough, the salient may become connected to the breakwater. Such a connection we call tombolo. In some cases, tombolos are formed behind natural rocks, just offshore of the beach.
Tombolo-formation occurs mainly at places where the tidal differences are very small, like along the Mediterranean coast.

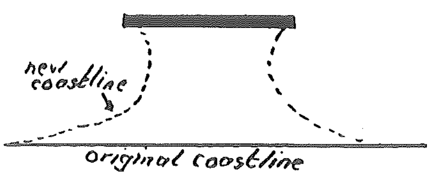

Figure 8.3: Tombolo formation

Remark:
In case the wave direction varies considerably over the year, one may have erosion and accretion zones at both sides of the construction. The principle remains the same.

8.4.2 *Crenulate shaped bays and headlands*

introduction

A coastline is rarely straight; some segments may curve gently in plan, while others are more indented, or even more intricate configuration with various shapes and sizes. These planforms have various names such as bay, gulf, and sea, depending on their indentation and magnitude of water enclosed. Bays containing sand are the most dynamic with regard to beach processes. Those formed between headlands on the coast can be observed on maps and aerial photographs. It is essential to know how and why they are formed and to determine their stability.

Under the influence of global wind systems most coasts receive persistent swell waves from some oblique direction. Already in 1938 Lewis remarked "Beaches tend to be built up transverse to the direction of approach of the most important beach constructing waves". Such features are observable on nautical charts and aerial photographs and can be correlated with the sea and swell charts available for the various oceans.

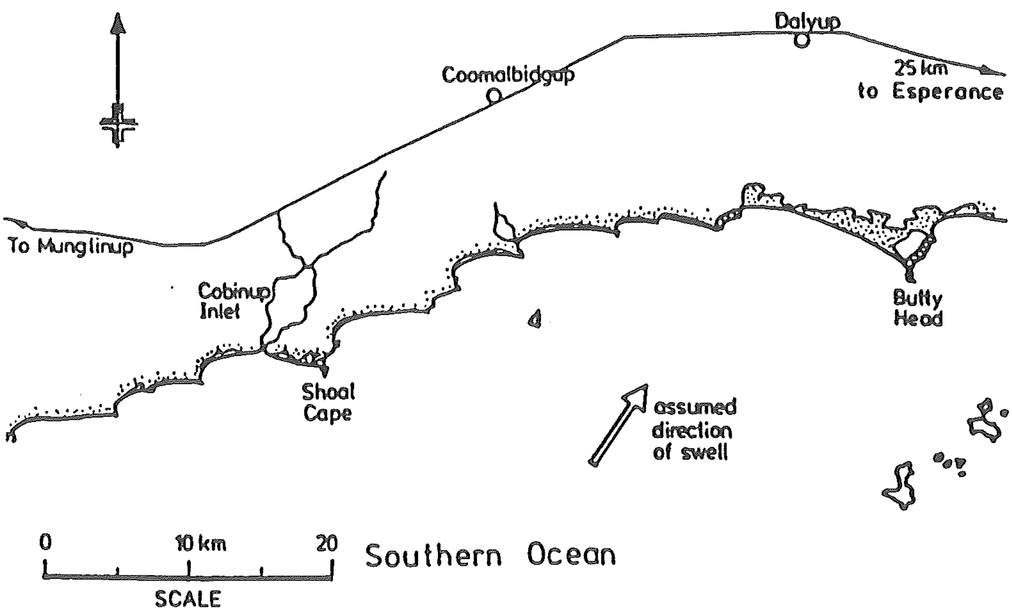

Figure 8.4: A chain of crenulate chaped bays in Wstern Australia [from SILVESTER AND HSU, 1993]

The orientation of these bays is an excellent indicator of the direction of net sediment movement along the coast. By observing these shapes along large reaches of shoreline it can be shown that littoral drift is in the same direction for many kilometres or even hundreds of kilometres. Half-Moon Bay in California is one of the best known examples of this kind, in which the curved waterline comprises three main segments, a downcoast tangential section, a curved element approximating a logarithmic spiral and an almost circular beach in the lee of the upcoast headland.

These crenulate shaped bays are ubiquitous, not only on oceanic margins but also along coasts of enclosed seas, lakes and river shorelines. The indicate Nature's method of balancing wave energy and load of sediment transport. In this manner coasts have been maintained in position for thousands, or even millions of years.

It will be shown that the complete periphery of each bay can be defined by a polynomial equation utilizing arcs from the point of diffraction at the upcoast headland to points on the coast. These radii are angled to the wave-crest alignment at this point, with a specific value for the bay extremity whose angle to this alignment defines the obliquity of arrival. A ratio of arc lengths to various points around the periphery is related to their respective angles in a manner that predicts the complete bayed shoreline for stable conditions, when no further littoral drift takes place. Even where a seawall is exposed within a developing bay the beach segments on either side will assume the crenulate shape as though the seawall did not exist.

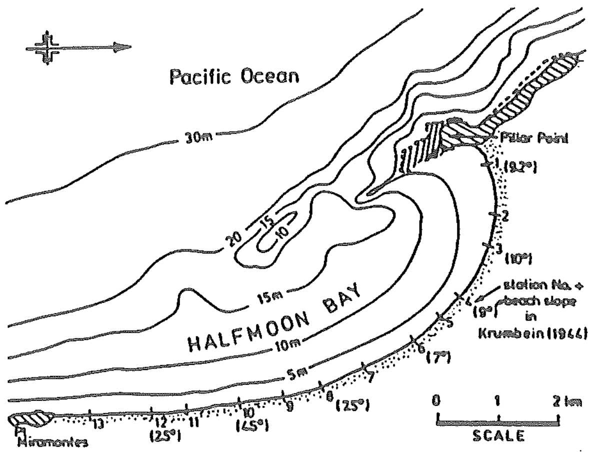

Figure 8.5: Planform of Half-Moon Bay

Silvester (1960) advocated very much the use of crenulate shaped bays as an engineering tool. In many cases mathematical description of the crenulate shaped bay allows a good prediction of the development of a future stable coastline after the construction of a civil engineering work. The use of artificial headlands and crenulate shaped bays between them can sometimes be used as a method for coastal protection. For example, part of the coastline of Singapore is protected this way. However, artificial headlands suffer from the same problem as groynes and detached breakwaters. There is always a leeside erosion downstream of the protected area.

bay orientation

When a reasonable length of coast is considered, the direction of the persistent swell is constant, even when refracted across the continental shelf to the beach. Should a series of headlands be almost aligned along it, the wave orthogonals should be parallel and hence the downcoast tangents of successive bays are also similarly oriented.

In terms of stability these bays may be in dynamic equilibrium with continual sediment supply or in static equilibrium when no further littoral drift is taking place, while many existing bays, as measured from maps or hydrographic charts, may not be in static equilibrium,but can remain in dynamic equilibrium for long periods. This is when constant supply of material from upcoast or within the embayment is passing through the bay, so maintaining a shoreline that is not so indented as that of the static equilibrium version.

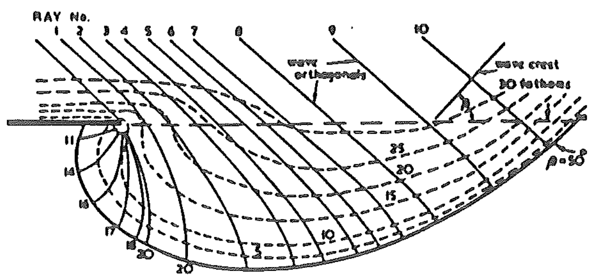

Figure 8.6: Wave refraction patterns in static equilibrium bay [Silvester and Hsu, 1993]

However, if the upcoast supply of material is cut off, the bay will become more indented until littoral drift ceases. The plan shape is then in static equilibrium and it is for this condition that it can be related to the wave obliquity. At this stage all waves arrive normal to the beach. Obliquity (β) is measured by the angle of wave crests at the upcoast headland to a control line joining the point of diffraction with the downcoast limit of the bay, see the following figure. This is the same angle as between the control line and the downcoast tangent to the bay.

Two empirical equations have been proposed in deriving bay shaped beaches in the last 30 years, these being referred to as *parabolic* and *logarithmic spiral.*

logarithmic bay shape

A definition sketch of such spirals is given in the following figures of which the equation is:

$$\frac{R_2}{R_1} = \exp(\theta \cot\alpha)$$

where θ is the angle between radii R_2 and R_1 (where $R_2 > R_1$) and α is the constant angle between either radius and its tangent to the curve.

The centre of the logarithmic spiral has no real physical meaning, it cannot be related to a given point in the topography. The ratio of R_2/R_1 or α changes as the bay is eroded from a straight alignment to its final equilibrium shape. There are specific values for the stable condition, varying with the wave obliquity β only, as depicted in the following figure. The value of β is the same as defined previously.

So, there is only one value of R_2/R_1 or α of the logarithmic spiral relationship for a stable bay for any given obliquity β. Using transparent templates one can find the direction of β, given the stable shape of a bay. Or, given a value of β one can determine the final, stable shape of the bay.

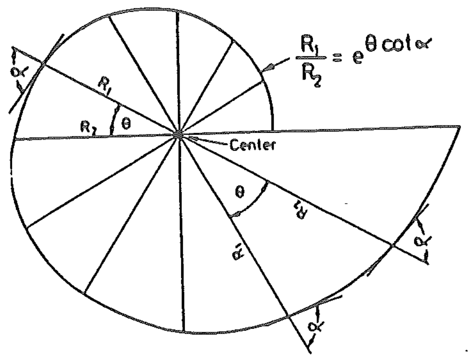

Figure 8.7: Definition sketch of logarithmic spiral

Although the scale of reproduction for maps or aerial photographs does not matter, since any part of a complete spiral can be used, the fitting of templates is not precise. The spiral applies only to the curved section of beach in the shadow zone of the upcoast headland, and its centre does not match the point at which diffraction takes place. It should be emphasized that bays assume this crenulate shape even before stability is reached and hence are sometimes assumed to be in equilibrium. For this reason it is difficult for engineers to apply this criterion for stability.

parabolic bay shape

The formula for the parabolic bay can be described using the values R_0 (or *control line*) at angle β. For a bay in static equilibrium this angle β is the same between R_0 and the tangent to the downcoast beach line. Even though the bay may not be completely stable, this tangent alignment is likely to be reached prior to the bay eroding back to its limiting shape. The subsequent erosion takes place at the deepest indentation zone.

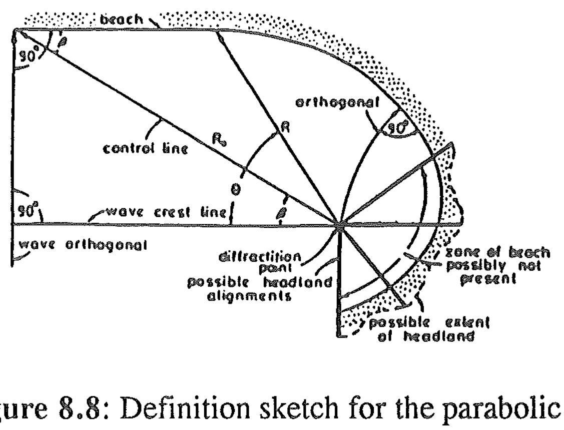

Figure 8.8: Definition sketch for the parabolic approach to bay shape

For engineering applications, nondimensional parameters are preferred. The value of R at any angle θ has to be normalized by taking its ratio to R_0, which is the length of the control line. It is desirable that the arc ratio R/R_0 be accurately predicted for any value of θ for a given bay with known obliquity β. The ratio R/R_0 can then be related in some way to β and θ, separately or in combination. This results in the following equation:

$$\frac{R}{R_0} = 0.81 \frac{\beta^{0.83}}{\theta^{0.77}}$$

which is also presented in the following graph, with data from specific model bays with different values of θ.

It can be seen that for $\theta = 45°$ to $90°$ the data point follow the curve closely, but for $\theta = 120°$ to $180°$ they are more scattered. It was found that for large values of β and θ the exponents for the best computer fit differed from the values in the above equation. So this formula is not universally valid and should only be used for θ less than $90°$.

In order to overcome this problem data from various models have been put into a curve fitting program in order to derive a polynomial formula. It was found that a quadratic polynome is sufficient to obtain bay outlines which match actual bays extremely well for the complete periphery. For any specific β and R_0 the ration β/θ, together with the three polynomial coefficients, provide ratios of R/R_0 from which the arc R is derived. The polynome used is:

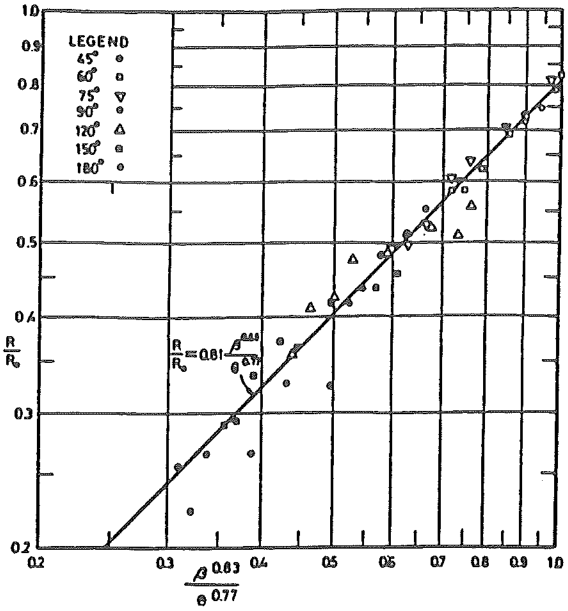

Figure 8.9: Parabolic bay curves [Silvester and Hus, 1993]

$$\frac{R}{R_0} = C_0 + C_1\left(\frac{\beta}{\theta}\right) + C_2\left(\frac{\beta}{\theta}\right)^2$$

The values of C_0, C_1 and C_2 vary uniformly with β as indicated in the following figure.

For convenience, these values are also presented in a tabulated form.

Comparison of both model and prototype data of various bays showed that both the logarithmic and the parabolic shape match very well with both model bays and prototype bays.

However, because the parabolic shape is much easier to handle in engineering practice it is therefore preferable.

Some variables associated with natural bays, such as their beach profiles and the wave characteristics, other than wave obliquity, are not included in this method of prediction. It is felt, however, that the primary parameter in shaping the curved waterlines is the value of β, whereas beach profiles, wave height, period (or steepness) are of secondary importance.

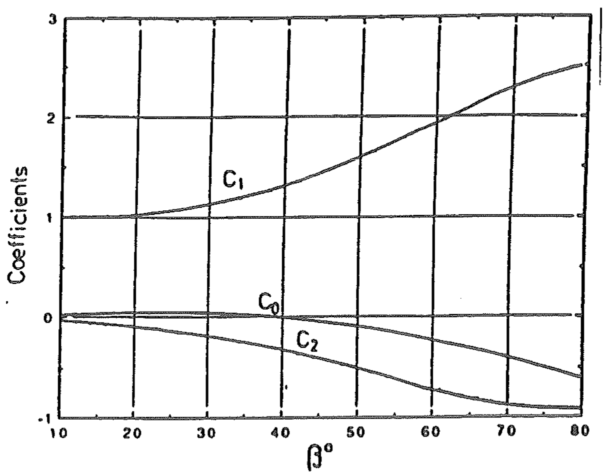

Figure 8.10: The three C coefficients over a range of wave obliquity.

β^o	Coefficients in Eq. C_o	C_1	C_2	Values of R/R_o for θ^o = 30	45	60	75	90	120	150	180
20	0.054	1.040	−0.094	0.705	0.497	0.39	0.324	0.280	0.225	0.191	0.168
22	0.054	1.053	−0.109	0.768	0.543	0.426	0.354	0.305	0.244	0.206	0.181
24	0.054	1.069	−0.125	0.829	0.588	0.461	0.383	0.330	0.263	0.222	0.194
26	0.052	1.088	−0.144	0.887	0.633	0.497	0.412	0.355	0.281	0.237	0.207
28	0.050	1.110	−0.164	0.944	0.677	0.532	0.442	0.379	0.300	0.251	0.219
30	0.046	1.136	−0.186	1.000	0.721	0.568	0.471	0.404	0.319	0.266	0.230
32	0.041	1.166	−0.210		0.763	0.603	0.500	0.429	0.337	0.280	0.242
34	0.034	1.199	−0.237		0.805	0.638	0.529	0.453	0.355	0.294	0.252
36	0.026	1.236	−0.265		0.845	0.672	0.558	0.478	0.373	0.307	0.262
38	0.015	1.277	−0.296		0.883	0.706	0.586	0.502	0.390	0.320	0.272
40	0.003	1.322	−0.328		0.919	0.739	0.615	0.526	0.407	0.332	0.281
42	−0.011	1.370	−0.362		0.953	0.771	0.643	0.550	0.424	0.344	0.289
44	−0.027	1.422	−0.398		0.983	0.802	0.670	0.573	0.441	0.356	0.297
46	−0.045	1.478	−0.435			0.832	0.698	0.596	0.457	0.367	0.304
48	−0.066	1.537	−0.473			0.861	0.724	0.619	0.473	0.378	0.311
50	−0.088	1.598	−0.512			0.888	0.750	0.642	0.489	0.388	0.317
52	−0.112	1.662	−0.552			0.914	0.775	0.664	0.505	0.398	0.322
54	−0.138	1.729	−0.592			0.938	0.800	0.686	0.520	0.408	0.327
56	−0.166	1.797	−0.632			0.960	0.823	0.707	0.535	0.417	0.332
58	−0.196	1.866	−0.671			0.981	0.846	0.728	0.549	0.425	0.336
60	−0.227	1.936	−0.710			1.000	0.867	0.748	0.563	0.434	0.339
62	−0.260	2.006	−0.746				0.888	0.768	0.577	0.441	0.342
64	−0.295	2.076	−0.781				0.908	0.787	0.590	0.449	0.345
66	−0.331	2.145	−0.813				0.927	0.805	0.603	0.456	0.346
68	−0.368	2.212	−0.842				0.945	0.823	0.615	0.462	0.348
70	−0.405	2.276	−0.867				0.963	0.840	0.627	0.468	0.349
72	−0.444	2.336	−0.888				0.981	0.857	0.638	0.473	0.349
74	−0.483	2.393	−0.903				1.000	0.874	0.649	0.478	0.348
76	−0.522	2.444	−0.912					0.891	0.660	0.482	0.347
78	−0.561	2.489	−0.915					0.909	0.670	0.486	0.346
80	−0.600	2.526	−0.910					0.927	0.680	0.489	0.343

practical application

In applying the parabolic form to natural bays, one difficulty is in defining the downcoast control point. This affects the value of *control line* length (R_0) and the value of wave obliquity (β). The latter is generally measured between the control line and the tangent at extremity of the bay. For a bay still in dynamic equilibrium this tangent may not be oriented to that for static equilibrium, although it will not change much as further erosion takes place. However, the scale of the bay being used from maps or aerial photographs could introduce a slight error. Taking a β value slightly greater than that measured by protractor could introduce offset this difference.

Some aerial photographs may contain crests of breaking waves which may be angled to the shore at the downcoast extremity of the bay.Using a tangent to these breaking crests for the determination of β could be erroneous, because the wave on the particular day may not be indicative of the swell just after a recent storm, which dictates the more persistent waterline. Thus, use of visual wave crests is not recommended in determining the tangent for β evaluation.

Generally, there is no difficulty in locating the upcoast control point where wave diffraction takes place. It may not be a point on the mainland but a small island or even reef offshore. An aerial photograph showing wave crests is an obvious advantage, but maps of suitable scale can exhibit points of diffraction. Once the wave orthogonal normal to the downcoast tangent is transferred across to the upcoast headland, it is the intercept with any of the above features that determines this control point.

Sensitivity of the location of the downcoast control point can be gauged by varying it slightly to increase R_0 with a commensurate decrease of β, or vice versa. This has been done for Magnetic Island in Queensland, Australia. As can bee seen in the following figure, the effect of selecting a different downcoast control point (B_1, B_2, or B_3) does not make much difference for the shape of the beach near the headland.

The wave-crest alignment is accepted as straight, which may not be the case if a shoal or depression is present in this region. Also it is assumed that the offshore area within the bay consists solely of sand with no rock outcrops or substantial sea-weed growth. Contours should be uniformly curved within the bay for waves to sculpture the predicted outline. Since for static equilibrium the waves arrive simultaneously around the bay periphery, any change in wave celerity due to local depth variations will cause a deviation in the waterline. For example, it a rocky shoal of reasonable dimensions exists within the bay, waves will take longer to arrive and so cause a protuberance on the coast. Equally, if depths are greater, a shorter time is taken by the waves and so a landward deviation will occur. All such variations should be smoothed when comparing existing to predicted shorelines.

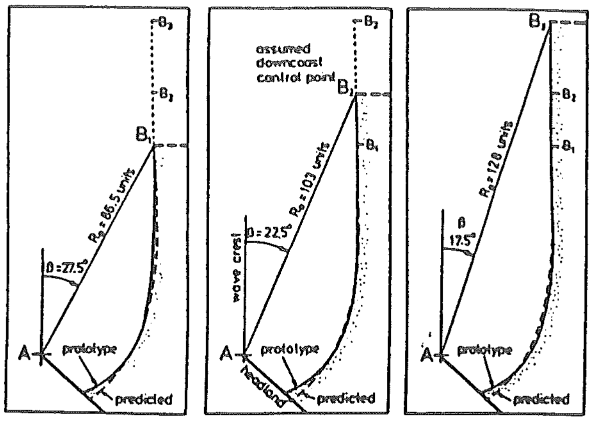

Figure 8.11: Effect of varying β on the bay periphery [Silevester and Hsu, 1993]

indentation ratio

Sometimes it is not essential to derive the actual static equilibrium shape to prove instability of an existing bayed coast. An alternative method that is swifter to apply could be what is termed the *indentation ratio*. As seen in the inset of the following figure, the greatest indentation (a) is measured normal from the control line to the point of largest retreat of the shoreline. This is obtained by drawing a tangent parallel to the control line which is asymptotic to the beach. The ratio a/R_0 has been correlated with β as in this figure both for model tests and prototype bays.

The equation of this curve is:

$$\frac{a}{R_0} = 0.0014\beta - 0.000094\beta^2$$

The indentation ratio can be measured expeditiously. Should the point fall below the line in the following figure, the bay is unstable and can erode back to a point on the line, but not above it.

As also seen in the figure, the angle θ_c to the radius at the point of contact with the greatest indentation can also be related to β. The equation so derived is:

Values of a/R_0, θ_c, and θ_c - β are listed in the table.

When sketching the static equilibrium by from the indentation ratio (a/R_0), it is commenced at the downcoast end at angle $\beta$to the control line and curved to become asymptotic to the tangent distant (a) from it. The beach line is then extended with greater curvature into the shadow zone until the upcoast headland is reached. Later, the more specific waterline can be drawn with the aid of formulae in the previous section of this chapter.

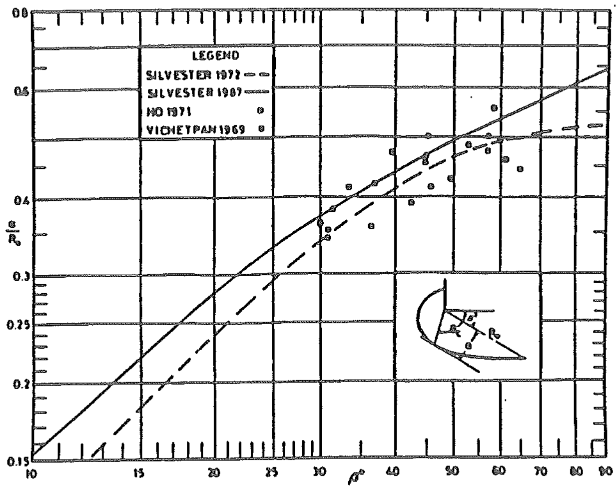

Figure 8.12: Intedation ratio (a/R_0) versus β

variations due to storms and seasons

The above equilibrium situation is based on the idea that there is a persistent swell from only one direction. As an average, this is true. But this results also in an average shape of the bay. Seasonal changes in the revealing swell direction may cause seasonal changes in the planform of the bay. Usually these seasonal changes are relatively small. However, in case of a bay with not supply of sand, one should investigate wether there is a given season or stormy period in which the sand may leave the bay (and probably will never come back). This effect may then result in a constant retreat of the coastline, although the bay seems to be stable acc. to rules given above.

Also due to storms there will be some change. Usually storms induce an increased transport in the offshore direction. This sand will come back in the following calm period. However, it will not always immediately come back to its original position. Also in this case one has to investigate whether the sand leaves the (closed) system of the bay.

$$\theta_c = 63° + 1.04\beta$$

β^o	a/R_o	θ_c^o	$\theta_c - \beta^o$
10	0.15	73.4	63.4
15	0.22	78.6	63.6
20	0.28	83.8	63.8
25	0.33	89.0	64.0
30	0.37	94.2	64.2
35	0.41	99.4	64.4
40	0.44	104.6	64.6
45	0.47	109.8	64.8
50	0.49	115.0	65.0
55	0.51	120.2	65.2
60	0.54	125.4	65.4
65	0.56	130.6	65.6
70	0.57	135.8	65.8
75	0.60	141.0	66.0
80	0.61	146.2	66.2
85	0.63	151.4	66.4
90	0.65	156.6	66.6

conclusion on artificial headlands

The technique of artificial headlands is strongly advocated by Silvester. He promotes is as a panacea for nearly all coastal problems. In many cases artificial headlands, or other type of hard protections, can be a very attractive solution. However one has to realize that along a coast with a continuous longshore transport, these constructions always have an influence on the transport, resulting in gradients. The consequence is thus a downdrift erosion, which can be mitigated by building another structure.

The technique of artificial headlands is very attractive for coasts with limited quantity of sand available, much rocky stretches and unstable (pocket) beaches. In those cases artificial headlands may improve the situation considerably.

8.4.3 Groynes

A groyne is a long, parallel-sided structure which is orientated approximately perpendicular to the coastline. It is relatively thin in relation to its length and extends from a point above the high water line to a point on the beach usually not far below the low water line. A groyne rarely extends beyond the zone in which wave-induced alongshore currents occur naturally. However, there are situations where longer groynes are built to deflect tidal currents.

Uses of groynes

Groynes are used by the coastal engineer as a means of beach management or control. If successfully controlled, a good beach is by far the best form of coastal defence because it combines optimum hydraulic performance (in terms of dissipating wave energy) with an environmentally attractive amenity. Beach material close inshore, particularly that associated with the formation of beaches along the coast, is continuously being moved around by wind, wave and current action. This littoral transport is classified separately as longshore and offshore/onshore movement.

A well-designed groyne system can:

- Arrest or slow down the longshore drift of material and, by building-up material in the groyne bays, stabilise the foreshore and protect the coastline.
- Deflect strong tidal currents away from the shoreline.

Groynes may also:

- In some situations, help to hold material on a beach that has no natural supply and has been artificially nourished.
- Control seasonal shifts of material alongshore and hence swings in shoreline within a bay.
- Reduce the long-term erosive effect of wave activity in an area of coastal defence by accumulating beach material in front of prominent hard beach heads such as sea walls, revetments, bulkheads and cliffs. This requires an adequate supply of material moving alongshore.
- Improve the extent and quality of an amenity beach.
- Increase the depth of beach material cover to an otherwise erodible sea bed soil.

It must be emphasised that groynes have no direct effect on onshore/offshore movement of the beach.

Recently KRAUS ET.AL [1994] analysed various decriptions of groyne properties in literature, and gave some useful comments. An overview is presented in the following table.

Types of groynes

Groynes can be either permeable or impermeable and there are several methods of construction for both types. In practice, a groyne can be designed to have a degree of permeability which suits the hydraulic environment. The permeability of a groyne can vary in the long term due either to structural deterioration or in blockage by drift material. For example, gaps can occur between the boards of a fence-type timber groyne due to abrasion or storm damage.

Availability and price of materials will often influence the type of construction adopted. However, other factors such as the possible need to vary the dimensions of a groyne whilst it is in service should also be considered. Both the length and spacing of groynes are usually considered fixed in the context of the design, although spacing can be effectively changed

Table I: Functional properties attributed to groynes and critical evaluation (from KRAUS ET AL, 1994)

	Property	Comment
1	Wave angle and wave height are leading parameters (longshore transport).	Accepted. For fixed groyne length, these parameters determine bypassing and the net and gross longshore transport.
2	Groyne length is a leading parameter for single groynes (length controls depth at tip of groyne).	Accepted, with groyne length defined relative to surfzone width.
3	Groyne length to spacing is a leading parameter for groyne field.	Accepted, see previous item.
4	Groins should be permeable.	Accepted. Permeable groynes allow after and sand to move alongshore, and reduce rip current formation and cell circulation.
5	Groynes function best on beaches with a pre-dominant longshore transport direction	Accepted, Groynes act as rectifiers of transport. As the ration of gross to net transport increases the retention function decreases.
6	The updrift shoreline at a groyne seldom reaches the seaward end of the groyne.	Accepted. Because of sand bypassing, groyne permeability, and reversals in transport, the updrift shoreline cannot reach the end of a groyne by longshore transport alone. On-shore transport is required for the shoreline to reach a groyne tip, for a groyne to be buried, or for a groyne compartment to fill naturally.
7	Groyne fields should be filled (and/or feeder beaches emplaced on the downdrift side).	Accepted. Filling promotes bypassing and mitigates downdrift erosion.
8	Groyne fields should be tapered if located to an unprotected beach.	Accepted. Tapering decreases the impoundment and acts as a transition from regions of erosion to regions of stability.
9	Groyne fields should be built from the downdrift to updrift direction.	Accepted, but with the caution that the construction schedule should be coordinated with expected changes in seasonal drift direction.
10	Groynes cause impoundment to the farthest point of the updrift beach and erosion to the farthest point of the downdrift beach.	Accepted. Filling a groyne does not guarantee 100% sand bypassing. Sand will be impounded along the entire updrift reach, causing erosion downdrift of the groyne(s).
11	Groynes erode the offshore profile.	Questionable and doubtful. No clear physical mechanism has been proposed.
12	Groynes erode the beach by rip-current jetting of sand far offshore.	Questionable. Short groynes cannot jet material far offshore, and permeable groynes reduce rip-current effect. However, long impermeable jetties might produce large rips and jet material beyond the average surfzone width.
13	For beaches with a large predominant wave direction, groynes should be oriented perpendicular to the breaking waves	Tentatively accepted. Oblique orientation may reduce rip-current generation.

by the introduction or removal of intermediate groynes. In some circumstances, it may be desirable to vary the permeability of a groyne whilst in service. Provision for varying the height of a groyne is for some types of beach an important design feature.

impermeable groynes

Truly impermeable groynes are constructed of sheet piling or concrete. This type of construction may be required where access is difficult for maintenance of other types of groynes. Timber groynes, usually comprise planks bolted between planks, can be made both permeable and impermeable. Low groynes consisting of rock (rubble) or placed blocks consist of impermeable material, but because the water can easily flow over the groyne, they behave in some cases like permeable groynes.

Vertical impermeable groynes reflect wave energy and this can result in adverse effects on the beach. Grouted rock, either using bitumen or cement grout, can be used to overcome this by forming low-profile non-reflective mound groynes. Because they have a fixed profile, they are not very suitable for beaches susceptible to large profile variations. Timber or steel piles

can be used to protect the toe of a grouted rock mound.
Sand filled porous bags made of geotextiles have also been used to form low-profile impermeable groynes. The height can be varied by adding a further layer. Such groynes are generally only used for temporary works as the exposed geotextile can be easily damaged. This last problem can be overcome to fill the bags with a sand/cement mixture. In Latin America this "Bolsacreto" system has been applied in several locations.
In the Netherlands, groynes are usually made using placed blocks (Basalton). The groynes are relatively low, and allow the tidal flow to go over the groyne without causing a lot of turbulence. Because the main sediment transport is in the lower part of the profile, these groynes nevertheless block a considerable part of the longshore sediment transport. Usually the Basalton is placed on a layer of crushed rock, which lies on top of a geotextile. Because during storm surges the groynes are well below the water level, the wave attack on the blocks is limited. In most cases a block-height of 25 cm is enough to prevent considerable damage after storm surges.

permeable groynes
Permeable groynes can be used on beaches which have sufficient supplies of littoral material for some proportion to pass through them. The resulting controlled beach tends to be more uniformly shaped in plan than that created by impermeable groynes.
Timber can be used in various ways to construct permeable groynes. A common method is the use of vertical piles driven into the beach. Usually the gap width between the piles is identical to the pile diameter.

composite groynes
In case of strong tidal currents and a large tidal difference it is useful to combine a permeable and an impermeable groyne. In such situation the stone, lower part has the function to more or less block the littoral drift. The timber piles on top of the groyne are intended to decrease the tidal currents over the beach, while the whole structure is intended to move the main tidal stream away from the coast. Especially when the tidal difference is great, groynes would become very high walls on the beach, resulting in a lot of reflection and scouring. Therefore the top part of the groyne is made of piles, decreasing the negative effects of a high wall.

Design of groyne lay-out
groyne height
The top level of a groyne will determine the maximum potential beach depth updrift of the groyne. The groyne structure should be designed for any combination of beach levels on either side of it between the local scour and the desired maximum beach depth. The extremes will usually be determined by the natural limiting winter and summer beach profiles.
Tidal variation should be accounted for in designing the height of a groyne.
Making groynes higher usually increases the effectiveness of a groyne as a "sand-catcher", however, crest levels above high water are not effective at all. Unfortunately higher groynes have some negative effects, like reflection of waves (causing extra erosion) and are more subject to wave action during storm conditions.
A disadvantage of high groynes is that they have a stronger tendency to create rip currents, running seawards on the updrift side of the the groyne. These rip currents and the flow at the ends of the groynes would easily be enough to cause erosion channels which would tend to transport some beach material out of the groyne bays. They also threaten the stability of the groyne structure.
In many papers it is reported that a groyne height of approx. half a meter above the beach

is an optimal height. In some places this rule of thumb is interpreted quite rigidity. As soon as a groyne becomes higher than 0.5 m above the beach (because of erosion of the beach), the groyne is lowered. As a consequence the efficiency of the groyne also decreases, resulting in more erosion of the beach. One should apply this rule of thumb only in the initial design phase, and never use it for groyne restoration projects.

groyne length

In order to stop the longshore transport, the groyne should extend until the end of the breaker-zone. Especially in case of a large tidal variation, this results in excessively long groynes. Shorter groynes will allow some sediment transport along the tip of the groyne. In most cases this is no problem because the intention is usually not to stop longshore transport, but to decrease the gradient in longshore transport. The amount of sand, passing along the tip of the groyne can be calculated, using the distribution of sediment transport capacity in the breaker-zone. For shingle beaches usually short groynes are sufficient, for coasts with fine sands, groynes are usually much longer.

groyne spacing

For a given groyne length, maximum groyne spacing should take into account the resulting variation in beach level at each side of the groyne, as illustrated in Figure 8.13. Thus, in a bay where the direction of wave attack is confined, groynes may be more widely spaced than on an exposed promontory. It also follows that steeper beaches require more closely spaced groynes.

Situations can arise where wave attack during the beach-building season (in NW Europe this is the summer) is at a relatively acute angle to the coastline (albeit that wave heights are moderate). This could require the groyne spacing ratio to be reduced to avoid large variations in beach crest levels. The rational determination of groyne spacing involves estimating the possible variation in beach shape that may take place within each groyne bay, whilst at the same time ensuring that an adequate reservoir of beach material is allowed to accumulate. The spacing should be selected so that, at periods of maximum drift, the angle of the beach head allows the requisite length of groyne to be constructed economically and avoids excessive loading on the groyne due to variations in beach level on either side. In addition, the beach crest must be sufficiently far seaward to ensure that any seal wall or revetment is provided with reasonable protection by the beach at all times.

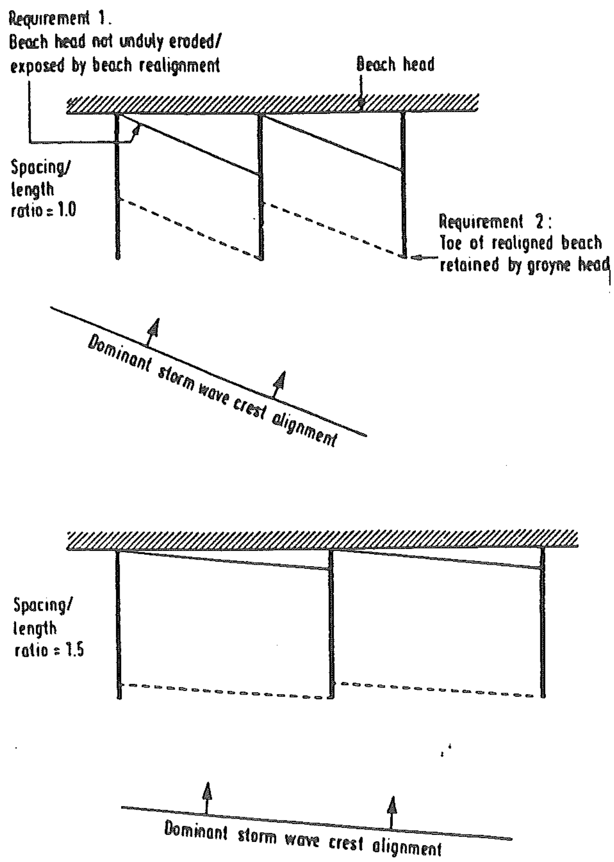

Figure 8.13: Effect of direction of wave attack on grone spacing

The latter design consideration requires a good estimate of the equilibrium beach profile geometry under storm conditions. For determination of these conditions, one may use the

Vellinga formula.

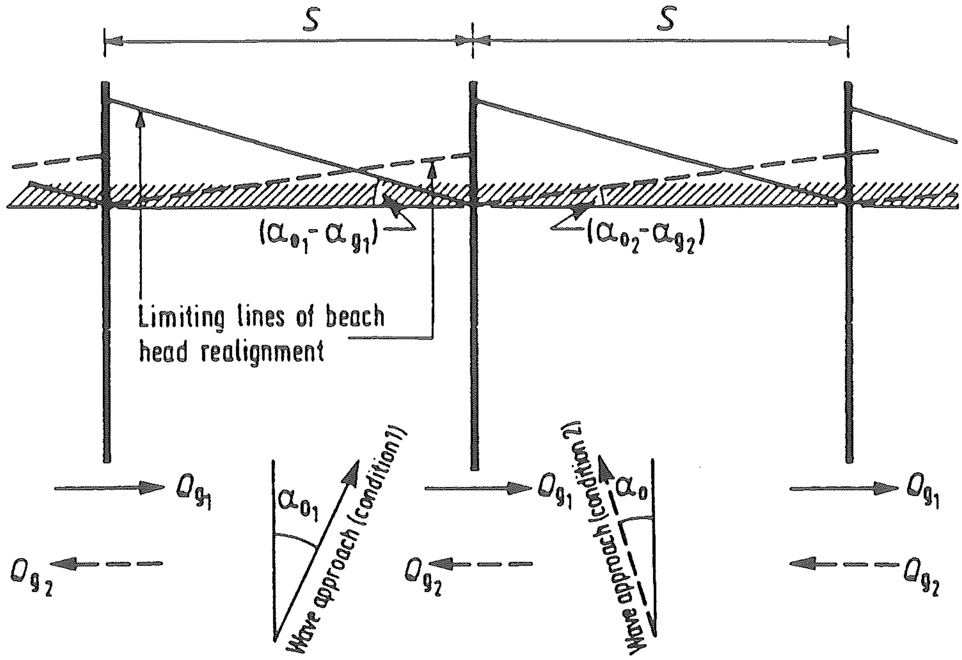

Figure 8.14: Variation of beach on a groyned shore

Figure 8.13 shows how groyne spacing S affects the extreme conditions on the beach. The horizontal distance between the beach crest on either side of the groyne is S tan $(\alpha_0 - \alpha_g)$. For a beach slope β the difference in level across the groyne will be [S tan $(\alpha_0 - \alpha_g)$] tan β. This is a conservative estimate, since local shelter in the lee of each groyne will reduce the difference in level, more so for sandy beaches.
In Great Britain the groyne spacing/length ratio varies between 0.5 and 2.5, depending on the beach material type. Elsewhere in the world, spacing to length ratios of up to 4.0 are common in areas where tidal ranges are small.
The selection of groyne spacing on the basis of littoral transport patterns can be done using a one line mathematical model (e.g. Unibest).

<u>inclination of groynes to coastline</u>
From the practical viewpoint, groynes are generally constructed transverse to the general direction of the coastline. In order to minimise structural damage during storms, groynes should ideally be aligned directly into the direction of the maximum storm waves. In practice, this is usually not possible. At many sites, there is, in fact, a substantial drift in both directions due to the multidirectional nature of the wave climate.
Groynes inclined slightly away from perpendicular to the coastline and in the downdrift direction (i.e. the direction of alongshore drift) are considered to provide the most effective control of littoral movement. However, where wave direction can vary and cause reverse drift, incline groynes become angled updrift. This can lead to scour on the new downdrift side of the groyne. Downdrift angling should, therefore, only be considered for conditions of predominantly unidirectional drift. So, in most cases groynes perpendicular to the coastline are generally more appropriate.

<u>beach head</u>
The design or rehabilitation of a groyne system should not be carried out without considering

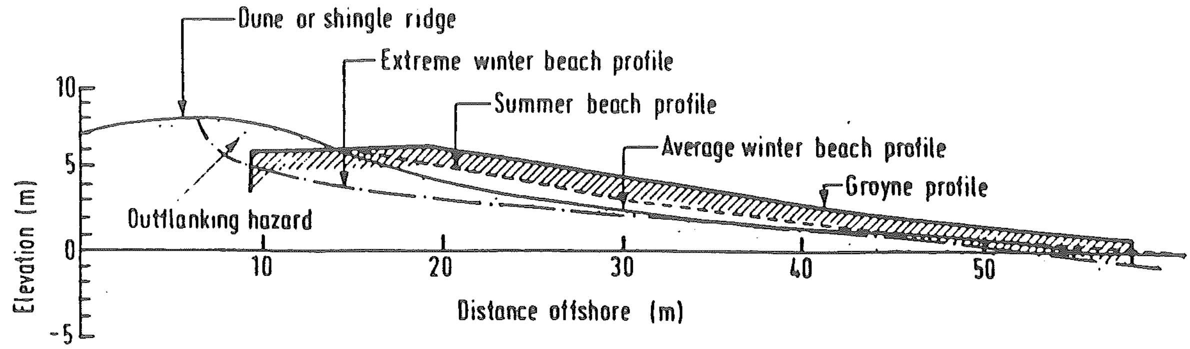

Figure 8.15: Groyne outflanking

the type of beach head. Wave energy reflected by a wall, cliff or over-steep beach head, is likely to encourage movement of beach material offshore. Such conditions would clearly not encourage a beach to improve or recover naturally, especially in the case of a sand beach. In these circumstances, it may be better to restore the beach by artificial nourishment and to consider a groyne system more as a means of stabilising and retaining the improved beach. Soft beach heads such as dune systems also require special considerations, since they are particularly susceptible to erosion under extreme wave and storm surge conditions. Providing the thickness of beach is sufficient, the profile will adjust to a storm profile accompanied by a temporary recession of the water line. Figure 8.15 illustrates the danger of outflanking the landward ends of the groynes.

8.4.4 Offshore breakwaters

Design parameters
The parameters for the design of the offshore breakwater can be divided into lay-out parameters and cross-sectional parameters.
For the lay-out these parameters are:

- distance from the coast (X)
- length of an (individual) breakwater (L)
- distance (gap) between the individual breakwaters (G)

For the cross-section these parameters are:

- crest elevation of the breakwater
- crest width of the breakwater
- porosity
- armour units
- toe depth
- slope

<u>distance from the coastline</u>
Most researchers report that the distance from the coast has to be wide enough to prevent the formation of tombolos. Therefore the distance to the coast has to be more than the length of the individual breakwaters. Also, the breakwater has to be located in the outer half of the breaker-zone. DALLY & POPE [1986] even suggest that a breakwater has to be located just outside the breaker-zone. A clear understanding of the optimal distance is not yet available. Also an analysis of 1552 constructed breakwaters shows a wide variety in distances.
Because the width of the breaker-zone depends not only on the bathymetry, but also on the

local wave climate, the distance from the coast also depends strongly on the wave climate. Therefore it is advisable to relate the distance from the coast to a wave condition with a certain exceedance frequency. This frequency depends on the aim of the structure (effective during average conditions or during extreme conditions). Suggested values for exceedance frequencies are 1/5 or 1/10 per year.

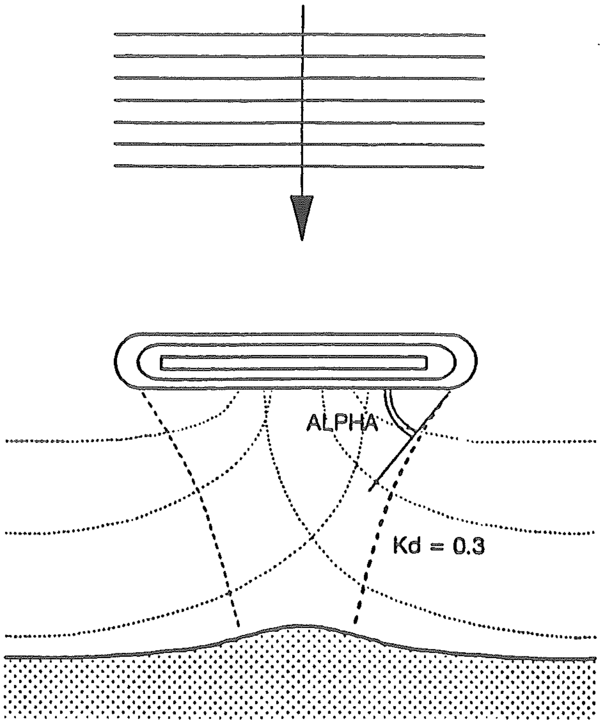

Figure 8.16: Definition of alpha

length of the breakwater

In the area behind the breakwater where the diffraction coefficient is less than 0.3, salient formation may occur; DALLY & POPE [1986]. This rule of thumb gives a lower boundary for the length of the breakwater. One should remark that this rule does not include any effect of the wave height. This is caused by the fact that most offshore breakwaters have been designed for situations with ocean swell, where the period is much more dominating than the wave height. It is therefore questionable if this rule is also valid at a coast with many short-crested waves.

gap width

The gap width should be more than one wave-length, for the prevention of rip-currents. However, it should be less than two wave lengths in order to have the set of breakwaters working as one system. In order to prevent the formation of a tombolo, $G > L^2/X$ and $G > L_p/2$ for breakwaters near to the coastline (L_p is the local wave length).

cross section

One of the major parameters of the cross-section is the crest height. The crest height determines the wave transmission. VAN DER MEER (1994) gives the following coefficients for wave transmision:

$K_t = 0.80$ for $-2 < R_c/H_i < -1.13$
$K_t = 0.46 - 0.3\, R_c/H_i$ for $-1.13 < R_c/H_i < 1.2$
$K_t = 0.10$ for $1.2 < R_c/H_i < 2.0$

in which K_t is the transmission coefficient (transmitted wave divided by incoming wave). H_i is the incoming wave, and R_c is the height of the breakwater above mean water level.
From the above it follows that a good starting point for the design is a value of $R_c/H_i = 1.2$.

other parameters

The influence of the slope, the porosity, the spectrum shape, the crest width, etc. are second order effects. Therefore it is advisable not to optimize the design of the detached breakwaters on these parameters, but to determine these parameters from the other aspects, like the availability of stones, the executional aspects, the accessibility of the crest by trucks, etc.

Design strategy
A detached breakwater can be determined using the following rules:

1. Determine the width of the breaker-zone x for a wave condition with an exceedance frequency in line with the design condition (1/5 or 1/10 year).
2. Select a distance from the coast $X > 0.5$ x.
3. Select a breakwater length L smaller than X, but larger than $2X/\tan \alpha$. The value of α is determined by the diffraction-line of a Kd coefficient of 0.3. (Warning: for short waves this step is questionable).
4. Select a distance between the breakwater G larger than L^2/X, but smaller than $2L_p$, in which L_p is the local wave length.
5. Determine the crest height of the breakwater with $R_c/H_i = 1.2$, in which H_i is the incoming wave height and R_c is the breakwater height above mean water level.
6. Determine the crest width, slope, porosity with other technical arguments (e.g. executional aspects).
7. Determine the effect on the coastline with a one-line model (e.g. Unibest).

Example:
Offshore breakwater in front of the coast near Scheveningen. The deepwater and breakerzone conditions for Scheveningen are as follows for return periods of 1, 5 and 10 years:

Return period	*(years)*	*1*	*5*	*10*
Waterlevel above M.S.L.	*(m)*	*2.35*	*2.80*	*3.00*
Deep water wave height	*(m)*	*5.7*	*6.4*	*6.5*
Peak period	*(s)*	*8.5*	*9.0*	*9.2*
waterdepth below M.S.L.	*(m)*	*-4.0*	*-4.0*	*-4.0*
Local wave height H_i	*(m)*	*3.20*	*3.47*	*3.58*
Distance of breakerline x	*(m)*	*225*	*225*	*225*

For the selected three conditions one finds that there is a significant increase in breaking around 4 m below M.S.L.. From the relation waterlevel/waveheight it follows that for all conditions the breakerindex is approx. 0.5. For Scheveningen this means that the breaker-line is approx. 225 m from the waterline.
The final selection of the distance from the coast depends also on factors like the shape of the bottom profile, the presence of groynes, etc. In case of Scheveningen the distance to the coast is selected as equal to the width of the breaker-zone. Because the waterdepth is not extreme, the volume of the cross-section remains rather limited.
The gap with should be twice the wave length. This results in a value of 60 m . The length of the breakwaters has to be at least 2 times the gap width, and no more than the distance to the coast (i.e. 225 m). The length of the breakwater therefore should be between 120 and 225 m. The crest height, based on a construction depth of 4 m below M.S.L., a Low Water of -0.74 and a breaker-index of 0.5 results in a crest of 1.36 m above Low Water. This is 1.22 m above M.S.L.

Summary of the design:

distance from the coast	*225 m*
gap width	*60 m*
length	*120 - 225 m*
crest height	*1.25 m above M.S.L.*

For the design of the stone weight of course a lower design frequency has to be selected. In *this case one may select 1/100 of 1/1000 per year. This results in the following stone weights:*

design frequency stone	*(yrs)*	*100*	*100*	*1000*	*1000*
tidal situation		*LW*	*HW*	*LW*	*HW*
waterlevel	*(m)*	*1.76*	*3.60*	*2.46*	*4.30*
H_s	*(m)*	*2.90*	*3.80*	*3.30*	*4.15*
T_p	*(sec)*	*11*	*11*	*11*	*11*
M_{50}	*(kg)*	*1044*	*1045*	*1025*	*1074*
S (in case M^{50} *= 2000 kg)*	*(-)*	*0.85*	*<0.5*	*<0.5*	*<0.5*

In 1 m of breakwater 95 ton of 1000/3000 kg stone and 33 ton of 10/200 kg stone is required. Together with 41 m² of geotextile this costs dfl 15,000. With a breakwater length of 120 m m and a gap width of 60 m, this results in a protection cost of dfl 10,000 per running meter of coastline. (prices at 1993 levels).

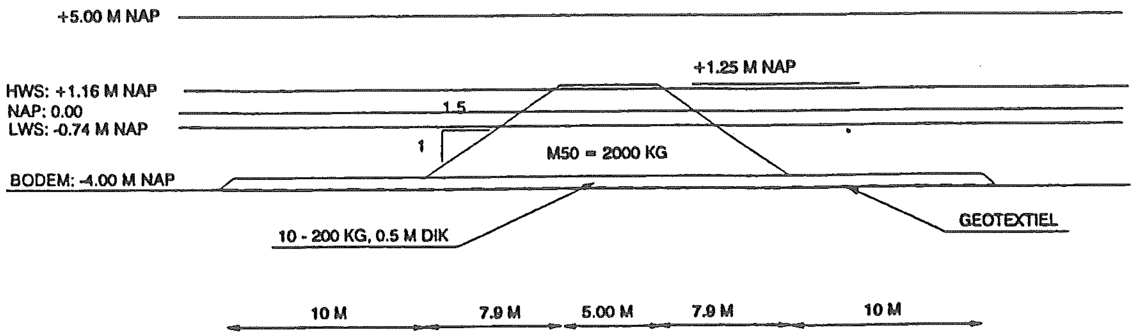

Figure 8.17: Cross section of the breakwater

8.4.5 *beach walls*

Text not yet available

8.5 *Effect on neighbouring coastlines*

8.5.1 *Sediment transport capacity*

Sediment transport formulae, like the Queens-formula, but also the Bijker approach in fact only describe the ability of the waves to transport sediment. If there will be really a transport of sand, depends also on the availability of sediment. When waves approach obliquely a rocky coastline, the braking of the waves will cause a sediment transport capacity, but because there is no sand available at the coast, the real sediment transport will be zero.

Example
Given is a straight stretch of coastline, as sketched in Figure 8.18 (I). The coast consists of

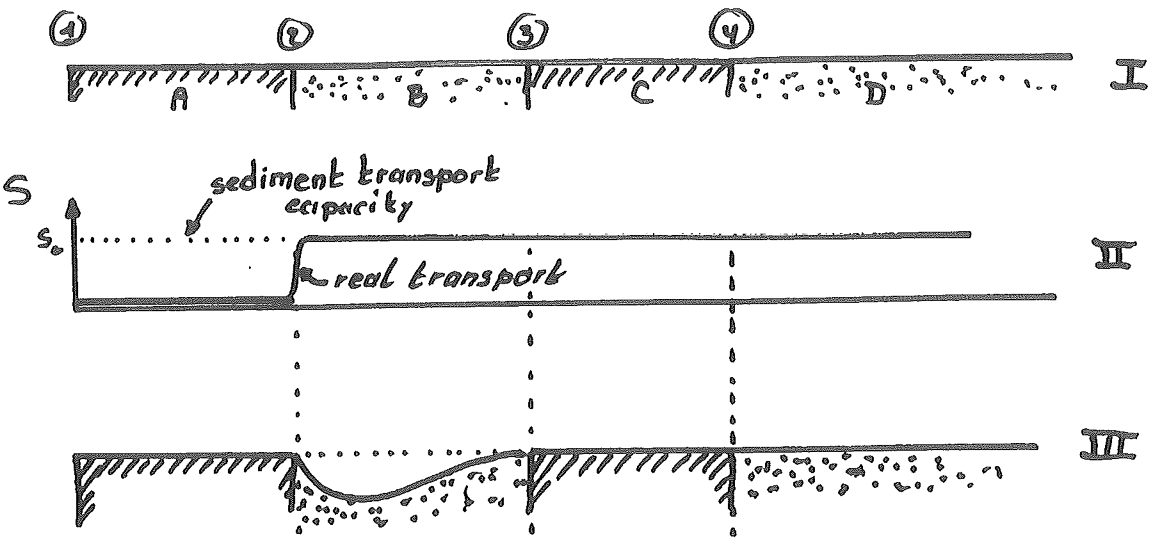

Figure 8.18: Sediment tranport and sediment transport capacity

two rocky sections, A and C, and in between sandy sections. Waves approach is under a given constant angel. So this implies that the sediment transport capacity (S_o) along the whole coast is constant. However, in section A there is no real sediment transport, there is no sediment available for transport . (One may say that the trucks available to move the sand drive from 1 to 2 drive empty). In front of section B sand is available. So the real sediment becomes equal to the sediment transport capacity. Because there is a change in real sediment transport (a gradient in transport) the effect is erosion of the coastline. (One may say that the trucks are loaded at the left side of section B, leaving an erosion gap on the beach).

At point 3 there is again a change in type of coast. But on both sides of point 3 the sediment transport capacity is equal.
Left of point 3 the sediment transport equals the sediment transport capacity. All this sand will also be transported to section C.
(One may say that the trucks, full with sediment move here from a sandy road onto a paved road, but this has not any change in the amount of sand transported by the trucks).

So consequently in front of section C there is a sand transport S_o, although a visitor at the coast does not see any sand transported. For a visitor there is no difference between section A and section C. However, in front of section A the transport is zero, while in front of section C the transport is S_o.

At point 4 there is again a change in type of coast. Like at point 3, the sediment transport capacity is equal at both sides. (The full trucks will move from the paved road onto a sandy road, but they are already full, so there will be no erosion).

Our coastal visitor does not see any erosion right point 4, which is different from the observation right of point 2, here significant erosion can be observed.

From the above example it becomes clear that is it very important to realise the difference between sediment transport capacity and the resulting real sediment transport. So, simply applying a formula does not give an answer on the question what is the resulting transport.

Also the geomorphology of the coastline has to be taken into consideration.

8.5.2 *Unexpected effects for the neighbours*

In some cases this may lead to very unexpected morphological changes. This can be explained with the following example.

Example

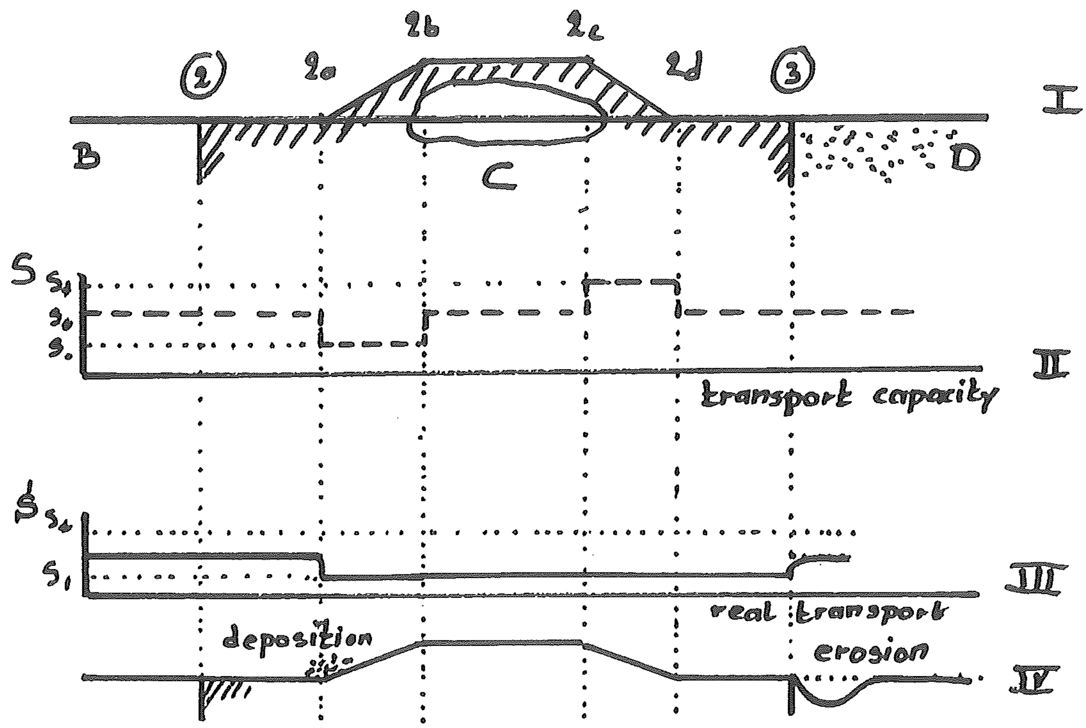

Figure 8.19: The effect of a construction

Suppose that a hotel-owner in section C (see previous example) wants to construct a swimming pool. Because space is limited, this swimming pool is constructed somewhat into the sea. (see Figure 8.19).

From the previous example we may conclude that along sections B and D sediment transport capacity is S_o. Because the sections c_1, c_3, and c_5 are parallel to the original section c, the sediment transport capacity is also S_o.
Because of the different orientation of the coastline, the sediment capacity in c_2 is <u>less</u> than S_o, and c_4, it is <u>more</u> than S_o (it is S_+). This is indicated in figure II (in other words, in section c_2 <u>less</u> trucks are driving, while in section c_4 <u>more</u> trucks are driving).

Let's now analyse the real transport.
In section c_1 sediment transport is S_o, as explained in the previous example. In point 2a suddenly the transport capacity decreases. (less trucks are available). Because the sediment transport capacity is only S-, no more sand than S- can be transported. So the supply of sand is more than the discharge of sand. Consequently, there will be a deposition of sand (those trucks not allowed to continue to stretch c_2 have to dump their sand). So just right of point

2a, one will observe deposition.

At point 2b there is an increase in sediment transport capacity (from S- to S_o). (The number of trucks increases again). But because there is no sand available, the real sediment transport in section c_3 will be S-.
(This means that the extra trucks will drive empty).
At point 2c the transport capacity increases again, but still no sand is available for feeding the transport capacity (more trucks, but when there is no sand available to fill them, also these extra trucks will drive empty).

At point 2d the sediment transport capacity decreases from S_+ to S_o. This is still more than the real transport (which is still S-). So potentially the coast could erode. However, there is no sand available, so also no erosion will occur, and consequently the real transport in section c_5 is S-.

At point 3 there is no change in sediment transport capacity. (It was S_o and stays S_o). But now the coast becomes erodible. This means that sand is available to load the trucks. The effect is that the coast starts to erode, just right of point 3.

If one looks to this example without knowing the differences between sediment transport and sediment transport capacity, it looks very strange. We start with a fully stable coastline B-C-D, build something at a rocky coast where no sand is observed at all, and consequently erosion starts some kilometers away from the construction site.

8.6 Artificial beach nourishment

why beach nourishment ?
For a good design, the purpose of the nourishment has to be clear. In general, there are three reasons for making a beach nourishment:
1. combat coastal erosion (chronic erosion)
2. prevention of flooding (safety)
3. making a wide recreational beach.

available tools
For the design of artificial beach nourishment, several tools are available. In general one can distinguish three kinds of tools:
1. mathematical models
2. measured data
3. physical models

Physical models are <u>not</u> recommended for the design of a beach nourishment. This is because the processes in changes of the beach are mainly caused by the irregularities in the wave conditions. It is very difficult to model that in a good way in a scale model.
If measured data are used, one does not have the problem of defining a good wave-climate, defining other boundary conditions, etc. They are automatically correct. The main problem is that for a good statistical analysis of measured data, one needs much data. Consequently one needs a great number of years with measurements, and very often these data are not available.
Mathematical models seems to overcome this problem. In fact they only change the problem. Now one needs good input data (waves, etc) and good calibration methods. And in order to

get them, one needs a great number of years with measurements!!

The Dutch Design Method

Beach nourishment in the Netherlands is designed in an extremely simple way. Experience has shown that this is a very reliable method. The method is very trustworthy, if applied.

- step 1: Perform coastal measurements (for at least 10 years).
- step 2: Calculate the "loss of sand" in m^3/year per coastal section.
- step 3: Add 40 % loss.
- step 4: Multiply this quantity with a convenient lifetime (for example five years).
- step 5: Put this quantity somewhere on the beach between the low-water-minus-1-meter line and the dune foot.

This method is simple and straightforward. It does not require mathematical models, but good quality profile measurements are absolutely necessary.

Problems

Of course, there are also problems with the Dutch Design Method. There is one very important general assumption:

> *The beach nourishment has no influence on the long-term natural behaviour of the coast. Or, in other words, the erosion rate before nourishment equals the erosion rate after nourishment.*

This general assumption is true in The Netherlands when beach nourishment is relatively long and the seaward displacement of the water line because of the nourishment is not too great. In The Netherlands the ratio between length of a nourishment (L) and seaward movement of the water line (width of the nourishment, w), L/w, is in the order of 20 - 40. Of course, one should realise that the Dutch coast is a coast with a tidal difference of 2 - 4 meters, a tidal current along the coastline and an almost perpendicular wave attack. Provided the L/w ratio is in the order of 20 or more, this assumption is valid in most areas of the world.

The erosion rate has to be calculated as a volume per unit of time (e.g. m^3/year per m of coastline) In cases were no profile data are available, one may also use the retreat of the high-waterline, but then one implicitly assumes that the coastal profile is also constant in time. That is usually not so. Apart from that the variation in the measured data is also significantly more if one uses coastline retreat instead of volume retreat.
So in fact the first step is to measure the coastal profile for a number of years, and calculate the volume in the profile (see Figure 8.20). It is important is to define good boundaries. The landward boundary has to be placed far enough landward that erosion (also storm-erosion) will "never" pass this boundary. "Never" has to be interpreted as: possibly not in the observation period and the extrapolation period. The offshore boundary has to be far enough into the sea, that no significant onshore/offshore transports take place over this boundary during normal situations. In any case one should try to place this boundary seaward of possible breaker-bars.

The next step is plotting the volume data as a function of time (see Figure 8.21a). Through the data in this figure one may draw a regression line. Usually a linear trend can be assumed, especially when the regression-period is in the order of ten years. This period is recommended for this type of analysis. In very special cases non-linear regression has to be applied. The slope of the regression-line indicates the erosion-rate, e.g. Q_m (m^3/year).

As next step a nice lifetime for a nourishment is selected, for example 5 years. One may se-

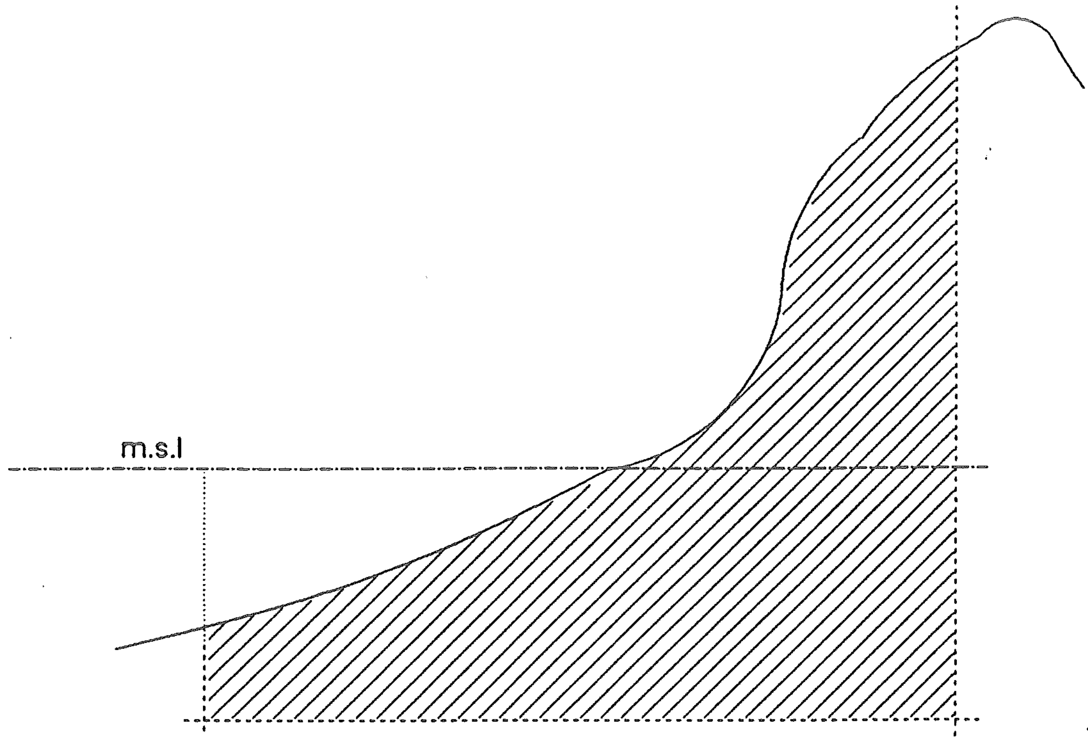

Figure 8.20: Control profile

lect any figure, and optimize this later on. Experience however, has shown that such an optimization is usually is completely overridden by non-technical issues, like available budget, available sand, execution schemes, etc. We call this lifetime T.
The volume to be nourished is thus $V_N = Q_m * T$ (m^3). (see b).
However "losses" in longshore direction are not considered. These "losses" occur, because the nourishment has always a limited length. Also there is a wash-out of finer particles (the grain-size distribution of the nourished sand is never equal to the original beach sand). Because the beach is somewhat further into the sea, the wave-attack is heavier, and, last-but-not-least there might be a profile adaptation outside the control volume. This last effect becomes less when we place our seaward boundary further out in a seaward direction.

For all these losses we should add something. A first estimate of this surcharge is 40%. This percentage covers all type of losses (see c). This percentage can be fine-tuned by using measured data from the evaluation of previous nourishments in the region. We found in the Netherlands that we could usually use a slightly lower percentage.

Placing the sand
From a morphological point of view there is not much preference where the sand is placed in the beach profile (provided it is between the breaker line and dune-foot or swash-line). Placement outside the breaker zone might be attractive in some cases, but that is outside the scope of this paper.
The first minor storm after the placement of the nourishment will adapt the profile to the natural profile. And nature can do this much better than bulldozers and scrapers.

Experience in the Netherlands has indicated that so-called profile nourishment (i.e. trying to make a naturally stable profile) does not have any influence on the erosion-rate. It was only found that at very steep, relatively fast eroding beaches, a high placement lasted somewhat longer, because the profile could not adapt quickly enough to the erosion. In fact, because of high nourishment the profile was constantly too steep, and the next nourishment was due before the profile could adapt.

From an economic point of view a placement just above H.W is preferable. Usually dumping from split-barges in the breaker zone cannot be realized, and one has to pump the sand ashore. The cheapest way is placing the discharge pipes just out of the reach of the waves, which implies a nourishment-level just above H.W. The seaward slope is formed by the free flowing sand. No bunds are used.

The consequence of this method is that just after the nourishment one has a beautiful, wide, high and dry beach, but with a seaward slope which is to steep to survive the stormy season.

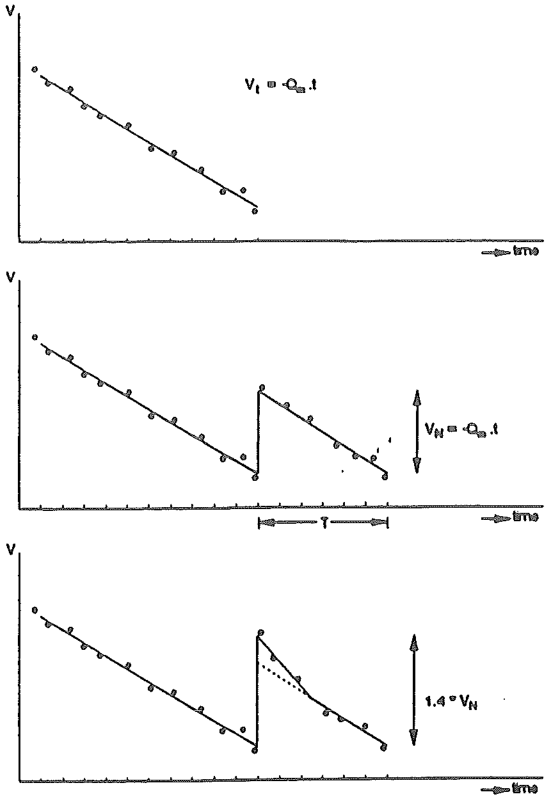

Figure 8.21: Principles of beach nourishment design

So, the first storm in autumn will rework the profile until it reaches its natural shape, and a lot of sand is transported from the H.W-line to just below L.W. From a morphological point of view this definitely "no erosion". In general the public has a different opinion. They see only the dry beach, and they see that it "disappears" after a minor storm. So they regard the nourishment as a failure.

Because of this psychological reason (for funding nourishments one needs public support), it is wise to make the shape of a nourished beach in such a way that the changes in autumn are not too big. In this way one will have more public support for nourishments.

The beach profile

It is very difficult to design the new beach profile. A very good assumption is that the profile after the nourishment will eventually be the same as before the nourishment, provided the same type of sediment has been used.
Nature will form that profile. Therefore it does not matter very much where the sand is placed in the profile. After one or two small storms the complete profile is reworked by nature and the natural (stable) profile is formed. From this one may conclude that one should dump the sand where dumping is the cheapest, as long as it is landward of the breaker line. In The Netherlands, the cheapest way is placing the sand on the beach, preferably on the higher section. All discharge pipes can be placed out of reach of the waves, and after the nourishment a beautiful, wide beach is formed. However, because the slope just under the low water line is too steep, the first storm in autumn transports sand from the beach towards the underwater shore. From a morphological point of view, this is no problem. From an economical point of view, this is the optimal solution. A number of nourishment projects in the Netherlands have been designed in this way.
However, from a political point of view this is not a good solution. The public has a

beautiful beach in the summer, directly after the nourishment. But in autumn, during a minor storm, the public observes that the beach largely disappears. They do not observe that the sand is deposited just below the low water line. The public draws the incorrect conclusion that the nourishment was not successful at all. The wide beach has disappeared.
In Australia (Gold Coast) the beach front is relatively steep. There it is possible to bring the sand by barge very near to the beach an dump it directly from the barge (using a boom) in the breaker zone. The beach itself is not influenced directly by this operation. After a storm this sand is also reworked by the waves and the profile is adjusted. In this case sand comes to the beach, and theoretically the beach will become wider. However, because during storms the coastal erosion also takes place, in general the loss by storm-erosion will be compensated by the beach growth from the depot in the breaker zone. Consequently, the storm has no effect on the size of the beach. From a morphological and economical viewpoint this nourishment is successful.
But here these is also a political problem. Because nothing happens, from the viewpoint of the public, they think that this nourishment was not necessary at all.

Because beach nourishment is generally paid for by a public authority, public opinion is important in acquiring sufficient funds. Therefore it is wise to design beach nourishment in such a way that the public sees that the beach is somewhat wider after the nourishment, but that there is no major adaption in the beach shape during the first storms in autumn. If the purpose of the nourishment is to make a wide recreational beach, this is very important. If the purpose is to prevent flooding, the best place is as high as possible on the beach. If the purpose is to combat chronic erosion, the best place is in the breaker zone.

Sediment size
The size of the sediment is important. If the sediment is finer that the original material, the equilibrium slope will be more gentle. Some researchers also assume that finer material causes bigger losses. However it is not clear whether these bigger losses are caused by the more gentle beach profile or whether they are real losses.
In order to get an impression of the effect of the grain size on the beach slope, one may use a general diagram as made by Dalrymple and Thompson [1976].
However, many beaches have slopes of 1:50 or less, and they are not in this diagram. Also the scatter of the data is too wide for practical application.
It is therefore recommended to start from the existing beach slope and use scale relations to derive the change in beach slope. These scale-relations have been developed by Vellinga [1982] and used for calculating the overfill ratio by Pilarczyk, Van Overeem and Bakker [1986].
The formula to be used is $(l_1/l_2) = (w_1/w_2)^{-0.56}$. In this formula l is a characteristic length and w is the fall-velocity of the beach material. The fall-velocity can be computed using:

$$^{10}\log(1/w) = 0.476\ (^{10}\log D)^2 + 2.180\ ^{10}\log D + 3.226$$

in which D is the median grain-size.

Example: See also Figure 8.22. Suppose the original beach has, at a certain depth a slope of 1:75. Then the value $l_1 = 75$. The original grainsize is 275 micron. Then $w_1 = 0.038$ m/s. When the grain-size of the nourishment is 225 micron, then $w_2 = 0.034$ m/s. Thus l_1/l_2 is 1.064, and consequently l_2 is 79.82. This means that at that depth the new slope will be 1:79.82.

This effect causes an so-called overfill ratio. Suppose the design nourishment width was 100 m, and the nourishment height is 5 m. Then the volume to be nourished is 5 x 100 = 500 m². Because the more gentle slope an extra volume of ½ x 5 x 5x(79.8-75) = 60 m² is necessary. This is an overfill of 12%.

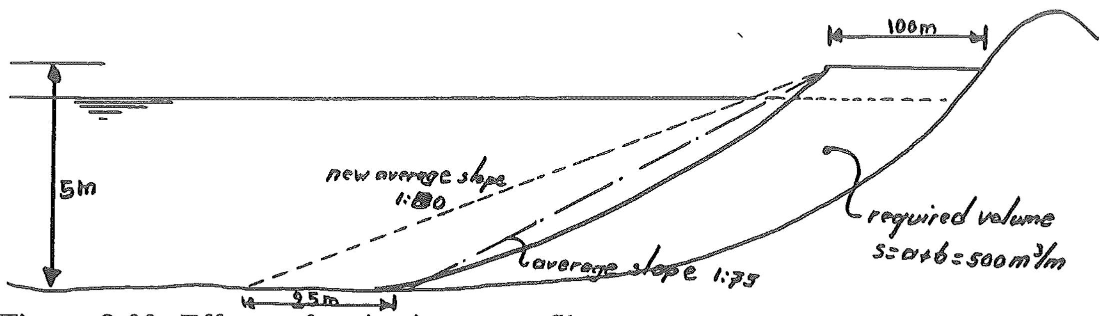

Figure 8.22: Effenct of grain size on profile steepness

If one applies the technique of James, as presented in the US Shore Protection Manual, which is based upon the sorting out of fine particles, in this example one finds a overfill ratio of
20%. Experience in Holland shows that the SPM method gives relatively high overfill ratios. Also it is expected that the mechanism of sorting out is not the governing mechanism, but only the fact that a more gentle slope will occur.
However, detailed research in this field, based upon a good set of prototype data is not yet available.

Conclusion
From the above one can conclude that the proposed design method for artificial beach nourishment is simple, straight forward and very reliable. No advanced models are required. The disadvantage is that beach profile data have to be available. Because of this, it is always good to have a good beach monitoring programme (profile measurements to be made once a year, at fixed profiles).

In the Netherlands all nourishments have in fact been designed using this method, except one. That nourishment was for an artificial peninsula, where (of course) no beach data were available. In that case mathematical models were used.

Directly after the nourishment, the shape of the beach is not optimal. Nature will adapt the shape of the nourishment. Also, because the nourishment protrudes into the sea, the current attack will be more. So, some extra loss has to be expected. It is difficult to calculate this loss exactly. Our experience shows that surcharge of 40 % on the designed quantity covers all losses, including the loss due of the extra current attack. A more mathematical approach to determining this loss is presented by Führböter [1991], although his assumption that an initial loss rate is proportional to the volume of sand available on the beach might not always be true.

The costs of beach nourishment
The costs of previous nourishments in the Netherlands varied greatly with the borrow site of the sand for the nourishment. Besides the borrow area, the way of determining the price is also of importance. Three scenarios are used to determine the cost of sand winning.

1 Present borrow areas nearer to the coastline
2 Alternative borrow sites, based on newest morphological knowledge
3 Present borrow areas, using a high calculation value

In the table can be seen that the price (including all taxes) varies from US $ 1.50 per m^3, if sand can be borrowed from areas near the coast, up to US $ 5.35, if the borrow area is far away (at a depth of 20 m, 20 km offshore) and a high calculation value is used.

scenario for sand winning price in US $ per m^3	wadden area	coast of Holland	coast of Zeeland
present borrow areas	1.50	4.80	2.40
alternative borrow areas	2.25	2.40	2.40
present borrow areas and a high calculation value	2.90	5.35	3.20

Table: price of sand

Dune nourishment is US $ 2.00 per m^3 more expensive. In determining these prices market effects are not taken into account. Because of overproduction the dredging prices on the world market are lower at this moment, but for forming the basis of a long term policy, this should not be taken into account.

8.6.1 *Sedimentation in basins*

In contradiction to sand, transport, erosion and accretion due to fine particles, like mud, does not depend very much on the local parameters. This is due to the fact that it takes quite a long time before the fine particles settle down. Because large parts of the Indonesian waters are along muddy coasts, this is a rather important point. In general one may state that sea-water contains a certain concentration of mud. This concentration depends on the supply of silt-laden wter from the rivers and the ability of the waves to keep this silt into suspension. Therefore, as long as there is wave action, only a small fraction of the fine particles will start to settle downs.

However, as soon as this water comes into a protected area (for example in a harbour basin), there are no waves any more to keep the sand in suspension and the settling process will start. In order to make an estimate of the amount of siltation in a harbour basin, one can calculate the total amount of "fresh" silt-laden water coming into the harbour every tide (= the tidal prism). Multiplying this quantity of water with the concentration of silt in the sea in front of the harbour gives the amount of silt which enters every tide into the harbour.

However, the quantity of water which had entered the harbour also has to leave the harbour. But the concentration of sediment in the water leaving the harbour is much lower. Multiplying the amount of water leaving the harbour with the concentration in "clean" water gives the total amount of sediment leaving the harbour. The difference stays in the harbour and causes the harbour siltation.

This process can be illustrated using the following example:

Given a harbour:

lenght	L (m)
width	W (m)
depth	d (m)

At sea, there is a tide with a tidal difference of Td (m) and a tidal period of Tp hrs (for example 12hrs, 25 min)

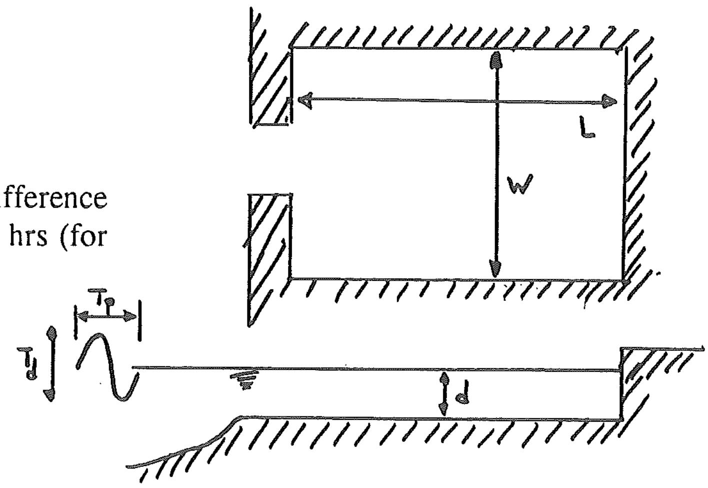

Figure 8.23: Definitions

Density

seawater	ρ_w (kg/m^3)
sediment	ρ_s (kg/m^3)
bottom mud	ρ_m (kg/m^3)

Concentration of sediment in:

outside water	c_{sea} (g/l = kg/m^3)
"clear" water	c_{clean} (g/l = kg/m^3)

Each tide Td * L * w (m^3) of water comes in and goes out of the harbour. The concentration of the incoming water is c_{sea}, of the outgoing water is c_{clean}. Consequently eacht tide:

$$S_0 = [c_{sea} - c_{clean}] * Td * L * d \qquad [kg] \tag{8.6}$$

stays in the harbour.

One m^3 of mud weights:

$$1 * \rho_{mud} = \nu_v * \rho_w + (1 - \nu_v) * \rho_s \tag{8.7}$$

Therefore the void ratio is:

$$v_v = \frac{\rho_s - \rho_w}{\rho_s - \rho_m} \tag{8.8}$$

So, 1 m^3 o mud contains $(1-v_v)*\rho_s$ kg of sediment.
Therefor, 1 kg of sediment gives:

$$\frac{1}{(1-v_v)\rho_s} \quad m^3 \quad \textit{of mud} \tag{8.9}$$

Multiply (1) with (4). This is the amount of m^3 that settles each tide in the harbour, expressin in m^3/tide. We call this value s1:

$$s_1 = S_0 \frac{1}{(1-v_v)\rho_s} \tag{8.10}$$

This is:

$$S_2 = S_1 * \frac{24}{T_p} * 365 \qquad [m^3/year] \tag{8.11}$$

This is:

$$S_3 = \frac{S_2}{(L*W)} \tag{8.12}$$

The parameters, usually unknown in this computation are c_{sea}, c_{clean} and ρ_{mud}.

For Rotterdam c_{sea} is 50 - 100 mg/l
c_{clean} is 10 mg/l
ρ_{mud} is 1200 kg/m^3

For the north coast of Java, during dry season c_{sea} is 75 - 100 mg/l, for the rainy season c_{sea} is 300 - 400 mg/l.

8.6.2 Sedimentation in mangrove areas

8.7 Subsidence problems

8.8 Sediment and ecosystems

8.8.1 Coral reefs

8.9 Other human effects

8.9.1 Mangrove (mis-)management

8.9.2 Coral reefs

8.9.3 Fish ponds

8.10 References

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9 Structural design

9.1 Introduction

Figure 9.1 shows some ways to defend an erodible coast. Artificial sand supply (example A) is an interesting and sometimes cheap solution, where the "natural" look of a coast can be preserved.

Detached breakwaters and groynes (B and C) are both meant to influence the coastal sediment transport. Groynes are only effective in longshore transport, while detached breakwaters (with a tombolo as the ultimate stage) can be effective both along and perpendicular to the coast. When they are stretched along the whole coast, they act mainly as a wave reductor. The structural aspects will be mentioned in this chapter, while the morphological effects are dealt with in chapter 8.

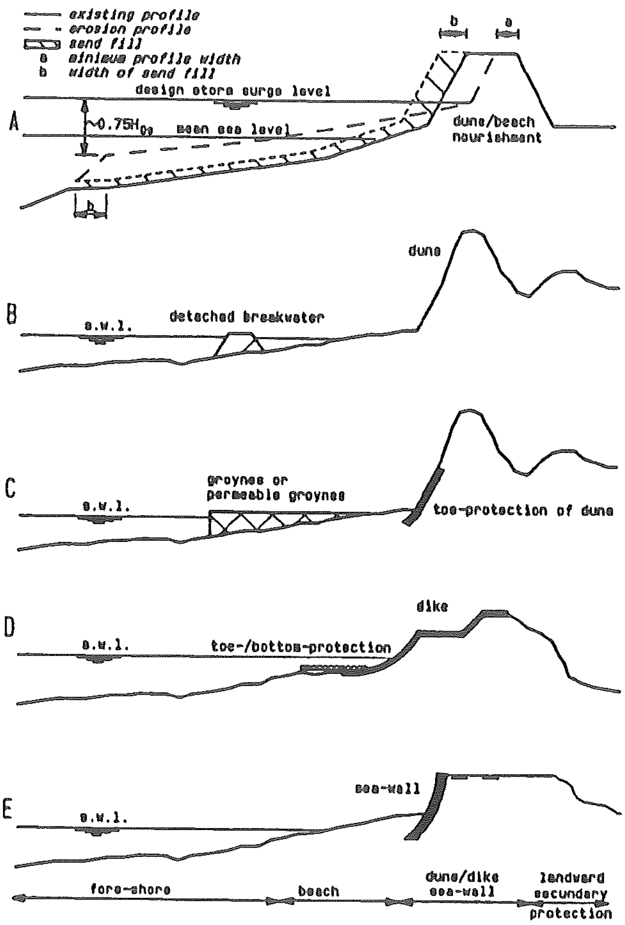


Figure 9.1 Examples of shore protection (Pilarczyk,1990)

Dikes, revetments and seawalls are really artificial coasts. They are much alike, but a seawall is usually made when there is little space, e.g in an urban environment. The seawall itself (made of concrete or brick) will not be elaborated here. In front of the seawall, a bottom protection is usually necessary, see section 9.8.1.

First some basic loading and stability mechanisms will be treated, due to flow, porous flow and waves respectively, while the soil mechanical aspects will be given in section 9.5. Next, the above mentioned types of coastal protection (revetments, breakwaters, groynes and seawalls) will be treated.

9.2 *Loads by flow*

9.2.1 *Stability*

Probably the most well known relation for uniform flow is the one by Shields from 1936. Shields gives a relation between a dimensionless shear-stress and the so-called particle Reynolds-number:

$$\psi = \frac{\tau_c}{(\rho_s - \rho_w) g d} = \frac{u_{*c}^2}{\Delta g d} = f(\Re e_*) = f\left(\frac{u_{*c} d}{\nu}\right) \tag{9.1}$$

Shields chose the shear stress as the active force. Note that the grain diameter and the shear velocity appear on both sides of the formula.

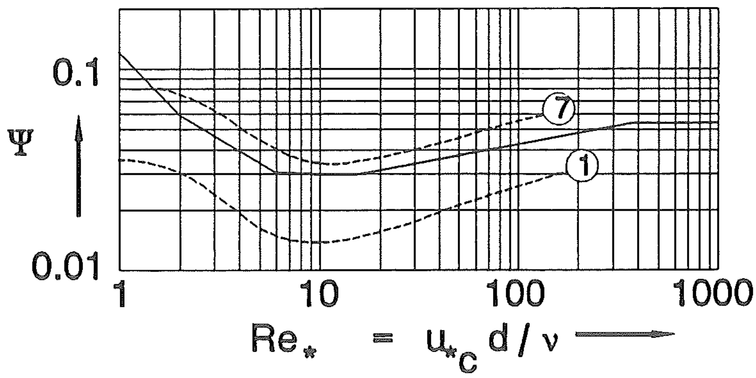


Figure 9.2 Critical shear stress according to Shields with experimental data for various criteria of stability

For high $\Re e_*$-numbers, Ψ becomes constant with a value of about 0.055, see Figure 9.2. In practice this is already the case for stones with a diameter of about 5 mm. Shields found his values by extrapolating a measured transport of material to zero. The value 0.055 for Ψ does not mean that no single grain is moved. In fact there does not exist such a thing as a critical velocity! The idea of a single critical velocity cannot be true because of the following:

- When the material is graded, the small particles can be taken away with the current, while the large ones remain, armouring the rest of the bed.
- The flow itself is turbulent, which means that peak pressures and velocities can be much higher than the average value. In theory there is no limit.
- The threshold of movement is a subjective matter when judged in an experiment; is it the first loosely lying stone that moves 1 cm and then stops again or should grains be moving all over. In the Shields graph (Figure 9.2), also the results of an investigation into the threshold of motion (DHL,1969) are given. 7 stages of transport were discerned. The first, (1), described as "every now and then movement of a single grain" and the seventh, (7), as "start of march of the grains". It appeared that the Shields criterion fits rather well stage 6: "movement of grains everywhere and permanently".

After what was said above, the Shields criterion can be seen as the start of sediment transport or erosion (Shields performed his research to find the beginning of sediment transport). For a first design it is still useful to work with a critical velocity or shear stress, but for the design of a protection, a lower value of Ψ should be taken. It was mentioned that Shields found $\Psi = 0.055$ for high $\Re_*$-numbers (large grains). $\Psi = 0.03$ is often recommended for aprons and the like, based on criterion 1, see Figure 9.2. Even then, some transport will occur and regular maintenance will be necessary.

Roughness

With uniform flow relations equation (9.1) can be rewritten: u_* (the shear velocity) = $\bar{u}\sqrt{g}/C$, where $\bar{u}$ is the velocity averaged over the vertical or the cross-section. This shows a dependency of the critical velocity on the waterdepth since C increases, given a bottom material, with the waterdepth: $C = 18\log 12h/k$. k is the equivalent sand roughness. Unfortunately, this is another point of uncertainty in the idea of a threshold of motion. k depends on the gradation and on the stacking of the stones, which is in turn determined by the way the bed is constructed. For a bed that is made under favourable conditions, e.g. on a dry and visible underground, $k \approx 2d_{50}$ seems achievable; $k \approx 3d_{90}$ is often used for less favourable situations, also depending on the grading of the material (the larger stones probably influence the roughness more than the median value). For the time being, $k \approx 3d_{90}$ seems a reasonable choice when no certainty about the quality of the construction can be ascertained.

Slopes

The Shields relation is valid for stones on a horizontal bed. A factor that influences the stability is the slope of the bed. When the slope is equal to the angle of repose, a stone is already on the threshold of motion and any load will induce movement.Figure 9.3e gives an idea of the value of ϕ for various materials. When the bed is between horizontal and the angle of repose, the diameter calculated with equation (9.1) needs a correction.
To determine the influence of a slope on the stability, it is assumed that for a stone on a flat bed the critical flow force can be expressed as $F(0) = W\tan\phi$, see Figure 9.3a. This would mean that the friction factor of a stone on a horizontal bed is equal to the tangent of the angle of repose, which will depend strongly on the protrusion of the grains. When This is not necessarily a problem, the slope correction factor is only used relatively.
When the flow is in the downstream direction of a slope, see Figure 9.3b, two factors have a negative influence on the stability. The component of the weight of a grain along the slope ($W\sin\alpha$) works with the flow, while the effective weight perpendicular to the slope becomes less ($\cos\alpha . W\tan\phi$). The critical flow force then becomes: $F(\alpha) = W\cos\alpha\tan\phi - W\sin\alpha$. (Note: when the flow is reverse, the grain gets more stable and $F(\alpha) = W\cos\alpha\tan\phi + W\sin\alpha$). The relation between $F(\alpha)$ and $F(0)$ now gives the reduction to be applied on the shear stress:

$$K(\alpha) = \frac{F(\alpha)}{F(0)} = \frac{W\cos\alpha\tan\phi - W\sin\alpha}{W\tan\phi} = \frac{\sin(\phi - \alpha)}{\sin\phi} \tag{9.2}$$

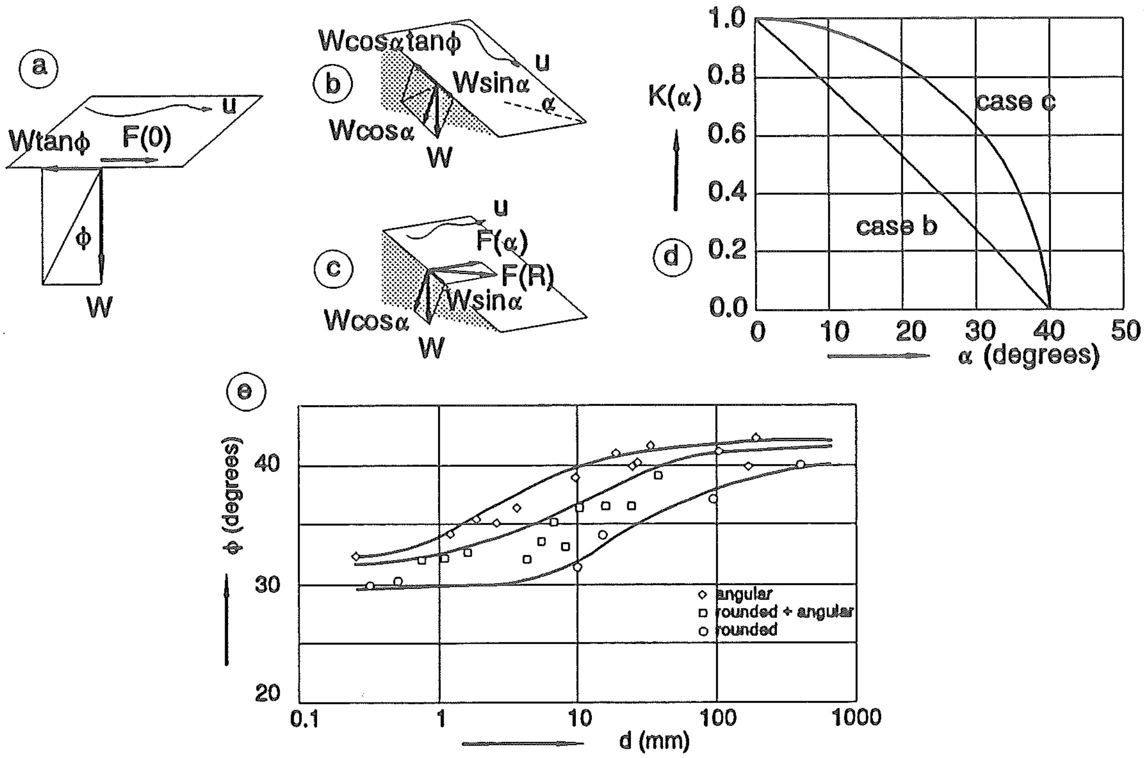


Figure 9.3 Influence of slope on stability and angle of repose for non-cohesive materials (Simons,1957)

For a side slope with angle α, the same procedure can be followed, see Figure 9.3c: $F(\alpha)^2 = F(R)^2 - (W\sin\alpha)^2 = (W\cos\alpha\tan\phi)^2 - (W\sin\alpha)^2$, leading to:

$$K(\alpha) = \frac{F(\alpha)}{F(0)} = \sqrt{\frac{\cos^2\alpha\tan^2\phi - \sin^2\alpha}{\tan^2\phi}} = \cos\alpha\sqrt{1 - \frac{\tan^2\alpha}{\tan^2\phi}} = \sqrt{1 - \frac{\sin^2\alpha}{\sin^2\phi}} \tag{9.3}$$

In Figure 9.3d, both the reduction factors $K(\alpha)$, for a slope in flow direction and for a slope perpendicular to the flow, are drawn for the case that $\phi = 40°$. It is clear that a slope in the flow direction is much more unfavourable. $K(\alpha)$ is a correction factor for the diameter and represents a reduction of the strength, hence $K(\alpha) < 1$.

Turbulence

Figure 9.2 is based on uniform flow investigations in a rectangular laboratory flume. When the flow is not uniform, the velocity in the stability relation has to be multiplied with a factor K(v). In open channel flow in river mouths and seas K(v) is usually no more than 1.1 to 1.2. In mixing layers, in strongly decelerating flow, K(v) can be around 1.5 applied to the local vertically averaged velocity. K(v) is a correction factor for the velocity and represents a load increase, hence $K(v) > 1$. Applied to the necessary stone diameter, $K(v)^2$ must be used.

9.2.2 Scour

Figure 9.4A shows a beautiful picture (from Breusers/Raudkivi,1991) of the flow around a cylinder. There is a downflow in front of the cylinder, due to the difference in pressure, which is in turn due to the difference in velocity. This downflow can reach a value of 80% of the undisturbed flow velocity and acts as a vertical jet. This jet together with the accelerated flow at the sides of the pier, start the erosion. Once there is an erosion hole, a circulating current in the scour hole appears. This eddy is taken away by the flow, leading to a horseshoe-shaped vortex. This vortex, together with the accelerated flow and wake vortices transport the sediment further downstream. Behind the pier, the transport capacity decreases again and a bar is formed.

Figure 9.4B shows the equilibrium depth (dimensionless with the pier diameter) with increasing velocity. When $u < u_c$, there is already a distinctive scour due to the described flow factors. The scour reaches a maximum when $u = u_c$; when $u > u_c$ it decreases again because then the whole bed is moving and the scour is reduced. The dotted line indicates the way this velocity function is approached in the final scour formula. This is the transition between clear-water and live-bed scour. A second peak can occur at the transition between ripples/dunes and anti-dunes where the bed is flat. Then, the flow does not need to spend energy in form drag and can devote itself completely to sediment transport. The second peak is lower than the first; for a preliminary design the first peak is recommended. Figure 9.4C finally shows the difference in scour development in time between clear-water and live-bed scour.

For slender piles, it is found in experiments that the scour depth increases when the pile diameter increases. This can probably be explained with the downflow, which is expected to grow when a larger width halts the flow and the vortices, which also grow as the width of the wake behind the cylinder grows. In experiments indeed $h_s \propto D$ is found, with a constant of proportionality ~ 1.5, (Breusers et al,1977). It can also be expected that both influences, sketched above, cannot go on growing when the diameter of the pile gets the order of magnitude of the waterdepth or more. The downflow will be limited when the pile acts as a complete "wall" and the size of the vortices will be limited by the waterdepth. Think of a "pile" with a diameter of 100 m in 10 m waterdepth (e.g. an offshore oil storage device) or even larger, e.g. an island. A direct relation between scour depth and diameter seems unlogic with such large diamters; the flow process around one edge of the structure will hardly be related to what happens round the other edge. A relation with the waterdepth seems more appropriate, which is also found in experiments.

Figure 9.4D and E shows the range of experimental results as given by Breusers et al,1977. A mathematical description of the whole range of the situations sketched above can be given by means of a hyperbolic tangent of the dimensionless parameters h_s/D and h_0/D. For large values of h_0/D (slender piles in deep water), $\tanh(h_0/D)$ becomes 1, giving $h_s \propto D$. For small values of h_0/D (large bodies in shallow water), $\tanh(h_0/D)$ becomes h_0/D, giving $h_s \propto h_0$. A constant of proportionality of 1.5 seems to describe the experimental results reasonably well, although Breusers et al,1977 recommend 2 for design purposes.
Note: The tanh function has no physical basis. It is just a function with convenient mathematical properties to describe the above mentioned relations.

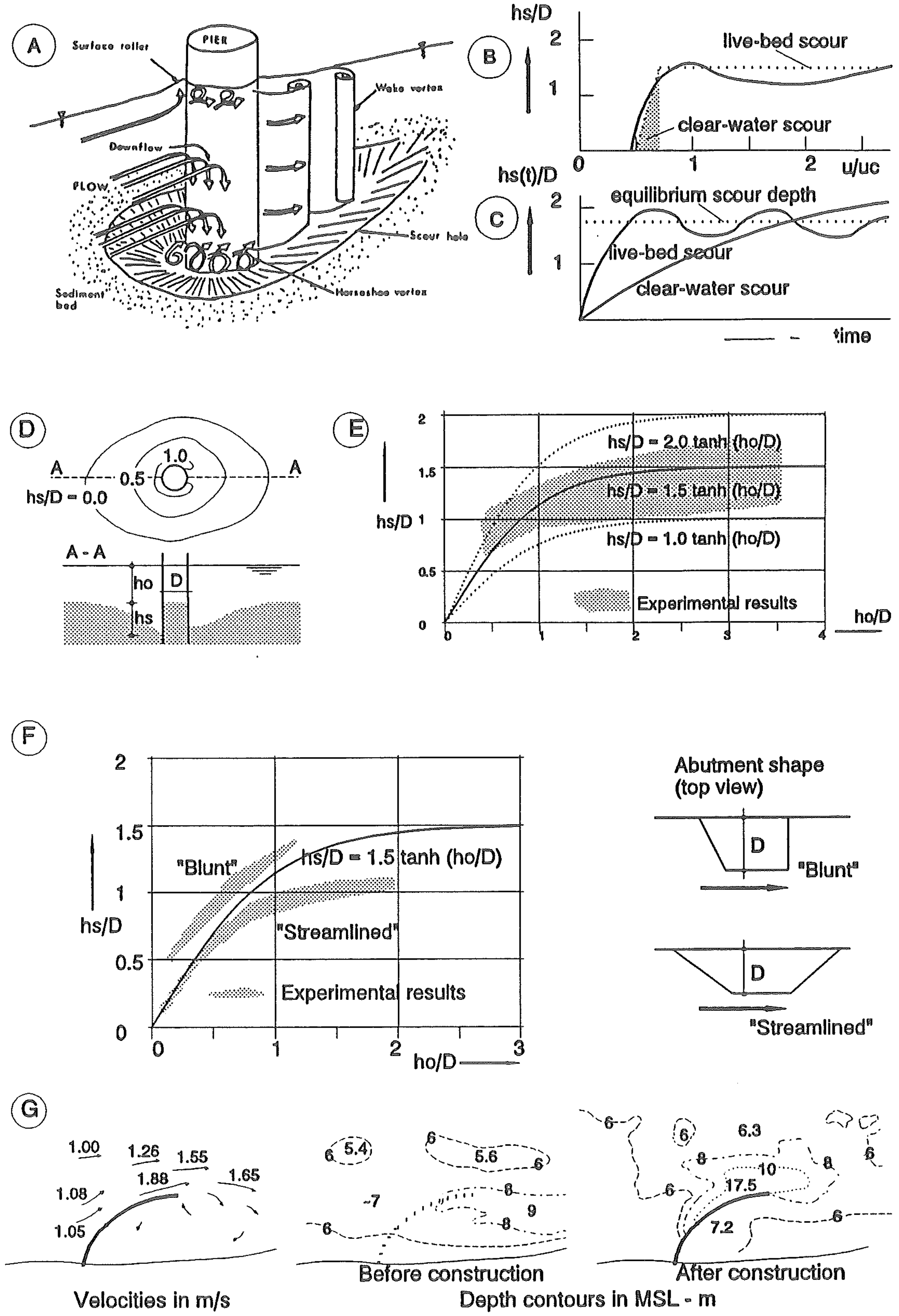


Figure 9.4 Scour phenomena

Figure 9.4E is valid for cylindrically shaped piles. For other shapes, the table gives values for a shape factor.

For a first design the next formula is proposed:

$$\frac{h_s}{D} = 1.5\ K_S\ K_V \tanh\left(\frac{h_0}{D}\right) \qquad (9.4)$$

in which:
K_S = shape factor, see table
K_V = velocity factor, see Figure 9.4B:

Shape factors for various pier shapes

Pier shape	l/b	K_S
cylinder	-	1.0
rectangular	1	1.2
	3	1.1
	5	1.0
elliptic	2	0.85
	3	0.8
	5	0.6

$K_V = 0$ for $u/u_c < 0.5$, $K_V = 1$ for $u/u_c > 1$ and $K_V = (2u/u_c - 1)$ for $0.5 < u/u_c < 1$

Note: - wave action around the pier will hardly increase the scour depth (max. ~ 10%)

Abutments

The first approach to abutment scour is simply to consider it to be a half pier. Indeed, the processes are very similar, but the numerical values are somewhat lower since the wall hinders the downflow and the vortices. Streamlining is relevant. Figure 9.4F shows the relation between scour depth, abutment width (D, the protrusion from the bank into the water) and waterdepth. Hatched area 1 was found experimentally for streamlined abutment shapes like wing walls. Hatched area 2 was found for more blunt forms. Note that the width of the flow is implicitely assumed infinite; narrowing of the flow width by the abutment does not play a role. This situation is encountered at a coast where the width of the sea can indeed be considered infinite.

Figure 9.4G shows the flow pattern and the erosion after the construction of the Zeebrugge breakwater in Belgium, which can be seen as an enormous abutment (D ≈ 1500m). The velocity is increased with a factor, a little less than 2, which is in line with what could be expected according to potential flow. The scour depth with regard to the original bottom is about 17.5-7 = 10 m, about 1.5 times the waterdepth, which is surprisingly in accordance with equation (9.4). So for a first approach, these equations are useful, despite the physical shortcomings.

9.3 Loads by porous flow

9.3.1 General

Porous flow is the expression used for flow through a granular medium, like sand, pebbles or stones. The loads due to porous flow often come from the soil side of the interface soil-water. Like in open channel flow, pressures and velocities are related and both can be considered as loads.

Common practice is to combine all square inertia and turbulence terms in the basic equations to one quadratic friction term and to replace the (linear) viscous gradient with a linear friction term. The result is (van Gent,1992 and van Gent,1993):

$$\frac{1}{\rho g}\frac{\partial p}{\partial x} = i = a\,u_f + b\,u_f|u_f| + c\frac{\partial u_f}{\partial t} \qquad \textit{with:} \qquad a = \alpha\frac{(1-n)^2}{n^3}\frac{\upsilon}{g d_{n50}^2} \qquad b = \beta\frac{(1-n)}{n^3}\frac{1}{g d_{n50}} \tag{9.5}$$

For stationary flow $\partial u_f/\partial t = 0$, this is the classical Forchheimer equation. The influence of the third term is usually small and can be neglected in most cases. In non-stationary, oscillatory flow, the coefficient b in the second term has to be corrected with a factor depending on the wave cycle (van Gent,1993). The expressions for the dimensional coefficients a (s/m) and b (s^2/m^2) have been derived theoretically, while the dimensionless coefficients α and β (depending, among other things, on the grading and the shape of the grains) have to be determined experimentally. Without any further information, $\alpha \approx 1000$ and $\beta \approx 1.1$ can be used as a first estimate (with possible values twice as low or high). For more accurate values, a and b can be measured in a laboratory or field test.
Note: In many formulae for the permeability of materials d_{n15} is used instead of d_{n50}. Since the permeability is governed by the smaller grains, the first would be preferable, but in fact, any characteristic length-scale in the granular material can be used. When using d_{n15}, the coefficients α and β become proportionally smaller.

The relation between velocity and pressure gradient is sometimes written as:

$$u_f = k\,(i)^{\frac{1}{p}} \tag{9.6}$$

in which k is the permeability (m/s) of the porous material. For laminar flow, p = 1, the Forchheimer equation reduces to Darcy's law and k is the inverse of a in equation (9.5). In the next section, only solutions will be given for the case of laminar flow.
The following table gives an indication of the permeability, k as defined in equation (9.6), of some often used materials:

MATERIAL	d_{50} (< 5.10^{-2} m) or d_{n50} (m)	PERMEABILITY, k, (m/s)	CHARACTER OF FLOW
Clay	10^{-7} - 5.10^{-5}	10^{-12} - 10^{-8}	laminar
Fine sand	5.10^{-5} - 5.10^{-4}	10^{-8} - 10^{-5}	laminar
Coarse sand	5.10^{-4} - 2.10^{-3}	10^{-5} - 10^{-2}	laminar
Fine gravel	2.10^{-3} - 5.10^{-2}	10^{-2} - 10^{-1}	transition
Coarse gravel	5.10^{-2} - 10^{-1}	10^{-1} - 2.10^{-1}	turbulent
Small rock	2.10^{-1} - 4.10^{-1}	3.10^{-1} - 4.10^{-1}	turbulent
Large rock	5.10^{-1} - 2	5.10^{-1} - 1	turbulent

Laminar flow

Instead of pressures it is often more convenient to use heads in Darcy's law:

$$h = z + \frac{p}{\rho_w g} \tag{9.7}$$

where h is the piezometric head or potential and z is the elevation above a reference level. The Darcy relations for two dimensions now read:

$$u_f = -k_x \frac{\partial h}{\partial x} \qquad w_f = -k_z \frac{\partial h}{\partial z} \tag{9.8}$$

These equations can be solved, but only for simple geometries, analytical, approximate solutions exist. For more complex situations, the equations can be solved numerically (either by finite elements or finite differences) or graphically (with a rectangular flow net pattern). Numerical solutions are easily available nowadays, but for a first idea, a sketch of a graphical solution can be useful.

In the following two examples of stability and erosion in porous flow, that are relevant for the situation in Indonesia, will be treated. Because of heavy rainfall, the phreatic level in a coastal embankment will usually be higher than the sea or river mouth level. The result is either an excess pressure against a closed revetment or an outflow through the soil. In the first case, pressure is the dominant factor in stability, while in the second case the forces due to the flow velocity play the dominant role in stability of a slope.

9.3.2 *Impervious slope protections*

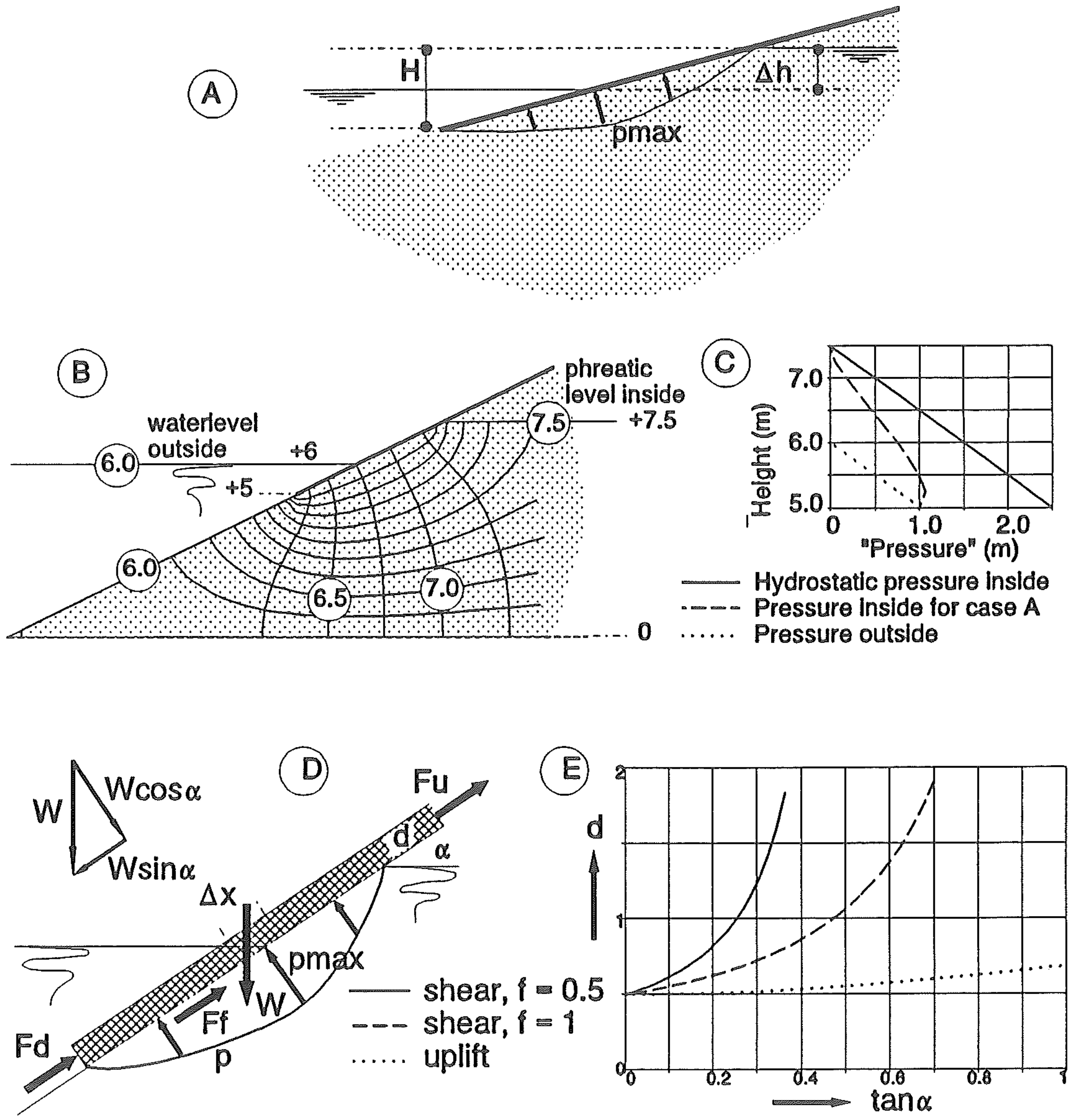


Figure 9.5 Pressures and stability for an impervious layer on a slope

Figure 9.5A gives an impervious revetment with a head difference from land to sea. For $\Delta h/H < 0.7 - 0.8$, the maximum excess pressure under a protection of negligible thickness, with regard to the waterlevel on the slope, is given in first approximation by:

$$p_{max} = \left(1 - \frac{\Delta h}{H}\right) . \rho_w g \Delta h \tag{9.9}$$

For a more detailed design, the use of a numerical model is recommended.

Figure 9.5B and C show the flow net and pressures for a slope of 1:2 with an impervious layer reaching 1 m below the outside waterlevel and a head difference from inside of 1.5 m. Figure 9.5B shows the potential lines and streamlines as determined with a numerical model. Figure 9.5C gives the pressures directly under the protection, compared with the hydrostatic values (simply the pressure due to the waterdepth). The values under the protection are lower due to the open slope below the protection. This pressure reduction due to the limited length of the protection can lead to piping at the toe! The excess pressures follow from the difference between the pressure inside and outside. Now also the influence of the layer thickness can be seen: only for a protection of negligible thickness, the excess pressure at the toe if the protection is 0. The maximum excess pressure occurs at the outside waterlevel and is, according to MSEEP, about 0.7 m water, while equation (9.9) gives: (1-1.5/2.5)*1.5 = 0.6 m. **Note:** The head difference, Δh, in a non-stationary situation should be determined with a numerical model. A first estimate can be 50% of the difference between high water and the average level.

Figure 9.5D shows the forces that play a role in the equilibrium of an impervious layer on a slope. The protection is loaded with the water pressure, p, due to the head difference, Δh. The maximum excess pressure, p_{max}, can be determined, either with equation (9.9), an electric analogon or with a numerical model. The weight, W, must counterbalance this uplifting load, either via the shear force between the layer and the slope or directly by the weight itself.

Shear

The friction between protection and subsoil must be large enough to balance the component of the weight parallel to the slope. The friction depends on the effective weight perpendicular to the slope. Stability for a length of slope Δx just below the outside waterlevel is ensured when:

$$f\,[\,(\rho_m - \rho_w)\,g\,d\,\Delta x \cos\alpha \;-\; p_{\max}\,\Delta x\,] \;\geq\; (\rho_m - \rho_w)\,g\,d\,\Delta x \sin\alpha \tag{9.10}$$

in which f is the friction between layer and slope. This leads to a layer thickness:

$$d \geq \frac{p_{\max} f}{(\rho_m - \rho_w)\,g\,(f\cos\alpha - \sin\alpha)} \tag{9.11}$$

Uplift

The protection is stable against uplift when there is equilibrium perpendicular to the slope:

$$(\rho_m - \rho_w)\,g\,d\,\Delta x \cos\alpha \geq p_{\max}\,\Delta x \quad \rightarrow \quad d \geq \frac{p_{\max}}{(\rho_m - \rho_w)\,g \cos\alpha} \tag{9.12}$$

Note: Above the waterlevel ρ_w must be used instead of (ρ_m - ρ_w) in these formulae.

Figure 9.5E gives the necessary layer thickness for slopes between 0 and 45 degrees using the following input parameters: p_{max} = 0.7 m, ρ_m = 2500 kg/m^3, ρ_w = 1020 kg/m^3, friction coefficient, f = 0.5 and 1 respectively. From this we can see the following:

- For a horizontal layer (α = 0), there is no difference between shear and uplift, or in other words: shear does not play a role.

- The shear criterion gives thicker layers than the uplift criterion. When $\tan\alpha$ becomes f, the necessary d becomes infinite, which, with $f\cos\alpha = \sin\alpha$, follows directly from equation (9.11).
- A larger friction, f, is favourable, compare f = 0.5 with f= 1. For a very high value f, the results for shear and uplift will coincide. The protection is "nailed", so to speak, to the slope. But, f is limited by the properties of the slope. It can never be higher than $\tan\phi$, the internal friction of the slope material, which becomes the weakest spot in the stability. For many cases, $2/3\tan\phi$ can be used as a first estimate for the value of f.

When the shear criterion is violated, part of the layer is going to hang on the upper part of the protection with force Fu in Figure 9.5D, or is going to lean on the lower part, which is again supported by the toe of the slope, with Fd. The tension in the layer can be computed from these forces: Fu = $(\rho_m - \rho_w).g.d.\sin\alpha$ - Fd - Ff(riction). (Actually both parts under and above water shoul be taken into account). Dependent on the strength of the layer, to avoid fatigue, this should not happen in situations with a frequency of occurence in the order of magnitude e.g. of weeks. Uplift is worse: sand can move under the protection when it is lifted and the layer can deform. This criterion should be met in design circumstances, e.g. a fast drop of waterlevel after a storm.

9.3.3 *Micro-stability of slopes*

The structure of grains and pores cause friction for the flowing water. Reversely the water exerts a force on the grains (action = reaction). This is the flow force (sometimes called the flow pressure, but it has the dimension of force per volume), which is important in the stability of the grains at the outer boundaries of the soil. It is given by:

$$p_f = \rho_w . g . i = \rho_w . g . \frac{\partial h}{\partial x} \tag{9.13}$$

for the x-direction; for the other directions, the expression is similar.

For the equilibrium of grains on a slope in porous flow from the inside of the slope (seepage), we cannot consider the stability of a single grain, since we have averaged the flow over all pores by means of u_f (equation (9.5)). By doing so, we have given up all information on velocities in the pores and instead, we consider a volume of soil. From equation (9.13) we know that the porous flow force on a unit volume, F_w, is equal to ρ_w g.i When θ is the angle between the porous flow direction and the horizontal, see Figure 9.6, the flow force along the slope is $F_W \cos(\alpha-\theta)$ and $F_W \sin(\alpha-\theta)$ perpendicular to the slope. The gravity forces on the soil are $(\rho_{SAT} - \rho_W)g.\sin\alpha$ along the slope and $(\rho_{SAT} - \rho_W)g.\cos\alpha$ perpendicular to the slope, where ρ_{SAT} is the specific mass of saturated soil. The equilibrium then becomes:

$$\left[(\rho_{SAT} - \rho_W) g \cos\alpha - i \rho_W g \sin(\alpha - \theta)\right] \tan\phi \geq (\rho_{SAT} - \rho_W) g \sin\alpha + i \rho_W g \cos(\alpha - \theta) \tag{9.14}$$

from which follows, with $\rho_{SAT} \approx 2000$ kg/m^3:
For some special cases the equilibrium condition leads to (see the cases A through D in

$$\tan\phi \geq \frac{\sin\alpha + i\cos(\alpha - \theta)}{\cos\alpha - i\sin(\alpha - \theta)} \tag{9.15}$$

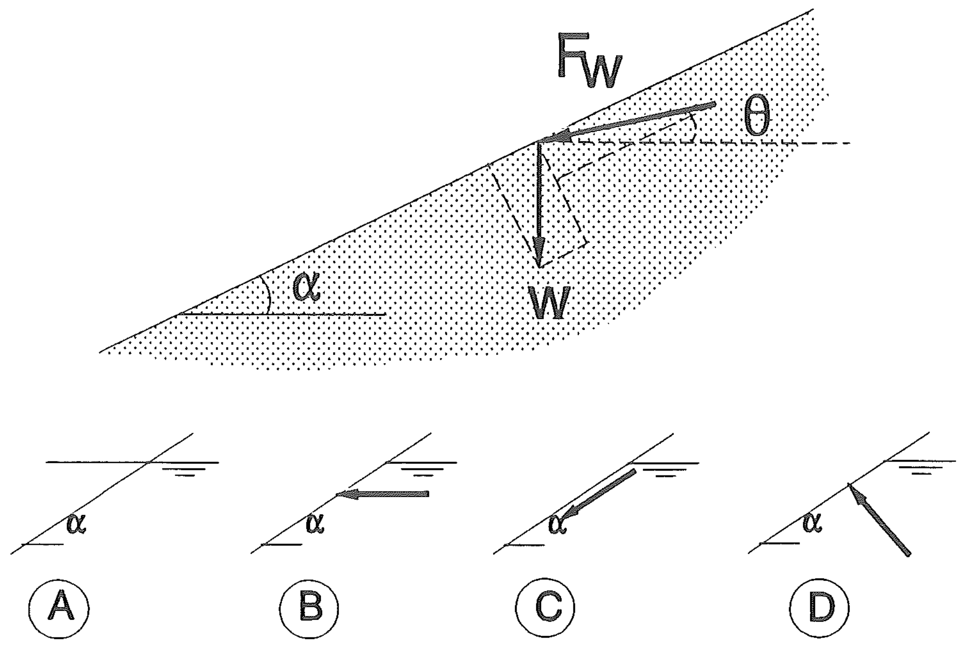


Figure 9.6 Microstability

Figure 9.6):

Situation	i	θ	Stability
A - No porous flow; dry or wet slope	0	-	$\alpha <= \phi$
B - Horizontal seepage	$\tan\alpha$	0	$\alpha <= \phi/2$
C - Seepage parallel to slope	$\sin\alpha$	α	$\tan\alpha <= (\tan\phi)/2$ (For small α equal to case b)
D - Seepage perpendicular to slope	i	α-90^0	$\sin\alpha/(\cos\alpha - i) <= \tan\phi$ (For $\alpha = 0$: i = 1; Fluidization)

There will always be a seepage surface on a slope where water flows out. Then, with $\phi = 30^0$, this leads to $\alpha \approx 15^0$ or a slope of 1:3.5 (case b or c). It is recommended to design an open revetment with this approach unless the soil is cohesive. In that case much steeper slopes are possible.

9.3.4 *Filters*

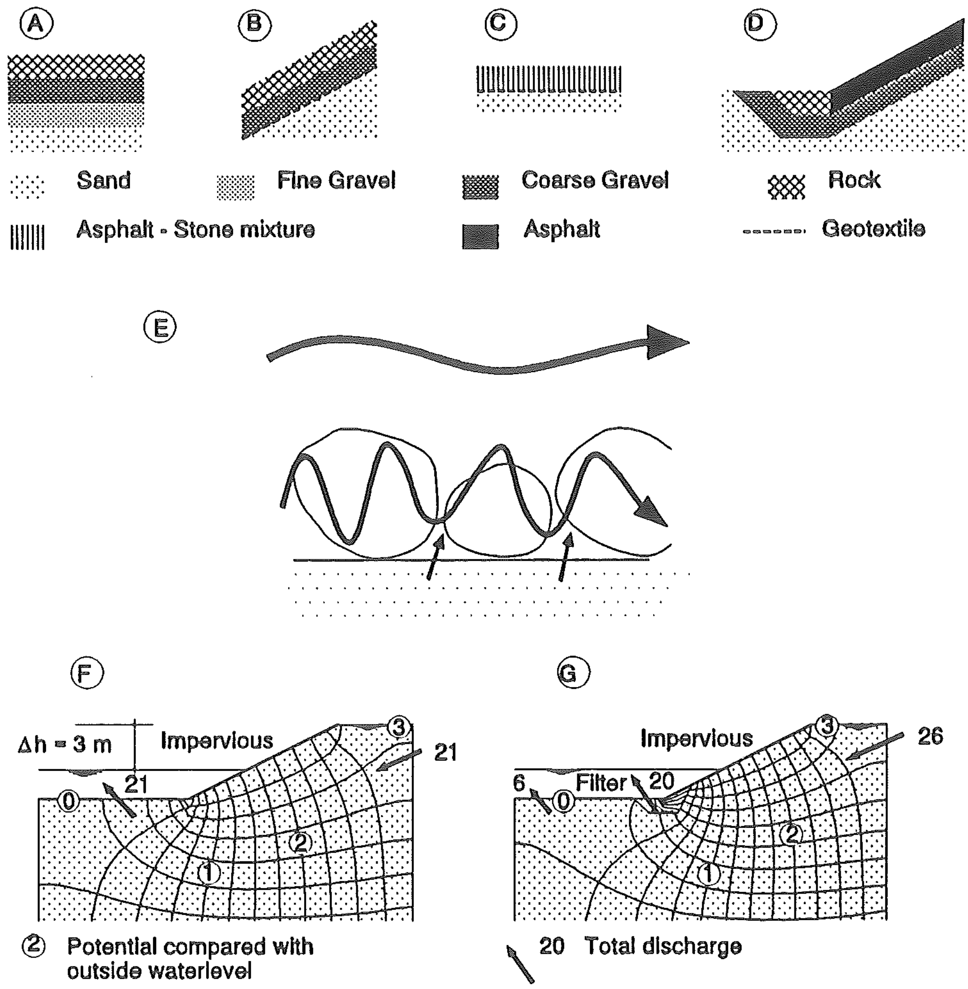


Figure 9.7 Examples of filters

Figure 9.7A through D give some examples of filters. Filters can have two functions: preventing erosion of the covered subsoil or preventing pressure build-up in the covered subsoil (drainage) or a combination of both.

Filters against erosion

Filters as protection against erosion are especially needed in situations with large gradients on the interface of soil and water, either bank or bottom. Examples are a closure dam with a head difference, where gradients parallel to the interface of 20-30% and more are possible, while that gradient in open water flow is usually two or three orders of magnitude smaller. In open channel flow, however, the flow velocity through the large stones of a bottom protection can cause erosion, while also the turbulence of the flow can penetrate through the top layer, see Figure 9.7E, especially in a flow deceleration zone downstream of a structure. The main function of protective filters, is to prevent the washing away of the material to be protected. That means that the underlying grains (the **base layer**) should not pass the pores of the upper layer (the **filter layer**). This can be done either by one or more layers of grains of varying diameter, a **granular filter**, see Figure 9.7A or with a **geotextile**, see Figure 9.7B. Figure 9.7C shows an example of a prefabricated combination, where stones with a diameter of a few cm are "glued" together with asphalt on a geotextile, reinforced with steel cables if necessary. With these relatively modern protection mats, it is possible to realise a reliable protection relatively fast in large quantities. The possible combinations of granular material, geotextiles, glues and reinforcements seem almost unlimited nowadays see also appendix A: Materials. It should be kept in mind, however, that many of these highly specialized products become economically attractive only when very large quantities have to be used. So, for many projects, protections with granular material and standard geotextiles, constructed on the spot, are the first thing to think of in a design.

Drainage filters

A filter as drainage, see Figure 9.7D, can help to prevent high excess pressures under an impervious revetment, see section 9.3.2, which is the second possible main function of a filter. Figure 9.7F and G show the influence of a filter at the toe of an impervious revetment (sand with k = 0.0001 m/s). A gravel-filter (k = 0.1 m/s) of 1 m thick and 2 m wide is applied at the toe. The filter reduces the maximum excess pressure under the revetment with 0.5 m (see potentiallines). Another effect is the decrease of the gradient in the sand. Although the filter "attracts" seepage, it is less concentrated than without filter (see the distance between streamlines at the toe in Figure 9.7F and at the transition between sand and gravel in Figure 9.7G). Of course, the filter itself should be able to resist the concentrated upflow and to prevent piping, which is also a type of erosion, for which extra layers of material can be necessary between the sand and the gravel.

In filters against erosion the emphasis is on **stability**, while **permeability**, in order to prevent pressure build-up, is the main property of drainage filters. But actually, both aspects play a role, as can already be seen in the example of a dainage filter given above, where erosion due to piping has to be prevented. In fact, both stability and permeability, are important in any filter design. Also in a filter that is only meant to prevent erosion, pressure build-up due to insufficient permeability has to be avoided in order to prevent lifting or sliding of a filter layer. Being closed for the base material and permeable for water are opposing demands; a solution has to be found within the margins left. The usual approach in design is to establish the necessary diameter for the stability of the top layer in waves and currents, to check the filter relations between the top layer and the original (base) material and to add as many granular layers or a geotextile in between as necessary to meet the filter demands. In the coming sections, rules will be given to dimension these filters.

For a state of the art in filter design, CUR,1993 gives an overview of the most relevant aspects.

9.3.5 Granular filters

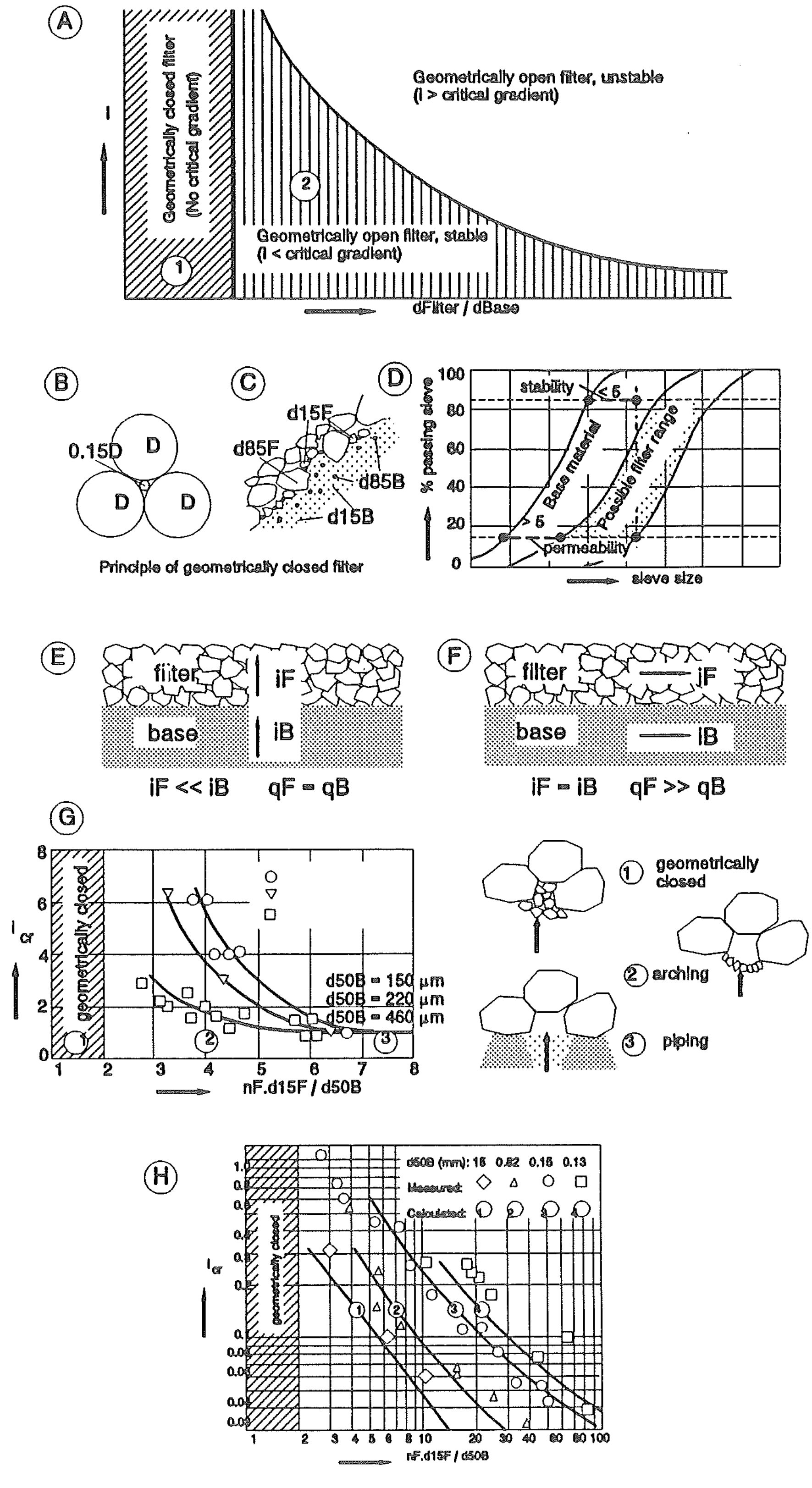


Figure 9.8 Possible design criteria for granular filters

The design of a granular filter can be based on different criteria, two of which will be discussed here. Figure 9.8A presents the relation between the critical gradient in porous flow, above which grains from the base layer are no longer stable under the filter layer, and the quotient of some representative diameter, to be defined later on, of the the two layers. In this figure two areas without erosion can be discerned:

1 Geometrically closed: These are the classical filter rules derived by Terzaghi in the 30's. The stability of the filter is based on the geometrical properties of the materials, the sieve curves. They are chosen such that grains cannot move in the filter. This often leads to a conservative design, since the filter can stand any load: there is no critical gradient. The filter layer diameters are a few times larger than the base layer diameters and the permeability of the filter layers has to be checked carefully.

2 Geometrically open: The grains of the filter layer are so much larger than those of the base layer, that the latter can move in and through the filter layer. But, like stability in open channel flow, there will be erosion only when the load is higher than some critical value. A critical filter gradient is now determined and a stable filter is designed, when this critical gradient is larger than the occurring gradients in the structure. So, now the hydraulic loads are taken into account, leading to a more economic design. Another name for the design criteria of these filters is hydraulic criteria. Permeability is usually no problem, since these filters are more open than the geometrically closed filters.

Sometimes, some loss of material is accepted within the limits of acceptable settlements, which leads to even larger quotients of filter and base diameters. This design approach is rather complicated and is outside the scope of this manual.

Geometrically closed filters

The idea behind a geometrically closed filter is the following, see Figure 9.8B, C and D. The space between packed grains is much smaller than the grains themselves. For spheres with equal diameter D, this space can be blocked by a sphere with a diameter which is more than 6 times smaller than D, see Figure 9.8B. In a layer of grains with varying diameter, the space between the grains is governed by the smaller grains, say d_{15}, the sieve diameter which is passed by only 15% of the grains. So, the escape route in the filter layer is determined by d_{15F} in Figure 9.8C. The largest grains of the base layer, d_{85B} in the figure, get stuck in the pores of the filter layer and block the passage of all other grains of the base layer, **provided the base layer is internally stable**, which means that the range of grain diameters in the base layer should not be too large, so that the larger grains of the base layer can block the smaller ones. In order to prevent pressure build-up, the permeability of a filter layer should be larger than the permeability of the base layer. Since the permeability is also governed by the smallest grains, this leads to a relation between the d_{15} of both layers. In words we just described three relations for a geometrically closed granular filter: **stability** between filter layer and base layer, **internal stability** and **permeability**. In formula, based on experiments:

$$\textit{Stability}: \frac{d_{15F}}{d_{85B}} < 5 \quad (\textit{provided int. stable}: \frac{d_{60}}{d_{10}} < 10) \quad \textit{Permeability}: \frac{d_{15F}}{d_{15B}} > 5 \quad (9.16)$$

Note: The demands for stability and permeability are contradictory: according to the stability rule d_F should be smaller than $5d_B$ and for permeability larger than $5d_B$. However, the use of two different diameters for the base material in the denominator (the 15% largest and the 15% smallest respectively) gives the margin to design a filter, see Figure 9.8D.
Note: The demand for internal stability of the base layer, is inextricably connected with the demand for stability between both layers. Only then, d_{15F} will withhold d_{85B}, which in turn will withhold the small grains of the base layer, making the filter stable as a whole.

With these rules, a stable and permeable filter can be designed, either starting from the original soil as base layer or from the top layer which is necessary for stability. This can lead to more than two layers in the filter, sometimes even 4 or 5. When using the relations from equation (9.16), out of each pair of two layers the finest is named the base layer and the coarsest the filter layer.

Geometrically open filters

As said in the introducion on filters, the idea behind a geometrically open or "hydraulic" filter is that the occurring gradient is lower than the critical value. The gradient is, via the Forchheimer equation (9.5), related to the velocity in the filter, which is again related to the forces on a grain. When the loading forces on a grain of the base layer are smaller than the resisting forces, there will be no erosion. Although the flow through a granular filter can have any direction, it is advantageous for design and research purposes, to discern the two possible main directions: perpendicular and parallel, see Figure 9.8E and F. The critical gradient in the case of parallel flow is defined in the filter layer and with perpendicular flow in the base layer, which is also related to the different erosion mechanism.

When the water flows parallel to the interface, the gradient in both layers is about the same, causing the flow velocity in the filter layer to be much larger than in the base layer, because of the larger permeability. At the interface there will be a velocity gradient, inducing a shear stress at the upper grains in the base layer. The situation is very much like incipient motion in an open channel and the critical shear stress, hence the critical velocity, hence the critical gradient is defined in the filter layer. With perpendicular flow, there is a serial system where the flow through base and filter layer has to be the same, causing a much larger gradient in the base layer, because of the larger permeability of the filter layer. Erosion of the base layer will take place due to fluidization, hence the gradient in the base layer determines the stability.

Note: In this chapter only steady flow situations will be described. In the sections on revetments and breakwaters, the influence of dynamically loading will be discussed.

Perpendicular flow

With flow perpendicular to an interface in a filter we have to take gravity into account. When the base layer is on top of the filter layer and the flow is downward, it is recommended to follow the rules for geometrially closed filters, since there will be hardly any critical gradient when the filter is open. The finer grains will fall through the filter layer. In the case of upward flow through a base layer lying under a filter layer, there is always a lower limit for erosion: the fluidization criterion, for which the gradient is about 1 or 100% which follows from the vertical equilibrium. The upper limit is again a geometrically closed filter.

Figure 9.8G shows the results of some tests. The diameter quotient on the horizontal axis is somewhat different from the one used in the stability relation of equation (9.16). Instead of d_{85B} now d_{50B} is used and the porosity in the filter layer is taken into account. The relation between the two can be estimated: assuming $n_F \approx 0.35$ and $d_{50B} \approx 0.85*d_{85B}$, $n_F*d_{15F}/d_{50B} \approx 0.35*d_{15F}/0.85*d_{85B} \approx 0.4*5 = 2$ as a conservative estimate for the geometrically closed limit, area 1 in the figure. The lower limit is indeed formed by fluidization when the filter grains become so large that the base acts like there was no filter (area 3 in the figure). In between, in area 2, several mechanisms play a role. Grains that flow from the base layer into the filter layer, come in much larger pores, hence in much smaller velocities, so the forces on such a grain decrease and it will not be transported further upward. Another mechanism is arching, where bridges of fine grains block the filter, although the geometrical criteria are not met. Note that the results are realtively more favourable for finer grains.

Parallel flow

The stability of base material in a flow parallel to the interface of a filter could be seen as the stability of a grain on the bottom of a very small channel. It is an attractive idea to link the Forchheimer equation (9.5) with the threshold of movement as given by Shields, see section 9.3.1, in order to establish a relation for the critical gradient in porous flow. This was tried and partially successful (de Graauw,1983), resulting in an empirical relation. The most important parameters appeared to be the diameter of the base material (as a measure for the stability) and the diameter and porosity of the filter material (both as a measure for the flow through the filter). The experimental results finally yielded:

$$I_C = \left[\frac{0.06}{n_F^3 \, d_{15F}^{4/3}} + \frac{n_F^{5/3} \, d_{15F}^{1/3}}{1000 \, d_{50B}^{5/3}} \right] u_{*C}^2 \qquad (9.17)$$

in which u_{*C} is the critical shear velocity according to Shields. The two terms in (9.17) come from the idea of a laminar and turbulent part like in the Forchheimer equation. Figure 9.8H gives a comparison between some computed lines and the experiments. Again it can be seen that for a given ratio of $n_F d_{15F}/d_{50B}$, the critical gradient is greater for fine than for coarse base material. This could be explained by the lower filter velocities in fine pores, hence a relatively larger gradient in the filter layer is needed to reach the critical velocity. With base material larger than about 5 mm, this effect appears to vanish.

Note 1: When the filter is on a slope, u_{*C}^2 in equation (9.17) has to be corrected in the same way as is done in open channel flow, with $\sin(\phi-\alpha)/\sin\phi$, see section 9.2.1.
Note 2: When there is a combination of a perpendicular and a parallel gradient, the stability is governed by the parallel flow as long as the perpendicular gradient is < 0.5.

Basically, everything what was said about filters in stationary flow is also true for filters in structures loaded by waves, with the following additional remarks:

The relations between diameters in geometrically closed filters are such that the grains can not move inside the filter, so this should also be good enough in unstationary conditions. This is indeed the case, only in very heavy cyclic loading perpendicular to the interface, the geometrically limits appear to be not always sacred.

In geometrically open filters with flow parallel to the interface, it was found that for cycles > 2 s, the critical filter velocities and gradients, were the same as for stationary flow (de Graauw et al,1983). For cyclic perpendicular loading (de Graauw et al,1983), a considerable difference with stationary tests was found, see Figure 9.9a. The critical gradient appeared to be lower, both for fine and coarse sand. An explanation was found in the arches, see section 6.2.3, which can not build up or are destroyed during reversed flow. For coarse sand there appeared to be a critical gradient of about 200 % inside the geometrically closed region. For e.g. a permeable dike toe, one does not have to worry about a geometrically closed filter drain. In the case of heavy dynamic loading, e.g. "rocking" of a structure on a filter foundation, tailored research should be done.

For the transition between armour layer and first filter layer in breakwaters, often relations like $W_F/W_B < 15 - 25$ are mentioned in manuals. With $W \propto d^3$ this comes down to $d_F/d_B < 2.5 - 3$, which is more strict than the geometrically closed criteria. The reason for this does not come from filter rules, but is probably done for mechanical reasons. Often, the individual elements in the armour layer, see Figure 9.9b, are too large to speak of a layer. With the geometrically closed rules, the surface would be too "smooth" to form a stable foundation for the large elements.

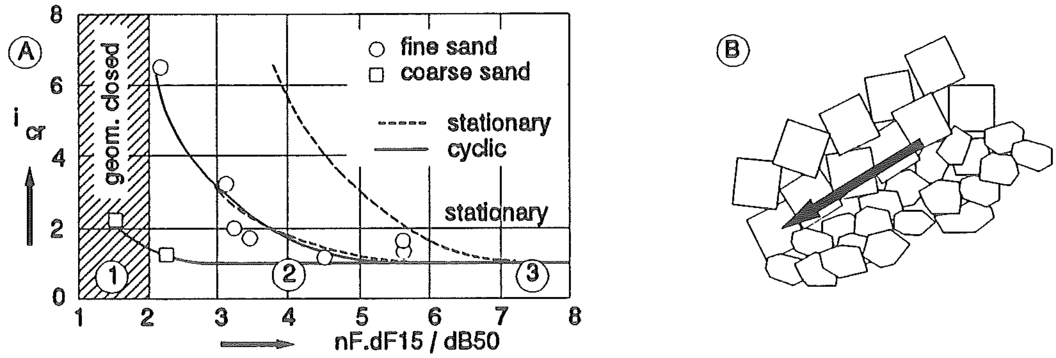


Figure 9.9 Filters in unstationary flow

9.3.6 Geotextiles

Geotextiles is a generic term for all kinds of cloth or foil-like woven or non-woven material from natural (coconut, jute etc.) ar artificial (polymer) fibres, which become more and more important in civil engineering. They are being used in armouring soil in foundations and slopes, as membranes to prevent seepage or to protect the environment against pollution from a dump area and as filter in hydraulic engineering. The latter is the subject in this paragraph. Geotextiles come in many shapes, depending on the production technique. The two main types are woven and non-woven. See annex A. A possible disadvantage of geotextiles can be the weathering e.g. by ultra-violet light, wear and tear by chemical, biological or mechanical processes. These aspects are outside the scope of these lecture notes. For an extensive treatment of geotextiles the reader is referred to Veldhuijzen van Zanten,1986.

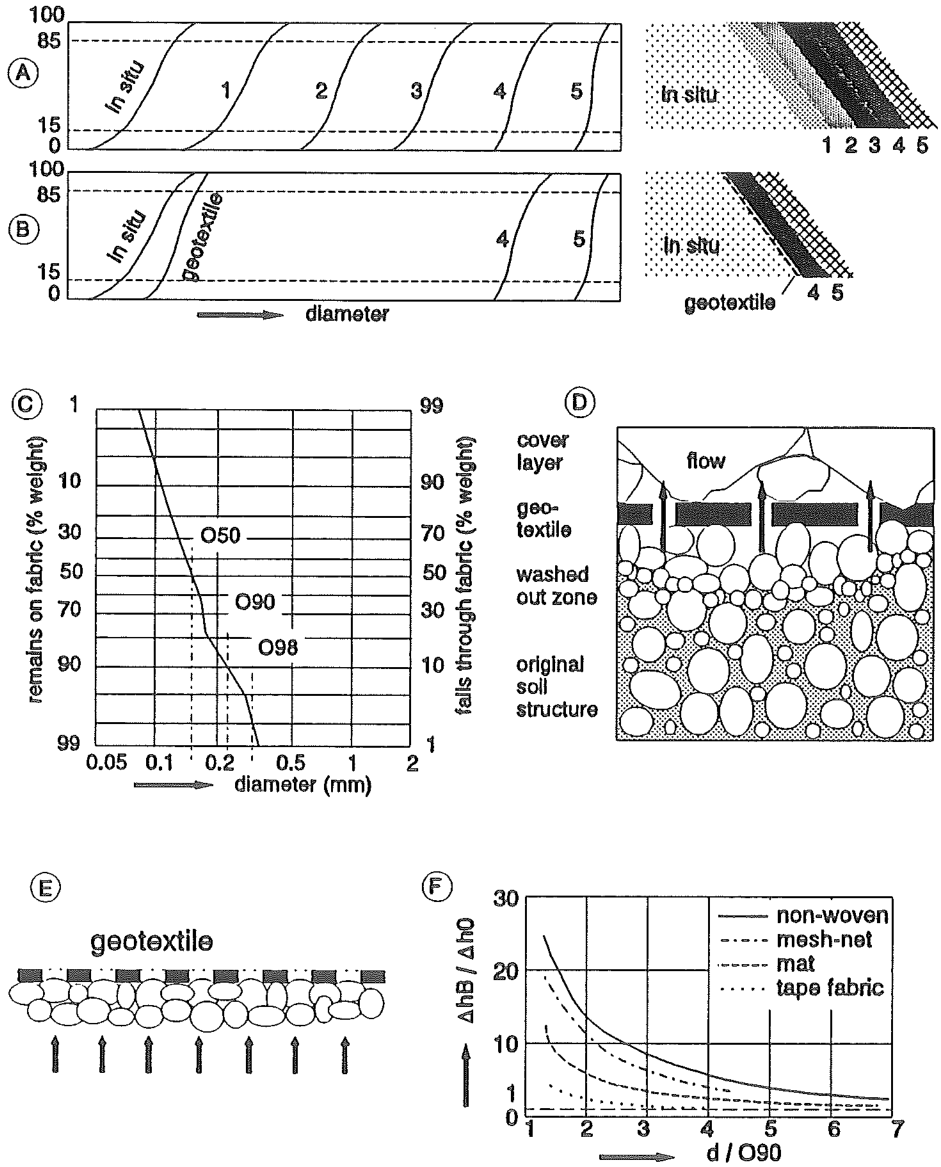


Figure 9.10 Various aspects of geotextiles

Geotextiles have given a breakthrough in filter design, both in cost effectiveness for large projects and in possibilities to design a sandtight filter with a limited thickness, see Figure 9.10A and B, where a geotextile replaces 3 layers of granular filter. This is especially useful in cases where there is no space for a granular filter of many layers or where it is difficult to construct such a filter. The geotextile in this figure is completely responsible for the filter function. The main functions of the layer between the top layer and the geotextile is to prevent damaging the geotextile by individual large stones and to prevent flapping of the cloth, possibly causing losses of the base material.

Like in granular filters, the two main items are again stability (sandtightness) and permeability. Another similarity is the existence of geometrically closed and open filters. Here only geometrically closed filters will be considered. Another item that can play a role is the stability of a top layer on a geotextile on a slope. Like with impervious revetments, see section 9.3.2, friction between geotextile and slope has to be checked.

Filter stability

Like in granular filters, the relation between the magnitude of the base material and the holes in the filtertextile is the dominating factor for sandtightness. To characterize the openings on the textile, a kind of reversed sieve test is performed with very uniform grains of different and known diameter. The textile is used as a sieve in a standardized procedure (which can be done in many ways, see Veldhuijzen van Zanten,1986; here the Dutch procedure as developed by Delft Hydraulics is presented). When X % of the grains with a certain diameter remains on the textile, that diameter is taken as O_X. The quantity (100 - X) % passes the "sieve", this is the reversed definition of a normal sieve procedure! Figure 9.10C gives the result of such a test. Important numbers in sandtightness are O_{90} and O_{98}, sometimes named O_{max}, a measure for the largest holes in the textile. Note that of course, contrary to granular filters, the escape route is now determined by the largest holes instead of the finest grains!

The most strict geometrical filter rule is that the smallest particles cannot pass the largest opening in the textile. This should be applied when the filter is cyclically loaded and no loss of material is accepted. In many situations, some loss of fine material is not detrimental to the functioning of the filter, since a small layer under the textile can act as part of the total filter system, see Figure 9.10D. The finer parts are washed through the textile and the coarser particles act as a filter for the remaining soil, provided the subsoil is internally stable, see granular filters.

The stability rule for stationary flow through geometrically closed geotextiles is simply:

$$O_{90} < 2d_{90B} \tag{9.18}$$

For geometrically open geotextiles, very little research was done. CUR,1993 gives some information.

Permeability

Like with granular filters, to prevent pressure build-up, a geotextile should be more permeable than the subsoil. A simple rule is that the permeability, k, of the geotextile should be more than 10 times larger than that of the subsoil. The permeability of a geotextile can be measured in the same way as for soils, using Darcy type relations, but a parameter often used to characterize the permeability of geotextiles is the permittivity, which is defined as:

$$\psi = \frac{u_f}{\Delta h} = \frac{k}{e} \tag{9.19}$$

in which Δh is the head difference, e is the thickness of the geotextile and k the "normal" permeability coefficient. ψ can be seen as the permeability per m thickness of geotextile and is a property of the material, regardless of the thickness of the geotextile. The following table gives an idea of the permittivity of some geotextiles:

Type	O_{90} (mm)	Permittivity (1/s)
Mesh net	0.1 - 1	1 - 5
Tape fabric	0.05 - 0.6	0.1 - 1
Mat	0.2 - 1	0.05 - 0.5
Non-woven	0.02 - 0.2	0.01 - 2

Usually, a geotextile is more permeable than the subsoil, but two factors can decrease the permeability: blocking and clogging.

Blocking is the phenomenon that large particles seal the openings in the textile. In that case, the permeability can decrease dramatically as is shown in Figure 9.10E and F where a very uniform base material is used in permeability tests (all grains have about the same size). When the diameter of the base material has the same order of magnitude as the O_{90}, the head difference over the textile can increase a factor 10-20 (compared with the situation without grains). Note that not every type of textile is equally sensitive to this phenomenon.

Clogging is the trapping of (very) fine particles in the openings of the textile, leading also to a decrease of permeability. This can happen in water contaminated with chemicals, e.g. iron. In contrast with blocking, clogging is a time dependent process. The idea is that it stabilizes at a certain level after a certain time, but not much experimental evidence is available to support this thesis. It should be realized that clogging is not special for geotextiles; it also occurs in granular filters.

When the permeability of the textile is 10 times greater than that of the subsoil, there is usually no problem with pressure build-up, not even with blocking or clogging. However, when used in very contaminated groundwater, one should examine the possibility of severe clogging. When the danger of blocking or clogging can be excluded, a permeability factor of 2 or 3, is sufficient.

9.4 Loads by waves

9.4.1 General

Under waves the sitiuaton is different from flow, because the relation between shear stress and orbital velocity is different. In short waves the boundary layer never gets the order of magnitude of the waterdepth, which is the case in uniform flow, hence high friction factors can be expected. The friction factor is defined in a way analogous to that in a flow:

$$\hat{\tau}_w = \frac{1}{2}\rho\, C_f \hat{u}_b^2 \quad with: \quad \hat{u}_b = \omega a_b = \frac{\omega a}{\sinh kh} \quad and: \quad u = \hat{u}_b \sin\omega t \tag{9.20}$$

where the index w stands for wave and b for bottom. The average value over a wave period is half the value of the maximum (the average of $\sin^2 \omega t$ is 1/2).

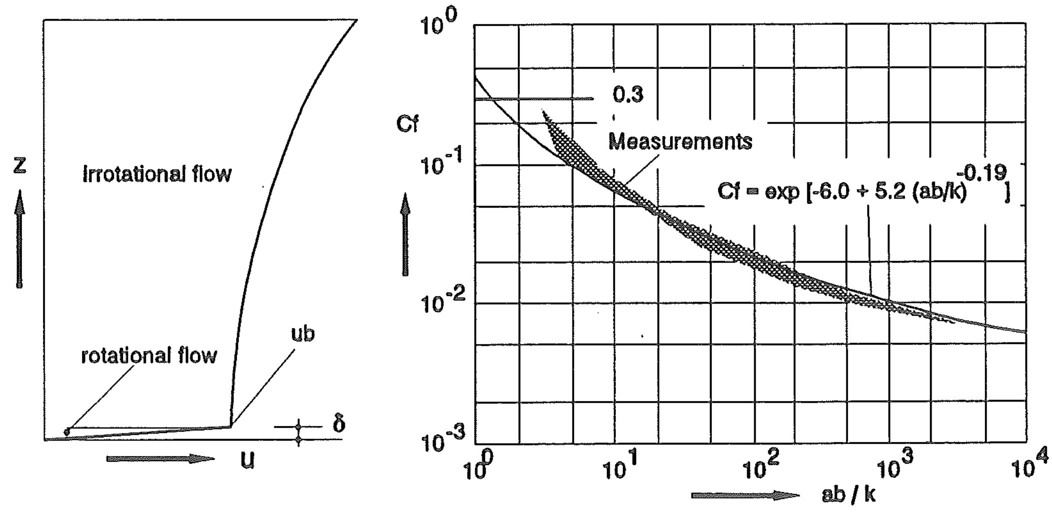


Figure 9.11 Friction under waves

Jonsson,1966 found an expression for turbulent flow over a rough bed based on experiments, rewritten in a more practical form in e.g. CUR/CIRIA,1991:

$$C_f = \exp\left[-6.0 + 5.2\,(a_b/k)^{-0.19}\right] \quad with\ C_{f\,\max} = 0.3 \tag{9.21}$$

Comparing the values in Figure 9.11b with those for flow, it can be seen that shear stress under short waves can be much higher than in flow. C_f in uniform flow is: $2g/C^2$ which results in values of 0.005 - 0.01, while in very short waves, where the amplitude at the bottom has the same order of magnitude as the roughness, values of 0.1-0.3 seem possible. Often, 0.3 is taken as the maximum value.

In most cases, both waves and flow will be present. When waves are dominant, the above mentioned approach is recommended, but when flow is dominant, the influence of waves can, according to Bijker,1967, be taken into account by adding vectorially the current and orbital velocity at a level where:

$$u_{c-t} = \frac{\sqrt{g}}{\kappa C} u_c \qquad u_{b-t} = \frac{1}{\kappa}\sqrt{\frac{C_f}{2}}\, u_b \sin(\omega t) \quad (u_b = \text{max. orb. vel. at bottom}) \tag{9.22}$$

in which $\kappa \approx 0.4$. Only the results are given here:

$$u_r = \sqrt{\frac{g}{\kappa^2 C^2} u_c^2 + \frac{C_f}{2\kappa^2} u_b^2 \sin^2(\omega t) + 2\frac{\sqrt{g}}{\kappa C} u_c \frac{1}{\kappa}\sqrt{\frac{C_f}{2}}\, u_b \sin(\omega t)\sin(\phi)} \tag{9.23}$$

where ϕ is the angle between wave and current direction. The resulting shear is:

$$\tau_r = \rho\, \kappa^2 u_r^2 \tag{9.24}$$

From equation (9.23) can be seen that the resulting velocity is maximum when the wave direction is parallel to the flow. In that case the velocities are simply added and the square of the resulting velocity gives the maximum shear, see Figure 9.12.

Note: For sediment transport, equation (9.23) is averaged over a wave cycle. The third term then vanishes and the resulting shear becomes independent of ϕ. For bottom protections, the maximum value determines incipient motion.

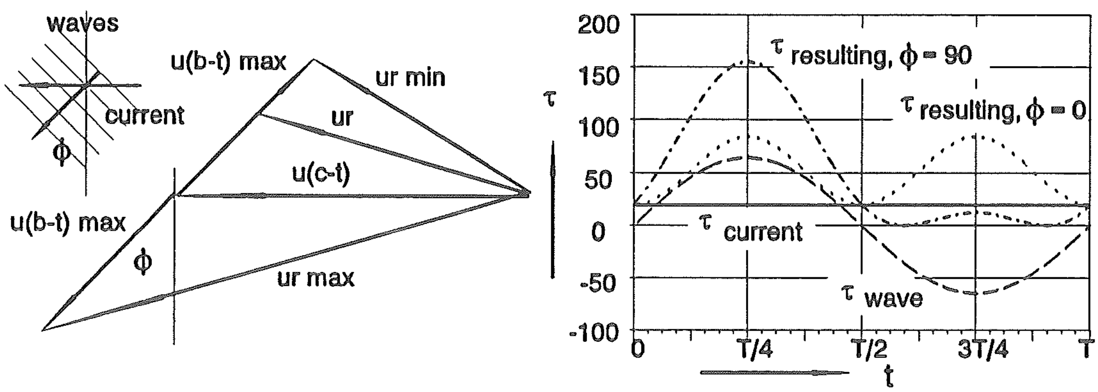


Figure 9.12 Combined wave-current action

9.4.2 Stability

With equation (9.21), the shear stress under a wave can be determined and with this shear stress we could use the same stability relation as in flow, like the one by Shields, which relates the dimensionless shear stress to the grain diameter. This appears to be not completely true, probably due to different (laminar) boundary behaviour in an oscillating flow.

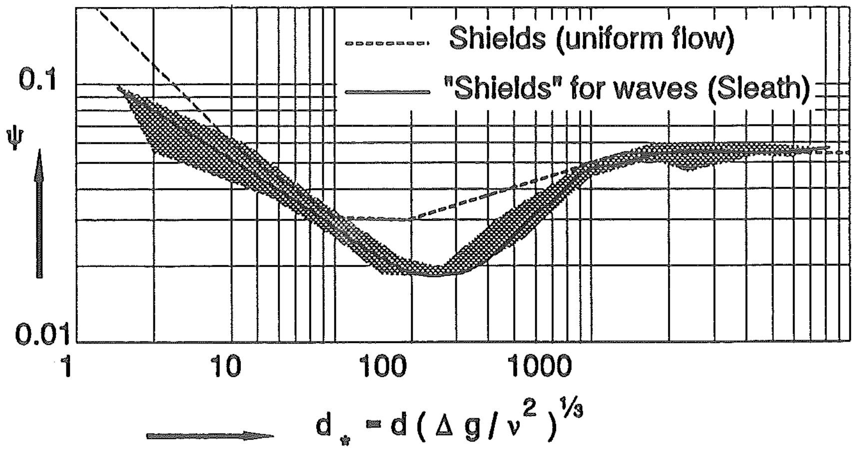


Figure 9.13 Incipient motion under waves: modified Shields diagram and stability on mild slope in non breaking waves

Figure 9.13 shows stability measurements in oscillating flow by various authors, summarized by Sleath,1978. Sleath uses the same dimensionless shear stress as Shields (ψ), and the same dimensionless grain diameter as vanRijn for flow, see chapter 4. So, the results can be compared directly, see Figure 9.13. For large d_* (turbulent boundary layer), Sleath found a value for ψ to be 0.055, which is the same value as Shields.

Stability in breaking waves

Most research on the stability of stones on a slope was done in the field of breakwaters, which is very much related to slope protections and revetments, but it is not the same. An important difference in the stability is the porosity of the structure as a whole. Breakwaters usually have a porous core, while in dike revetments the core is made of clay or sand. This has a significant influence on the stability of the protecting armour layer as we will see later on in this section. We will start, however, with simple basic principles and will discuss shortcomings and extensions later on.

Encouraged by the succesful results of the modified Shields curve for waves, the next step would be the use of a slope correction factor and to take the breaking of the waves into account. We use the same slope factor as for flow in the slope direction (for the 1:25 slope in the previous section, this factor $\approx$ 1). For the velocity in a breaking wave, no reliable expression is available. As a first guess, we will assume that the velocity in a breaking or broken wave on a slope is proportional to the celerity in shallow water with the wave height as a representative measure for the waterdepth: $u \propto \sqrt{gH}$. Following the same reasoning as with stability in flow, we find:
Note: + and - in the slope correction are for uprush and backwash respectively.

$$\underset{\text{"drag"force}}{\rho_w g H d^2} \quad \propto \quad \underset{\text{resisting force}}{(\rho_s - \rho_w) g d^3} \quad \underset{\text{slope correction}}{(\tan\phi \cos\alpha \pm \sin\alpha)} \tag{9.25}$$

Raising all terms to the third power and working with the mass of the stone (M $\propto$ $\rho_s d^3$) we find:

$$M \propto \frac{\rho_s H^3}{\Delta^3 (\tan\phi \cos\alpha \pm \sin\alpha)^3} \tag{9.26}$$

This is Irribarren's formula for stability of rock on slopes in waves, dating back to 1938. After the second world war, many tests were performed (particularly by Hudson,1953) to find the constants of proportionality in equation (9.26). Hudson finally proposed, for practical reasons, another formula:

$$M = \frac{\rho_s H^3}{K_D \Delta^3 \cot\alpha} \quad (or: \quad \frac{H}{\Delta d} = \sqrt[3]{K_D \cot\alpha}) \tag{9.27}$$

Note: the use of H/Δd as a stability parameter is convenient and is similar to flow, where $u^2/\Delta d$ is used. The slope correction in Irribarren's formula is now reduced to cotα. This means that the validity of Hudson's formula is limited, because cotα is insufficient to describe friction and equilibrium on a slope: for $\alpha = 0$, W = 0 and for $\alpha > \phi$, W has still a finite value, both of which are nonsense. The range of α for which Hudson is valid is about 1.5 < cotα < 4. The Hudson-formula is tested for waves that did not break at the toe of the slope and did not overtop it. For cases where this is not true, extra corrections for K_D are sometimes applied. Hudson's formula is simple and is used worldwide. K_D is again a dustbin-factor in which implicitely the accepted degree of damage. K_D has different values for different kinds of elements (3-4 for natural rock to 8-10 for artificial elements like tetrapods and tribars). The Shore Protection Manual (SPM,1984) gives values for K_D for various circumstances.

The simplicity of the Hudson formula has its price. Some limitations of the presented formulae have already been mentioned. The most important are:

Wave period
Already in the Irribarren-formula this parameter is absent. There are two ways in which the period can have influence on stability. The period is related to the wave-length, hence to the wave-steepness and hence to the breaking pattern on the slope, which will definitely play a role. The other one is the possible role of inertia-forces on a grain which depend on du/dt, hence on the wave-period.

Permeability
The permeability of the structure must play an important role. In the assumptions on which Irribarren's formula is based, only a kind of drag force on the slope is included. The forces under and behind a grain will certainly be co-responsible for the equilibrium. It is easy to imagine that a homogeneous mass of stones will react different from a cover layer of stones on an impermeable core. In the first case, wave energy is dissipated in the core, while in the latter the pressure build-up under the cover-layer can be considerable.

Number of waves
All modeltests in the 50's and 60's were done with regular, monochromatic waves. It appeared in those tests that the equilibrium damage profile was reached in, say, one half hour. The wave height in the tests was then usually declared equivalent to H_S. It appeared from tests with wave spectra that the number of waves is not unimportant, which is logic, since more waves mean a greater chance for a large one.

Damage level
Like stones in flow, the threshold of motion is not always clear. The K_D-values in the Hudson formula are supposed to be valid for 5% damage, but the definition of damage is not very clear.

All these objections led to new research activities, especially after some failures of breakwaters. In a period of about ten years the recommended coefficients in the Shore Protection Manual for breakwater design with the Hudson formula led to an increase in weight of about 200%. In the Netherlands, extensive modeltests were done to overcome the limitations of and objections against the Hudson-formula. The results of curve-fitting of these large and small scale tests with irregular waves, finally led to the following equations, see van der Meer,1988 summarized in CUR/CIRIA,1991:

$$\frac{H_S}{\Delta d} = 6.2\,P^{\,0.18}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5} \qquad (\textit{plunging breakers})$$
$$\frac{H_S}{\Delta d} = 1.0\,P^{-0.13}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi_m^{P}\sqrt{\cot\alpha} \qquad (\textit{surging breakers}) \qquad (9.28)$$

The transition between the two expressions is found by equating them, giving:

$$\xi_{transition} = \left[6.2\,P^{\,0.31}\sqrt{\tan\alpha}\right]^{\left(\frac{1}{P+0.5}\right)} \qquad (9.29)$$

when $\xi > \xi_t$ the equation for surging breakers has to be used, for $\xi < \xi_t$, the equation for plunging breakers. In practice, for $\cot\alpha >= 4$ surging waves do not exist and only the expression for plunging waves is recommended for use.

These equations are a step forward compared with Hudson, because more parameters are included. As stability parameter vanderMeer used $H_S/\Delta d$ (to which also Hudson easily can be rewritten, see equation (9.27)), the wave-period appears in the surf-similarity parameter, ξ_m (m means related to the average wave period), while there is a discontinuity in the stability relations between surging and plunging breakers. In the following the influence of the parameters in equations (9.28) will be demonstrated with some computations and compared with the Hudson formula (equation (9.27)). As standard case, the following parameters will be used:

$H_s = 2$ m	$T_m = 6$ s	$\Delta = 1.65$
$\cot\alpha = 3$	$N = 3000$	$S = 2$
$P = 0.5$	$d_{n50} = 0.6$ m (300 - 1000 kg)	$K_D = 3.5$

Wave period

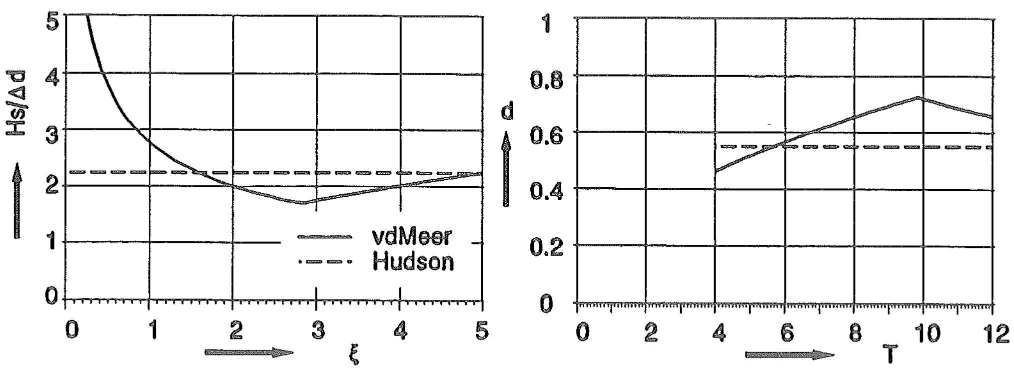


Figure 9.14 Stability parameter as a function of ξ and diameter as a function of T

The influence of the wave period is incorporated in the breaker parameter, ξ. Figure 9.14a shows the relation between the stability parameter, $H_s/\Delta d$, with $d = d_{n50}$, unless otherwise stated, and ξ (with all other parameters as in the standard case). Near $\xi = 3$, $H_s/\Delta d$ has its minimum. This is in the transition zone between surging and plunging breakers, which appears to give the most severe attack on the slope, see also the section on breaking waves in chapter 4. Note that typical values of $H_s/\Delta d$ lie around 2 for both formulae. In Figure 9.14b the period is the independent parameter. Note that periods lower than 4 s have not been drawn, since a maximum wave steepness ($s = H_s/L_0$) = 0.06 has been assumed, which with a wave height of 2 m leads to a minimum possible wave period of $\approx 3\sqrt{H_s}$. In the Hudson formula, T does not play any role.

Permeability

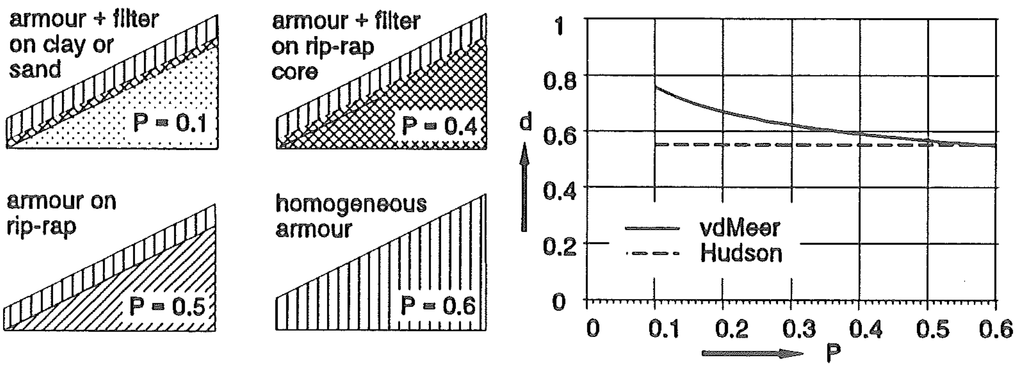


Figure 9.15 Stone diameter as a function of slope permeability

A permeability parameter P was introduced and the value for different structures was established by curve-fitting of the results. Figure 9.15a gives the values for various situations. Homogeneous rock (no core) gives $P \approx 0.6$, a rock armour-layer with a permeable core ($d_A/d_F \approx 3$): $P \approx 0.5$, an armour layer with filter ($d_A/d_F \approx 2$) on a permeable core ($d_F/d_C \approx 4$): $P \approx 0.4$ and an "impermeable" core: $P \approx 0.1$. Impermeable is again a relative notion, wave penetration in clay or even sand is almost negligible, so, in these stability relations the slope is considered impermeable. Attempts were made to derive P from porous flow calculations, see van Gent,1993. Figure 9.15b shows the influence of P on the necessary diameter. For permeable slopes, the results from Hudson or vanderMeer are almost identical, but for impermeable slopes, like dikes or revetments, the difference is considerable (0.75 m versus 0.55 m, which is a 2.5 times heavier stone).

Number of waves

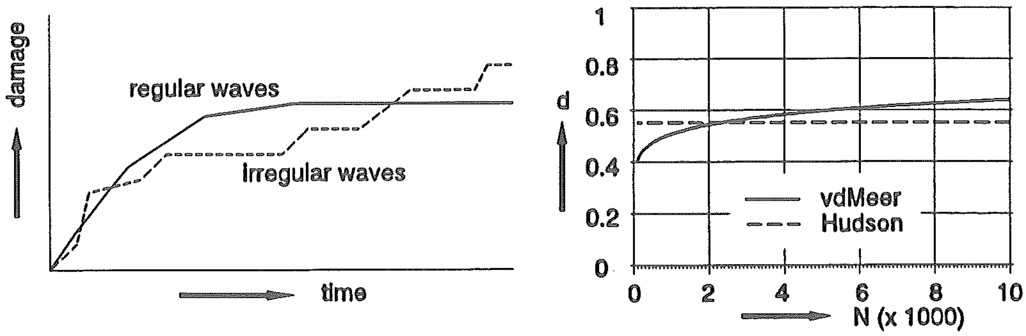


Figure 9.16 Stone diameter as a function of number of waves

N is the number of waves. Figure 9.16 gives the increase of the damage with the number of waves during a test. With N = 7500, the damage can be considered to have reached an equilibrium. When only storms of short duration occur and intensive maintenance will be done, N can be chosen lower. 3000 waves with an average period of 6 s, represent a storm of 5 hours.

Damage level

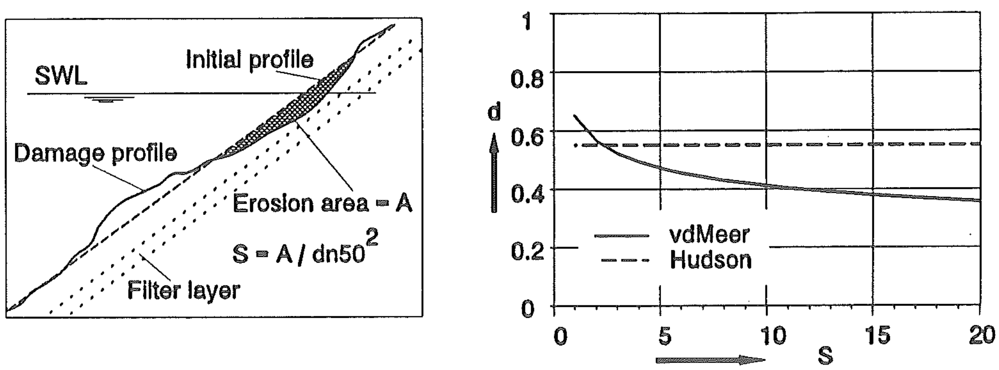


Figure 9.17 Stone diameter as a function of the damage level

The damage level was defined in a more manageable way, see Figure 9.17a: $S = A_e/d^2$. This is an erosion area divided by the square of the diameter. In a strip with a width d perpendicular to the paper, S is more or less equal to the number of stones that is removed. The advantage of S is the use of an area which can be measured objectively from soundings. For the threshold of damage, S = 2-3, can be used. When the armour-layer is locally removed completely and the filter-layer becomes exposed, the damage can be defined as failure of the structure. Depending on the slope, the matching S is about 10.

Wave height and slope angle α

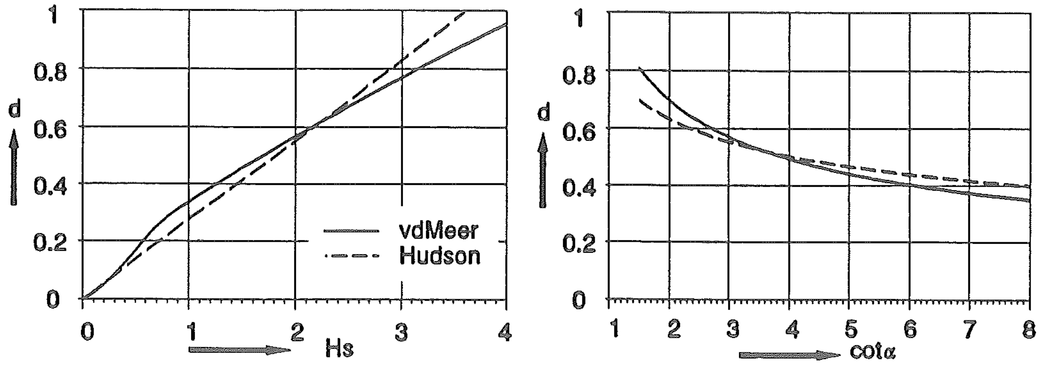


Figure 9.18 Stone diameter as a function of wave height and slope angle

Figure 9.18 finally shows the influence of wave height and cotα of the slope. For these two parameters, Hudson and vanderMeer show the same tendency.

Damage during construction

Another advantage of the vanderMeer formulae, is the possibility to take the damage development into acccount. This can be important for repair and maintenance policy, but also for construction of a protection. S can be computed from equation (9.28), rewritten to compute S explicitely.

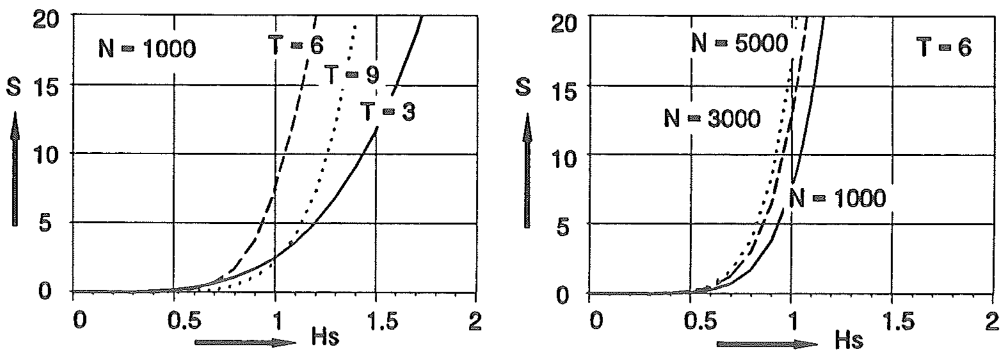


Figure 9.19 Damage as a function of wave height, wave period and storm duration

Figure 9.19 shows the damage as a function of wave height for stone with d_{n50} = 0.25 m, cotα = 2 and P = 0.6. This could be the construction of the core of a breakwater, which is exposed to waves. By building the breakwater in the "moderate" season, the waves are usually lower than the design waves for the breakwater. Figure 9.19a shows the influence of the wave period on the damage, for a duration of 1000 waves and Figure 9.19b the influence of the number of waves with a period of 6 s. Note that a period of 6 s gives more damage than 3 or 9 s, which again has to do with the transition between plunging and surging breakers. The number of waves is not so important. With these figures and wave or wind statistics from the area, an expectation of the damage can be determined.

9.4.3 Erosion

Sand

Before considering the necessity of protecting a bank, a shore or a bottom against waves, one should have an idea of the possible erosion. For banks and shores this will be done on the basis of beach and dune erosion research, hence for sand.

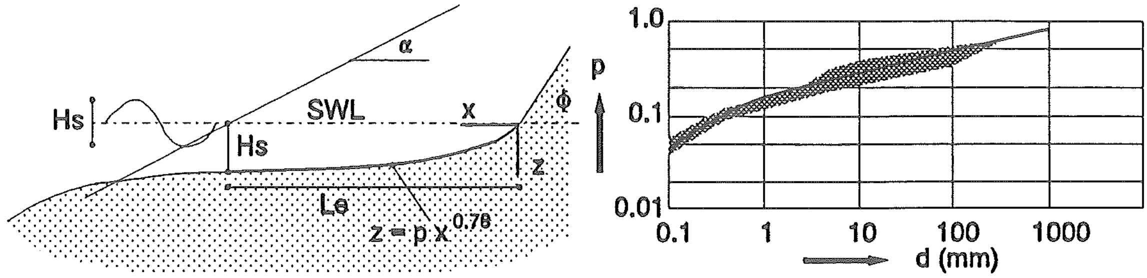


Figure 9.20 Erosion of slope by waves

Waves acting on an unprotected slope, cause a step-profile. This is the natural equilibrium profile for all slopes composed of loose material (see e.g. Vellinga,1986). From theoretical considerations for beaches a parabolic profile is found with z :: $x^{2/3}$. This theory however is valid for monochromatic, linear waves. Vellinga proposes:

$$z = 0.39\, w^{0.44}\, x^{0.78} = p\, x^{0.78} \tag{9.30}$$

in which w is the fall velocity of the particles; for the definition of z and x, see Figure 9.20a.
Note: this equation is a simplification of Vellinga's complete equation, see chapter 4.

This is an equilibrium profile; development in time is not considered. The erosion depth below SWL is about H_S. From equation (9.30) the intrusion depth of the waves in the profile is then found by:

$$L_e = p^{-1.28}\, H_s^{1.28} \tag{9.31}$$

Above the still water level the slope is assumed to be equal to the angle of repose. Figure 9.20b gives a relation for the value of p in the profile-formula. The given relations can serve only as a first indication and should be applied for wave attack well beyond the limit of stability, see section 9.4.2.

Clay

In cohesive material, like clay, there is no such thing as an equilibrium profile. Erosion will go on when wave attack remains present. There is no build up of the slope as is the case with sand and L_e grows with time without reaching a final value. In that case, a protection will usually be necessary.

******** In the final version more information on this subject will be included ********

9.5 Soilmechanical aspects

9.5.1 General

******** Soil mechanical aspects play an important role in the design of coastal structures. In the final version of the pedoman, this has to be included ********

9.5.2 Macrostability of slopes

******** Mainly slip circle stability will be dealt with in this section

9.5.3 Soft-soil engineering

******** This section will have to be drafted in cooperation with participants of the workshop on soft soil engineering.

9.6 *Revetments*

9.6.1 *General*

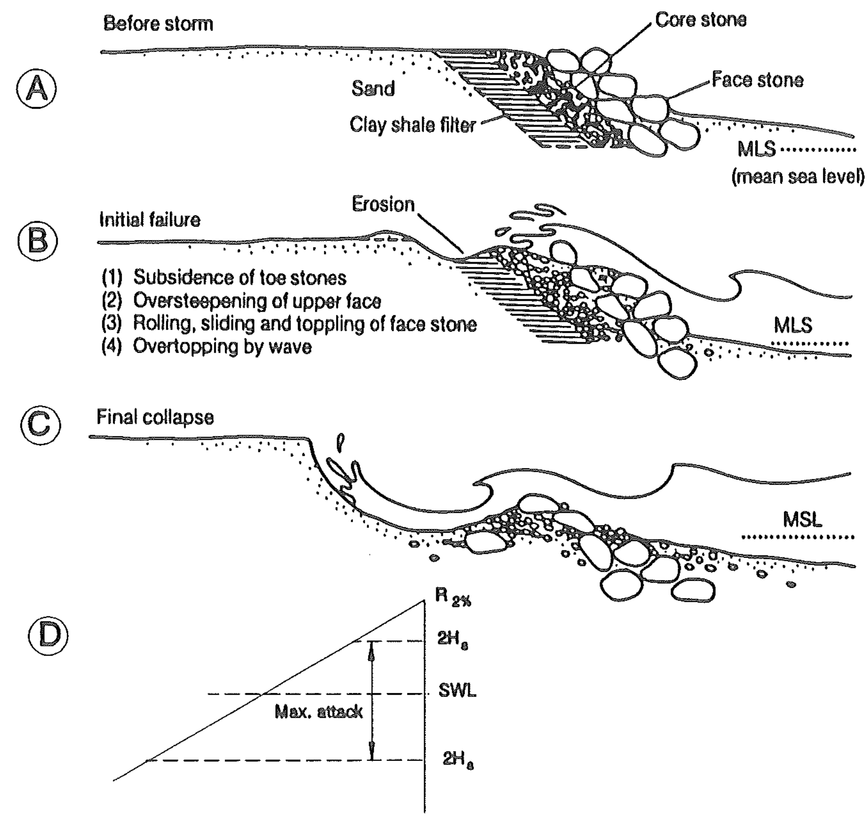


Figure 9.21 Ill-designed upper limit

The upper and lower limits of a protection on a slope are determined by the loads. Figure 9.21A through C give an example of an upper limit which was clearly not well-considered. The cover layer as such seems allright and also the filter design has been given attention. But the run-up of the waves, see chapter 4, is such that they attack the area on top where there is no cover and where the filter layers cannot stand the wave forces. Even the original unprotected soil is eroded by the waves and a total collapse is the result. The protection should either be made higher or strong enough behind the top to withstand the loads.

Usually the lower limit of a protection on a slope is taken 1.5 H_S to 2 H_S below SWL, see Figure 9.21D. In a tidal area, SWL of course can vary considerably, which has to be taken into account. Below the lowest waterlevel, the protection can be dimensioned for possible currents. In relatively shallow water, which is often the case, the protection extends to the toe of the slope. The upper limit can be taken as the $R_{2\%}$, although the protection in the uppermost half is not necessarily as heavy as around SWL.

9.6.2 Toes

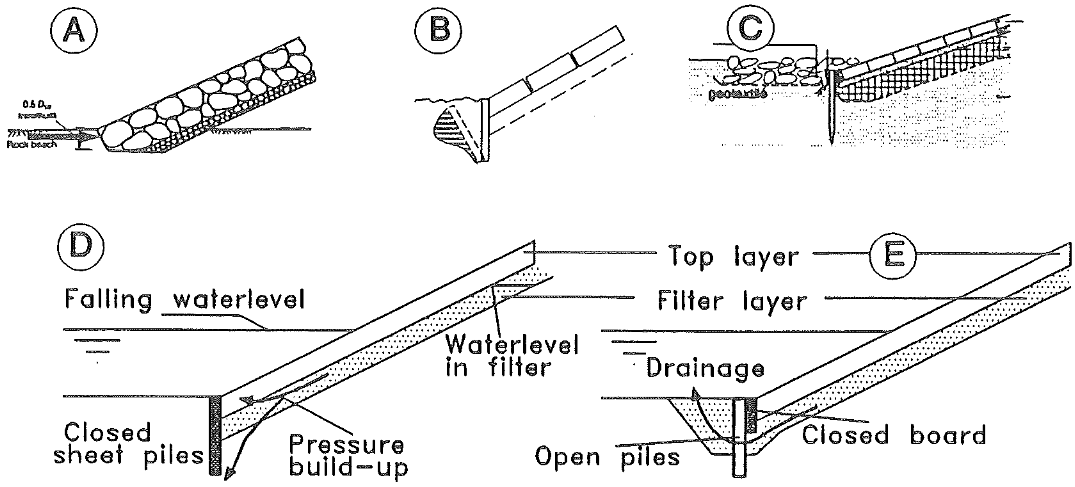


Figure 9.22 Revetment toes

Toes are a transition from a slope to a horizontal part. The primary function is the support of the revetment. When the friction along the slope becomes too little, either due to a drop in the waterlevel or to waves, the toe has to deliver the resisting force. So, it should be able to withstand a horizontal force. In rock bottom this is very easy and very much like the connection of two wooden roof beams, see Figure 9.22A.

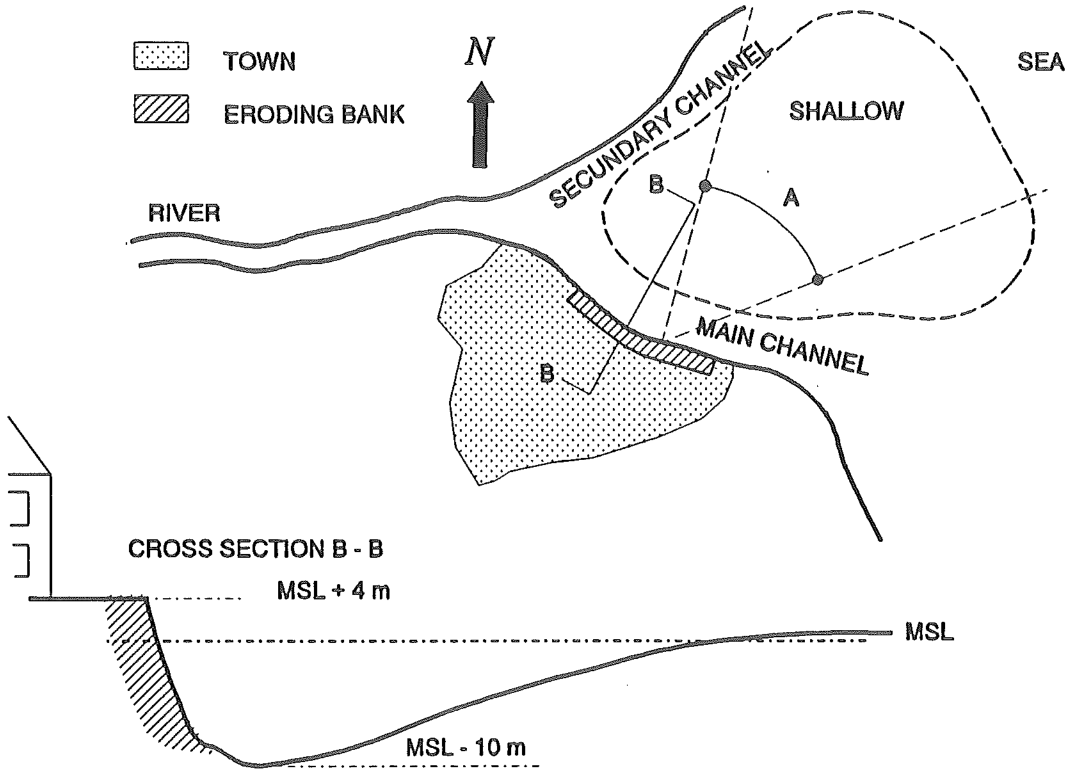
In soft(er) bottoms, toes are often made of piles, sheet piles, strips or combinations. It should be remembered that those can only give a reaction force by means of passive soil pressure for which some displacement is necessary, see Figure 9.22B, leading to deformation of the revetment, leading to seams etc. Therefore when considerable sliding forces can be expected, a layer of stone in front of the toe might help, see Figure 9.22C.

When the toe is threatened by scour, some bed protection is needed, like a mattress covered with stones, which can be also useful for the above mentioned horizontal forces.

When the toe consists of a closed layer or is constructed with sheet-piles, the pore pressure under it can form a threat to the soil directly beside the toe, see Figure 9.22D. The solution can be a combination of piles, a strip and some granular filter with enough space between the piles to let the pressure escape, see Figure 9.22E.

9.6.3 *Example*

INTRODUCTION



Situation of river bank located at river mouth

The waterfront of a town, located on the outer bank of a river mouth, is subjected to erosion. The groundlevel of the town is high enough to prevent flooding, but the erosion is threatening the road along the waterfront.

The average depth of the river at the eroding location is about 5 m below MSL, but near the town, in the outer bend, it is 10 m. The erosion process has not stopped and shifts the main channel continuously to the south. The main channel is 500 m wide at MSL.

Waves enter the area from sector A. Their height is limited by a shallow area in the river mouth, which lies approximately 0.5 m above MSL. The river bed and banks consist of sand with a diameter of 300 μm.

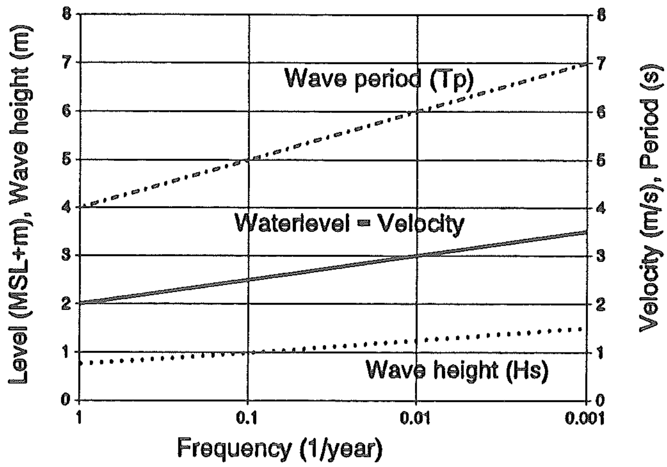
BOUNDARY CONDITIONS

The following table and graph give the boundary conditions for this case.

Exceedance frequency (1/year)	Waterlevel (MSL + m)	Wave height Hs (m)	Wave period Tp (s)	Maximum ebb flow velocity during tidal cycle (Q/A in m/s)
Average HW	1.0			
Average LW	-1.0			1.5
1	2.0	0.75	4	2.0
0.1	2.5	1.0	5	2.5
0.01	3.0	1.25	6	3.0
0.001	3.5	1.5	7	3.5

Note:

- The waves are correlated with the extreme waterlevels (a high waterlevel is also caused by the wind).
- The velocities are given as the discharge divided by the area (Q/A) and are not correlated with the high waterlevels (the location is very close to the sea and a high river discharge or high tidal range is independent of the average waterlevel at sea). In a tidal cycle these velocities occur around MSL.



Boundary conditions

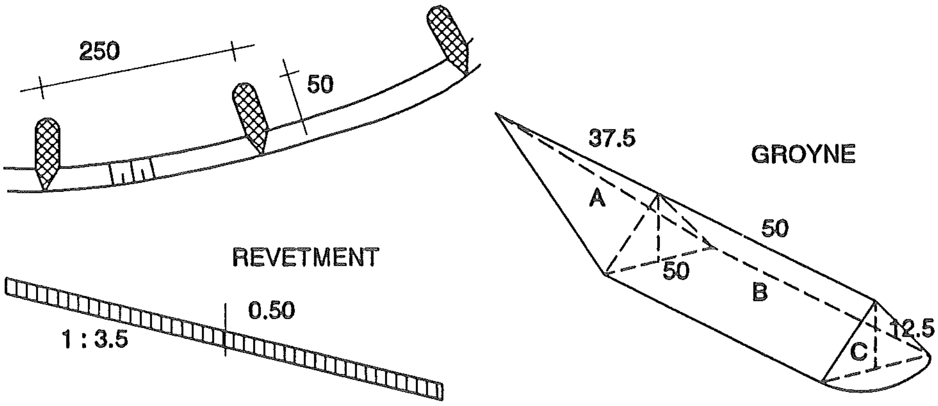
DESIGN FREQUENCY

There is no danger of flooding, the bank protection can easily be inspected and repaired and damage of the road will not lead to great danger of the buildings at the other side of the road immediately. With inspection every year and a accepted chance of exceedance of 10%, this leads to a design frequency of 0.1/year.

LAY OUT

There are several ways to protect the river bank against erosion, e.g. by means of groynes or with a revetment from rock. At first we will make a rough comparison between the two.
To keep the main flow from the bank, the groyne should have a length of at least 1/5 of the distance between the groynes. A distance of 250 m and a groyne length of 50 m is taken. For the slopes 1:2 is chosen (rather steep to save material). The crest of the groynes is taken equal to the waterlevel of the design flood to prevent high velocities directly along the bank. For the slope of the (unprotected) bank between the groynes, 1:3 is taken.

For the dimensions of a revetment, we need the width and the thickness. The width is determined by the slope. An important parameter for the slope of the revetment is the possible seepage. Due to rainfall and/or to fluctuating waterlevels in the river, seepage is almost always present. In that case, equation 11.13 gives simply: $\alpha <= \phi/2$. From figure 9.8 we find for sand with d_{50} = 0.3 mm: $\phi \approx 30^0$. From this we find: $\alpha <= 15^0$, which means a slope of 1 : 3.5. The thickness depends on the diameter which still has to be determined. For now we make a safe estimate: 0.5 m thick.



Groynes or revetment

It is difficult to compare all aspects of the two solutions, groynes and revetment. Costs and construction method, for example, heavily depend on local circumstances. Therefore, the comparison is made, simply based on quantities of material. For the groyne, the volume of stone for 250 m bank length (one groyne for 250 m bank) is: A + B + C = 1/2 x 1/2 x 12.5 x 50 x 37.5 + 1/2 x 12.5 x 50 x 50 + π x 25^2 x 12.5 / 6 = 5860 + 15625 + 4090 = 25575 m^3. For the revetment, the volume of stone per 250 m bank length then becomes: 12.5 x $\sqrt{(1^2 + 3.5^2)}$ x 250 x 0.5 = 5690 m^3.
Groynes need over 4 times more stone (the volume of stone is quadratic with the depth for groynes and linear for revetments, so only in shallow water groynes will need less stone), but we neglect now the material to make the slope for the revetment 1:3.5 and the extra protection at the toe of the revetment to withstand continuing erosion. On the other hand bottom protection at the head of the groyne will be necessary and it can be expected that the construction costs also will be higher for groynes. Moreover, we will see that wave attack is an important load on this bank, so between the groynes an additional protection will be needed. So, the choice is for a revetment (groynes are e.g. applied in a shallow river when a navigation channel with a certain depth is wanted).

TOP LAYER

The river bank is attacked both by flow and by waves. To start with the flow, we need the design velocity near the bank. The values from the boundary conditions can not be used directly, since they represent an average in the cross section (Q/A). Since the bank is located in an outer bend of the river, the velocity will be higher than this average. For a first estimate we take 40% higher, leading to a design velocity of 1.4 * 2.5 = 3.5 m/s (measurements are, of course, preferable and not so difficult to obtain!). We further assume that this velocity occurs at the average depth on the slope ($\approx$ 6 m during design conditions).

From the Shields equation, with $\psi = 0.03$ (see section 9.2.1) and $\Delta = 1.65$, we find iteratively with C = 18*log(12*h/k) and k = 2*dn: $dn = u^2/(C^2*\psi*\Delta) = 0.13$ m.
This is however for uniform flow in a flume and with a horizontal bed. The flow in a river is more turbulent than in a uniform flow, due to variations in cross section and the stones lie on a slope of 1:3.5. To take this into account we apply two correction factors, see section 9.2.1.
K(v) takes the extra load due to turbulence into account and is assumed 1.2.
$K(\alpha)$ takes the decreased strength into account and is calculated with: $K(\alpha) = \sqrt{(1 - \sin^2\alpha/\sin^2\phi)}$. For $\alpha = 16°$ and $\phi = 40°$ this leads to $K(\alpha) = 0.9$. dn then becomes: $K(v)^2*dn_{Shields}/K(\alpha) = 0.2$ m (K(v) is an amplification factor for the velocity, hence $K(v)^2$ is used for the diameter; $K(\alpha)$ is already defined as a correction factor for the diameter). From the curves in annex A we find that this is equivalent with stone 10 - 60 kg.

For the wave attack, we will use the van der Meer formula (for a rough first estimate also Hudson can be used, but for revetments, van der Meer is preferred since the porosity of a revetment is quite different from that in a breakwater, for which Hudson was derived).
With H = 1 m, $\cot\alpha = 3.5$, P = 0.1 (relatively short waves hardly penetrate into the subsoil, hence it is considered "impermeable"), Tm = 0.85 * Tp = 4.25, S = 2 (threshold of damage, since the chosen boundary conditions already have a rather high probability of exceedance: 10%) and N = 7000 (maximum value, for the same reason), we find: $\xi = 1.5$, while for the transition between the two van der Meer formulae $\xi t = 2.2$, so we are dealing with the formula for plunging breakers. With $\Delta = 1.65$ this leads to a dn = 0.38 m.
From the curves in annex A we find that this comes down to rock 60 - 300 kg.

It is obvious that the wave attack is dominating. The wave attack, however, does not act on the whole slope. The region of wave attack lies approximately 2*Hs around the waterlevel. For the final design we also have to take into account lower waterlevels (with lower wave heights). A practical measure (also for construction) is then that we apply 60 -300 kg from Mean Low Water (MSL - 1 m) up to MSL + design waterlevel + 2* design Hs = MSL + 4.5 m. Depending on the height of the bank, a practical transition to what lies above has to be made.
In this case we deal with a city waterfront with a road along the river, so, one can think of a wall of masonry. Below the low waterlevel, the attack on the slope comes from the flow only. So, down to the bottom, the stone 10 - 60 kg can be applied.

FILTER

Between the stone revetment and the original bank material, a filter will be necessary because of the turbulent flow, but most of all because of the wave-action. In that case the geometrical rules should be followed:

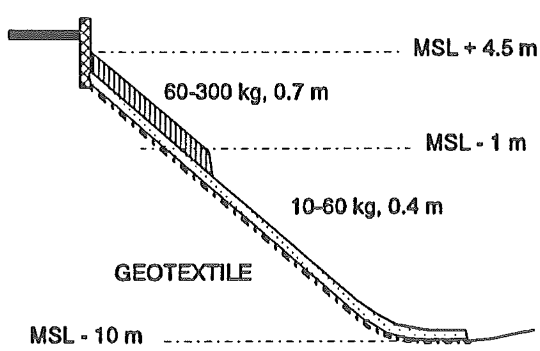
$d_{15F} < 5 \times d_{85B}$

From annex A, figure 56 we find for the small particles of the top layer ($dn_{50} \approx 0.38$ m, stone of 60 - 300 kg): $d_{15F} \approx 0.3$ m. The large particles of the second layer (d_{85B}) should be larger than 0.06 m. In that case, the smaller particles of the second layer will have a size of around 3 cm. This is still too large to lay on top of the original material (0.3 mm). So, more layers in between are necessary, leading to several filter layers. In that case, a geotextile can offer a good solution.
From the table in the section on geotextiles we find that the maximum opening in the textile (O_{98}) should be smaller than $2*d_{85}$ of the base material, leading to a value of about 0.8 mm.

A practical solution then becomes the following:
Take a geotextile along the whole slope, together with a layer of the stones 10 - 60 kg. When these are not too sharp, they can act as a weight for the geotextile, without damaging the textile. In the zone of wave attack, the rock 60 - 300 kg is applied.

SCOUR AND TOE

As already said in the introduction, the erosion process of the outer bend of the main channel goes on, so at the toe of the slope, scouring also will go on. Since the shift southward is now blocked, the erosion will now cause deepening of the bottom at the toe. From morphological computations, an estimation can be made of the course of this process in the years to come. Several measures are possible to deal with this problem:



Final design

- excavate down to the expected scour depth and continue the revetment
- apply a falling apron
- continue the revetment on the bottom of the river, assuming that the geotextile and the stones on top of it are able to follow the erosion process

Both the falling apron and the third solution will need inspection, e.g. every year or after a high discharge or extreme tide. The third solution has been chosen here and the figure above shows the "final preliminary" design. This is only a rough sketch without any detail. These can vary, depending on the construction method. For example, as a transition between waterborne and land-based operations, a berm could be useful.

9.6.4 *Placed block revetments*

An important material in revetments, especially in sea defences, is formed by placed elements, mostly concrete. Much research has been done in the last ten years in the Netherlands on the stability of placed blocks on a slope under wave attack. Therefore, this construction type will be discussed here in more detail.

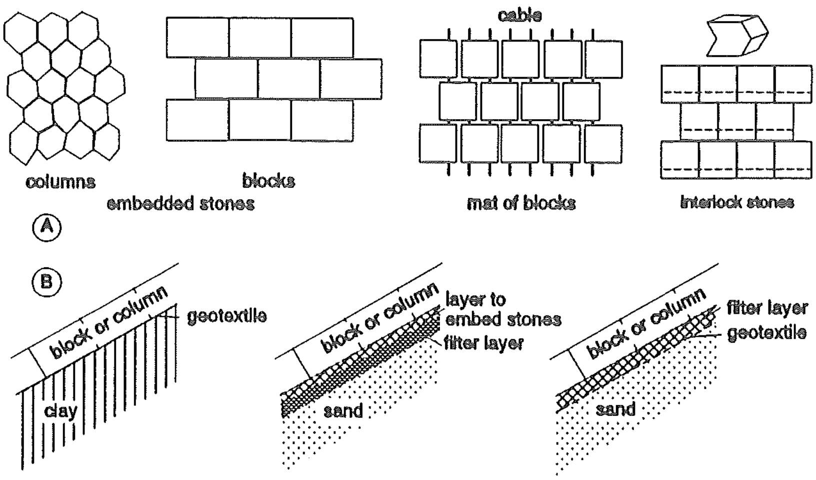


Figure 9.23 Block types and filters in revetments

Placed blocks come in many shapes where only the human phantasy is the limiting factor. The variation is in the coherence: pinched, connected with cables or geotextile or interlocked. Another variation is the shape of the upper side of the blocks which is meant to reduce the wave run-up; for this chapter this is not so important. Figure 9.23A gives some examples. The transition between the blocks and the underlying soil is another variable. Figure 9.23B gives some possibilities (many other combinations are possible). Sometimes the blocks are placed directly on clay; this requires a high quality standard during construction.

Stability mechanism

Figure 9.24A shows the phenomena that play a role in the stability of placed blocks during a wave cycle. For loose grains we saw in section 9.4.2 that beside velocity, friction and gravity, inertia and porosity play a role. For placed blocks the situation is all the more complex and much energy has been put into the fathoming of the secrets of placed block stability. From tests, calculations and reasoning it was established that phenomena b and c are dominant in the process. In fact, the wave action on and under the blocks can not be separated and porous flow phenomena have to be taken into account.

Figure 9.24B shows the dominating situation at maximum downrush: the pressure <u>on</u> the blocks is low in front of the wave, while <u>under</u> the blocks it is high, due to the propagating wave pressure in the filter-layer and to the relatively high phreatic level in the slope. This causes an uplift-force on a block. Whether a block is pushed out or not depends further on the strength of the revetment in which two factors play an important role, see Figure 9.24C. In the first place of course the coherence of the blocks, which is

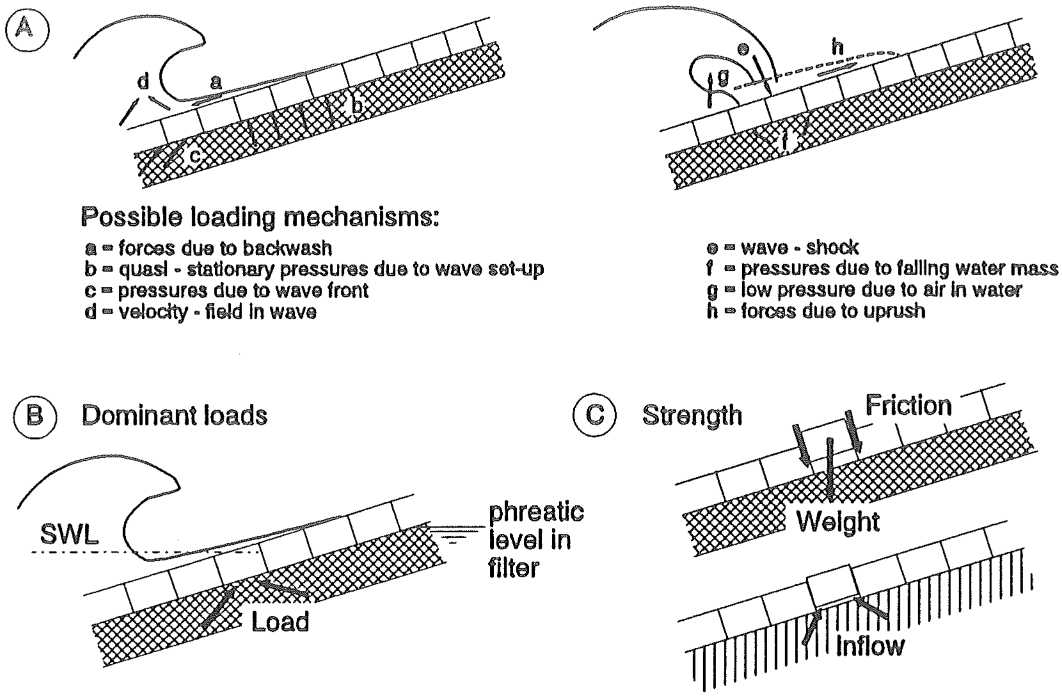


Figure 9.24 Load and strength of placed block revetments

in the case of placed blocks the friction between them and, even more important, the clamp-phenomenon which occurs when a block gets stuck between neighbouring blocks. The second factor is the flow towards a stone when it is pushed out. With a relative small permeability of the filter layer, the block is sucked on the slope because only very little water can flow into the growing hole leading to a sudden decrease of the pressure under the block, see also Figure 9.24C.

It will be obvious that, with so many assumptions and so many factors, it is impossible to give more than proportionalities in the relation between all the parameters involved. For various placed block revetment types, experiments were done to establish the constants of proportionality, leading to graphs as presented in Figure 9.25A, which serve as a, very rough and first, indication of the stability. In Figure 9.25b the results for placed blocks are compared with the stability of loose stones.

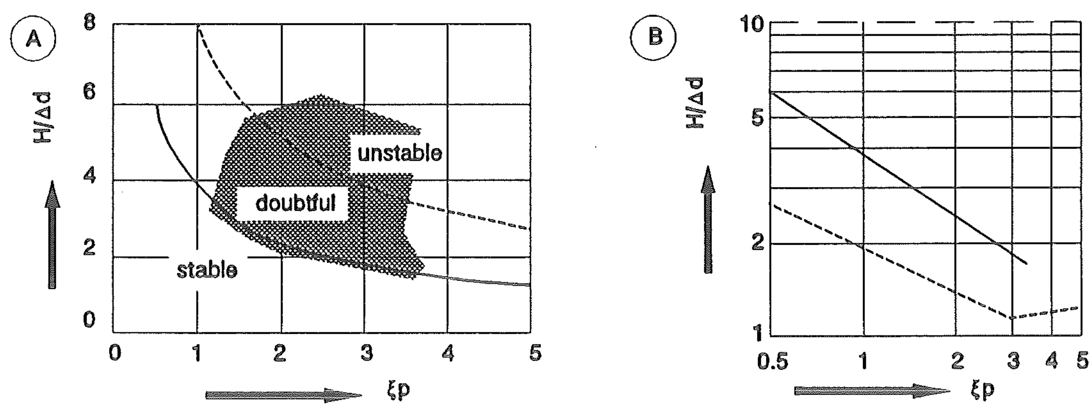


Figure 9.25 Test results for placed blocks on filter layer and comparison with loose rock

9.7 Groynes and (rubble mound) breakwaters

9.7.1 General

Groynes

As said before, groynes are only effective when a coast erodes due to longshore sediment transport. With transport perpendicular to the coast, groynes do not have any positive contribution at all. This again shows that knowledge of the morphological process is of utmost importance. When applied, groynes stretch from the upper part of the beach (preventing outflanking which can lead as far as to the toe of the dunes) till the necessary depth, which depends on the sediment transport and the width of the beach that is wanted. Lengths of 25 to 100 m outside the low water line are normal. To be effective, the distance between them should be 1 to 3 times the length.

On mud coasts, groynes should be used with care. Little is known up to now, of the morphology in such a case. When the groyne(s) create an area with very low currents and waves, siltation of this area will happen.

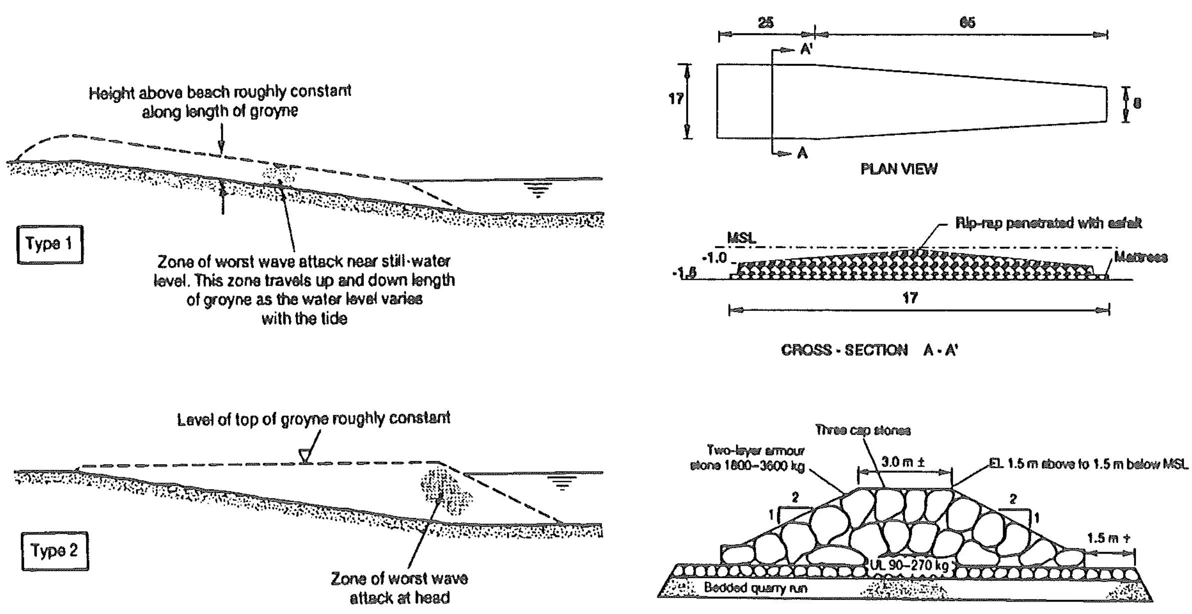


Figure 9.26 Groyne types

In general a groyne should not be made too high, in order to prevent unnecessary blocking of currents (scour!) and wave attack. To give an idea: 0.5 to 1 m above the wanted beach level. In that case the zone of attack goes up and down with the waterlevel, see Figure 9.26 type 1. A simple way to make such a groyne is by means of (penetrated) stones on a mattress (either made of natural material or geotextile). Penetration can be beneficial here: the stability of the stones is increased tremendously, while large overpressures due to head differences are not to be expected.

When the groynes are necessary to keep the current from the shore at high waterlevels, the crest will be about horizontal; in that case the attack is concentrated at the head and provisions against scour at the head should be taken (Figure 9.26 type 2). A type 2 groyne is very much like a breakwater. The geometrical filter rules should be followed here because of the severe dynamic loading, see section 9.3.5.

Breakwaters

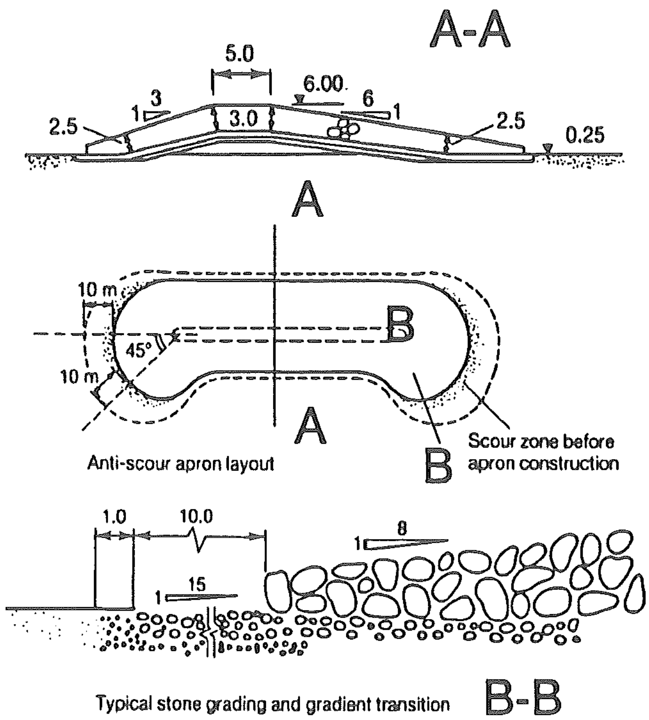


Figure 9.27 Offshore breakwater

Breakwaters will be mentioned here only briefly. Figure 9.27 gives an example of an offshore breakwater to protect a local spot. The sea-side slope is gentle (1:6), to reduce wave reflection to reduce scour in front of the breakwater. The round head has a diameter equal to about the wave length and a slope of 1:8 to increase stability of the coverlayer, but particularly to prevent eddies with a vertical axis, again to reduce scour. Even then, an apron is necessary. The height of the breakwater is usually chosen at HW level or 1-2 m above that.

Breakwaters can also be meant to reduce waves at a harbour entrance. The structure as such does not differ from a shore protection breakwater, nor do the morphological consequences. The nautical aspects of the lay-out of the breakwater design, however, are outside the scope of this manual.

9.7.2 *Stability*

Section 9.4.2 was about the stability of rock in breaking waves on a continuous, non-overtopping slope. In this section, the focus will be on some specific locations, for which the relations of the previous section will give either too optimistic or too pessimistic results for design. These locations are toes, overtopping crests and heads of breakwaters.

Toes

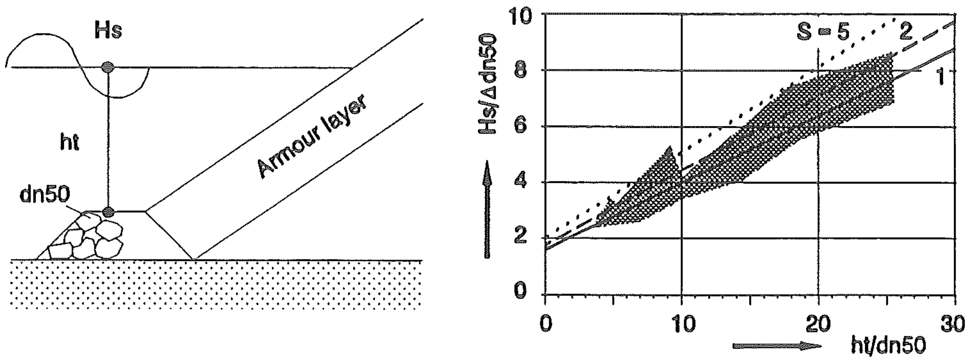


Figure 9.28 Stability of stones in toe

The function of toes is mainly to support the armour layer of a slope. The stones in a toe can often be smaller than those in the armour layer, since they are less heavily attacked by the waves. With relatively deep lying toes, the waves even do not break. Figure 9.28 gives the results of experiments on toe stability, see vanderMeer et al,1995. The lines in the figure represent the relation:

$$\frac{H_s}{\Delta d_{n50}} = (0.24 \frac{h_t}{d_{n50}} + 1.6) S^{0.15} \qquad (9.32)$$

(vanderMeer uses a slightly different definition for S in this case, which has been neglected here). From this relation can be seen that considerable reduction on the necessary stone diameter is possible, when the toe lies relatively deep. When the toe reaches the waterlevel, the minimum value of $H_s/\Delta d_{n50}$ becomes 1.6 - 2, which is equal to values that follow from equation (9.28), see also Figure 9.14. This is logical, since the difference between toe and slope vanishes in that case.

The experiments in Figure 9.28 were done for breakwaters with a slope of 2:3, which is rather steep. The influence of slope variation was not studied. Toes at vertical walls lie in the antinode of a standing wave where, theoretically, only the pressure fluctuates. Wider toes come in the neighbourhood of the node where the velocities have their maximum. For non-breaking waves, it is recommended to follow the procedure with the modified Shields expression of section 9.4.2. For shallow water, with breaking waves, no higher values than $H_S/\Delta d_{n50}$ = 2, should be used. For more detail see CUR/CIRIA,1991.

Low crests

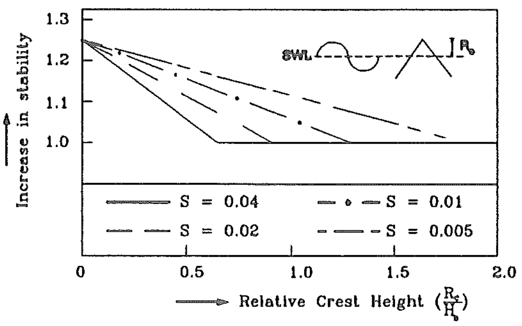


Figure 9.29 Stability increase for low crest

The van der Meer and Hudson formulae were derived for non overtopping slopes. When this is not the case, there is a certain wave transmission. That means that not all energy will be destroyed on the slope and the stability increases.

For crests <u>above</u> the still water level van der Meer found a stability increase factor:

$$K_{H/\Delta d} = 1.25 - 4.8 \frac{R_C}{H_S} \sqrt{s_p / 2\pi} \tag{9.33}$$

where R_C is the crest height with respect to SWL, see Figure 9.29 and s_p is the wave steepness related to the peak period; the minimum value of the stability increase factor is 1. As a result of the wave transmission, however, on the other side of the breakwater, armouring needs to be heavier and the question is whether the total damage will reduce, see also Burger,1995. For crests <u>below</u> the still water level van der Meer derived a reduction for the necessary Δd, see Pilarczyk,1990

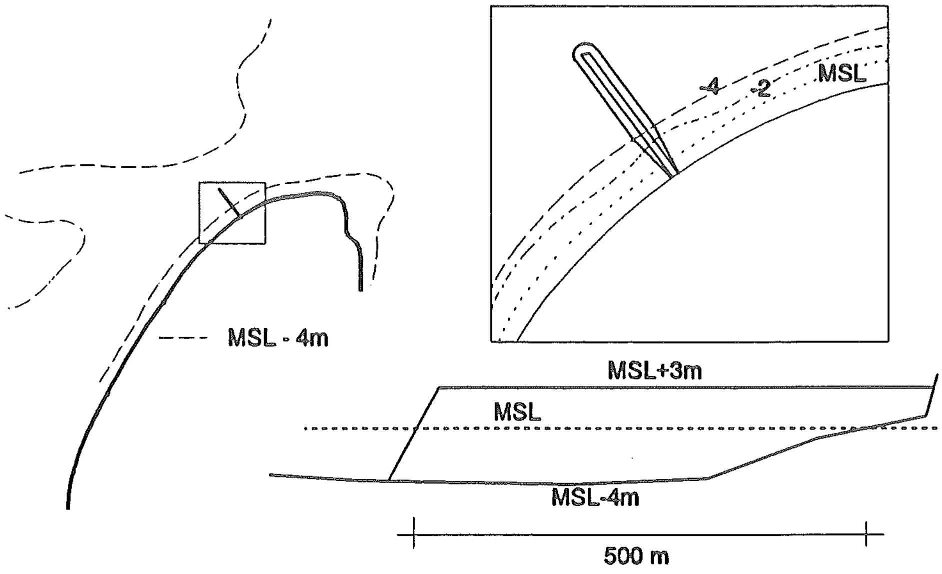
These reduction formula have been established separately and should be used with caution. Further research is needed in order to establish an overall relation for stability increase of low crested dams.

Heads

In contrast with the two previous examples, which were more favourable compared to a stone on a slope, stones on the head of a groyne (or breakwater, again for which the investigations were done) are attacked heavier. No systematic test results are available. The Shore Protection Manual (SPM,1984) recommends a mass at the head twice as high as at the trunk, or as an alternative, a twice gentler slope.

9.7.3 *Example*

INTRODUCTION



Situation coastline

On a sandy coast, a 500 m long breakwater will be constructed to influence the longshore sediment transport, see figure. The crest will be made at MSL + 3 m with a width of 5 m, for reasons of accessibility for maintenance and fishing. The subsoil consists entirely of sand of 200 μm, which is densily packed. The manager of this coastline makes yearly inspections and does not want too much maintenance, so the design will be made for circumstances with an exceedance frequency of 0.01/year.

BOUNDARY CONDITIONS

In the following table and figure, the most important boundary conditions are presented. The waterlevels come from observations and extrapolation. The wave height is calculated as follows: the location where the breakwater is going to be built, is sheltered from the prevailing winds by shallow areas with a depth of about MSL - 2.5m. In the future, due to morphological changes, a lowering to MSL - 3m is possible. Wave heights under extreme conditions will therefore be depth-limited and are estimated from: Hs $\approx$ 0.5 * (3 + waterlevel).

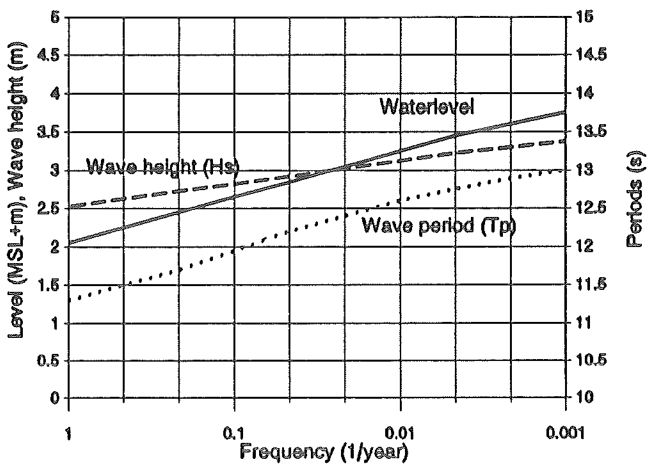


Figure 9.31 Boundary conditions

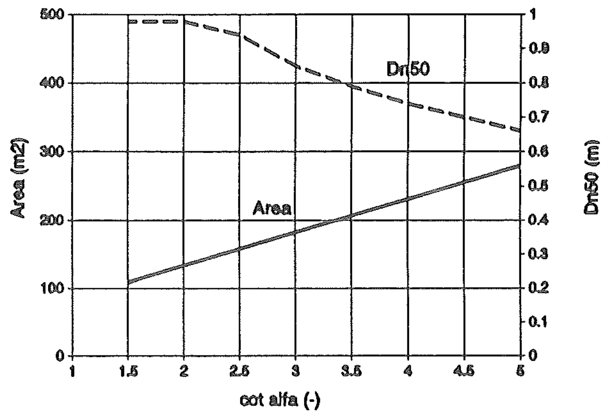
Exceedance frequency (1/year)	Waterlevel (MSL + m)	Wave height Hs (m)	Wave period Tp (s)
Average HW	0.8		
Average LW	-0.9		
1	2.05	2.52	11.3
0.5	2.25	2.62	11.5
0.2	2.45	2.72	11.7
0.1	2.65	2.82	12.0
0.05	2.85	2.92	12.2
0.02	3.05	3.02	12.4
0.01	3.25	3.12	12.6
0.005	3.45	3.22	12.8
0.002	3.60	3.30	12.9
0.001	3.75	3.38	13.0

The wave periods come from observation in deep water. The assumption is that the peak period of the wave spectrum is not changed by the shallows. This is on the safe side, since the shallow areas will generate higher harmonics in the waves, decreasing Tp somewhat, while a longer period is here unfavourable in the stability calculations.

Under average conditions, the waves are not depth limited. So, for the construction period, wave data are necessary. It appears that, during the summer season (in which the breakwater will be built), 20% of the time waves higher than Hs = 1 m with Tp ≈ 5 s occur; 10% of the time Hs > 1.5 m with Tp ≈ 6 s and 1% of the time Hs > 2 m with Tp ≈ 8 s.

TOP LAYER

CROSS-SECTION



Cross-sectional area and diameter

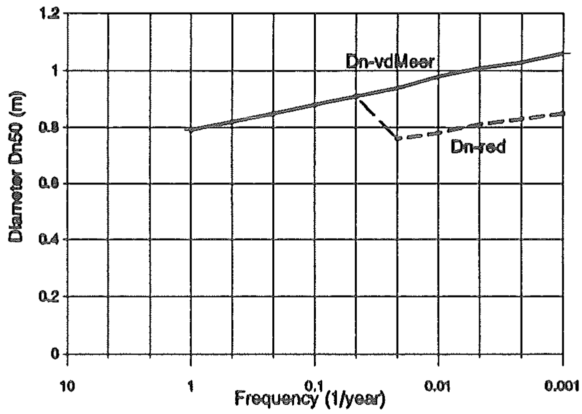
At first, the slope of the breakwater trunk will be determined. The gentler the slope, the smaller the stone that is needed for stability in waves, but the larger the total quantity of stones. The figure shows the result for a dam with a 5 m wide crest at MSL + 3m and a bottomdepth of MSL - 4m. The stability was calculated with the van der Meer formula for 0.01/year conditions. From a slope of 1:1.5 and 1:2, there is a transition between surging and plunging breakers, hence the same calculated diameter. In general, the decrease of the diameter will not compensate the larger area: the price per ton for these stones will not differ very much. In this case, we assume there is also no significant difference in the price of placing the stones. A slope of 1:2 is chosen although 1:1.5 is cheaper. This is a preliminary design which should not be made too sharp for a first estimate of costs.

Warning:
This example comes from the Netherlands. The conclusion: "the steeper the cheaper" can change completely depending on the price and the availibility of rock of a certain diameter in the considered area. In that case the available diameter determines the slope. Another reason to avoid larger stones can be when the equipment to place the stones has its limitations. This is an important factor in the design of coastal structures in remote regions in Indonesia.

STONE DIAMETER

To calculate the necessary diameter of the stones in the top layer of the breakwater, the formula of van der Meer is used for conditions with a frequency of 0.01/year. From the boundary conditions we find: waterlevel = MSL + 3.25 m, Hs = 3.12 m and Tp = 12.6 s. In the van der Meer formula the period is not Tp but Tm, so a correction is made: Tm $\approx$ 0.85Tp = 10.7 s.
Other parameters in the formula are:
P (porosity): for a breakwater with a permeable core a value of 0.5 is recommended
Δ (relative density): use is made of basalt with a ρ = 3000 kg/m3. With a seawater density of 1020 kg/m3, this leads to Δ = 1.95.
S (damage): some damage from the design waves is allowed, but removal of the top layer should be avoided, so a value of 4 is used.
N (number of waves): the maximum number is used (N = 7000) since storms in the area can last for several days.

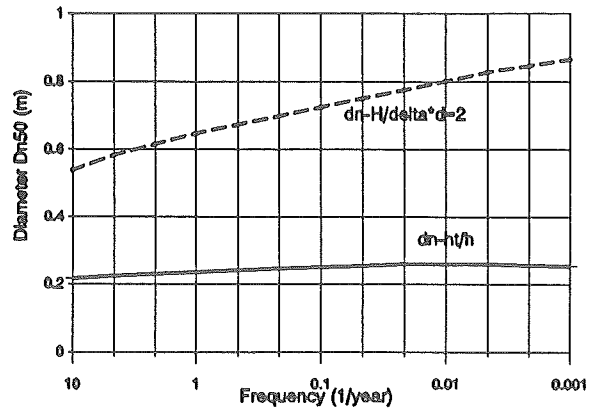
From all this we find: ξm = 3.79, while the transition between the formulae for plunging and surging waves is given by equation 10.5: ξm = 3.54, so we have to use the equation for surging breakers (10.4b), leading to dn = 0.98 m. A reduction factor can be applied, since the waterlevel is higher than the crest level of the breakwater, see equation 10.6 in figure 10.12. Since the crest is under water, we assume the maximum effect of this equation: 1.25 increase in strength or a reduction of 0.8 on the diameter, leading to dn = 0.78 m.



Calculated diameters

Now, however, the question arises whether this wave condition is dominant, since the breakwater crest is under water and with lower waterlevels and waves, the reduction can not be applied, possibly leading to larger stones. The figure shows the result for all waterlevel and wave conditions. It appears that up to a frequency of 0.05/year, there is no reduction, while for more extreme values, the crest disappears under water leading to the maximum reduction. So, the 0.05/year conditions are dominating leading to a dn = 0.91 m. From the curves in annex A, we find that this equivalent with rock 1000-3000 kg.

BERM



Diameter on berm

For support of the rocks at the toe, a berm is recommended (otherwise, stones at the toe will easily move under wave attack). To determine the necessary diameter for the berm, this figure gives some information. For waterdepths of 6-8 m and a berm height of 1.5 m, we find values of $H/\Delta dn$ between 5 and 7, leading to a necessary dn of around 0.25 m. There is however a warning with this figure, namely that for shallow water, $H/\Delta d$ should be taken not higher than 2. Another reason to use larger stones is that the diameter of stones of a berm should not be less than one class lower than the top layer in order to provide sufficient side support, hence with 1000-3000 kg as a top layer, the berm should be made of at least 300-1000 kg. For practical reasons, in order to avoid all different kinds of stones, we will use use the same stones as those in the top layer (1000-3000 kg).

HEAD

The head of the breakwater is attacked more heavily by the waves than the trunk. To account for this effect, there are two possibilities: a gentler slope or a larger stone. Again, a steep slope is cheaper, provided the stone weight does not become prohibitive, but means an extra class of stone. Here for the head, rock of 3000-6000 kg will be used instead of 1000-3000 kg.

CORE

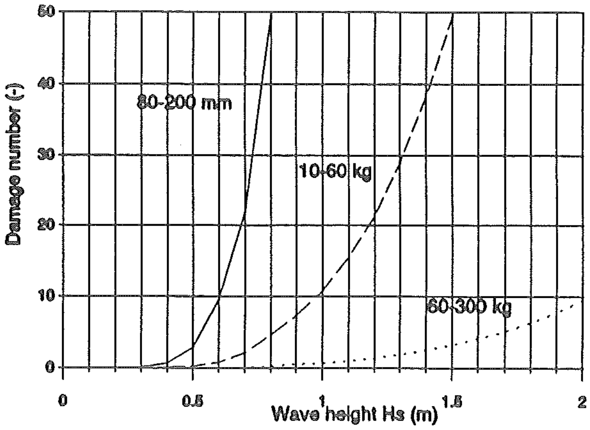
FILTER RULES

The main function of the breakwater is to interrupt the sediment transport locally. Therefore, finer material inside the dam is necessary, since rocks with a diameter of around 1 m will cause much turbulence inside the dam, making the sandtightness questionable. Another reason to use finer material can be that it is cheaper (when there is a large difference in diameter); on the other hand it is more complicated to build the dam from different materials, leading to more actions.

Applying geometrical filter rules (dynamic loading!), we find from the curves in annex A for the 1000-3000 kg rock a d15 of 0.7 m. For stability, this leads to a minimum d85 for the next layer of 0.14 m corresponding with stones of 80/200 mm as an absolute minimum. For the construction and friction between the layers a somewhat larger stone would be preferable, leading to 10-60 kg.
The final choice for the cross-sectional build-up of the breakwater will be made after the construction is considered.

CONSTRUCTION

The stability of the core material in the construction phase should be considered, since too many losses of dam material will cost a lot of time and money. In the paragraph on boundary conditions it was said that waves of 1 to 1.5 m can occur during construction.
Figure 10.4 shows the damage number in the van der Meer formula as a function of the wave height. In order to calculate S, the van der Meer formula was rewritten in the shape:
S = f(H,P,N, ... etc.). For the number of waves N = 2000 was taken and Tm = 6 s (a summer storm of 3 - 4 hours). A damage number of 25 already means considerable flattening of the slopes.



Possible damage during construction

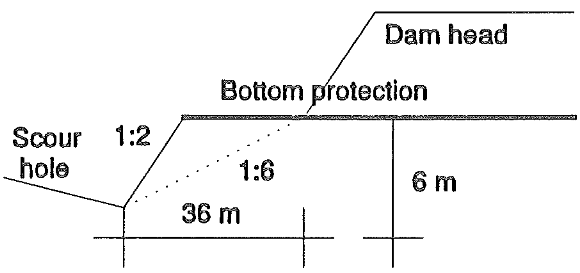
From this result we can see that the core material is only stable at very small wave heights and that even rock of 10-60 kg is easily damaged with wave heights between 1 and 1.5 m. Therefore, 60-300 kg will be used on top of a core of stones of 80-200 mm. The dumping of the core material will be followed immediately by the second layer. After that it seems possible to leave the dam for some weeks before the top layer is constructed.

BOTTOM PROTECTION

WIDTH

The functions of the bottom protection are mainly:
- to keep scour holes away from the dam
- to make a transition between the large rocks of the top layer and the sandy bottom

For the first function, the bottom protection should be wider than the dam itself. Scour can come from tidal currents or rip currents and is not easy to determine. A first rough estimate can be made, by considering the dam as a blunt abutment, leading to a scour depth of about 1.5 times the waterdepth. Maximum flow due to the tides usually occurs around MSL, giving a scour hole of 6 m deep. With an assumed slope of 1:2 and an end slope after possible sliding (densily packed sand), the protection should be at least 24 m from the toe of the dam.



Bottom protection

TOP LAYER

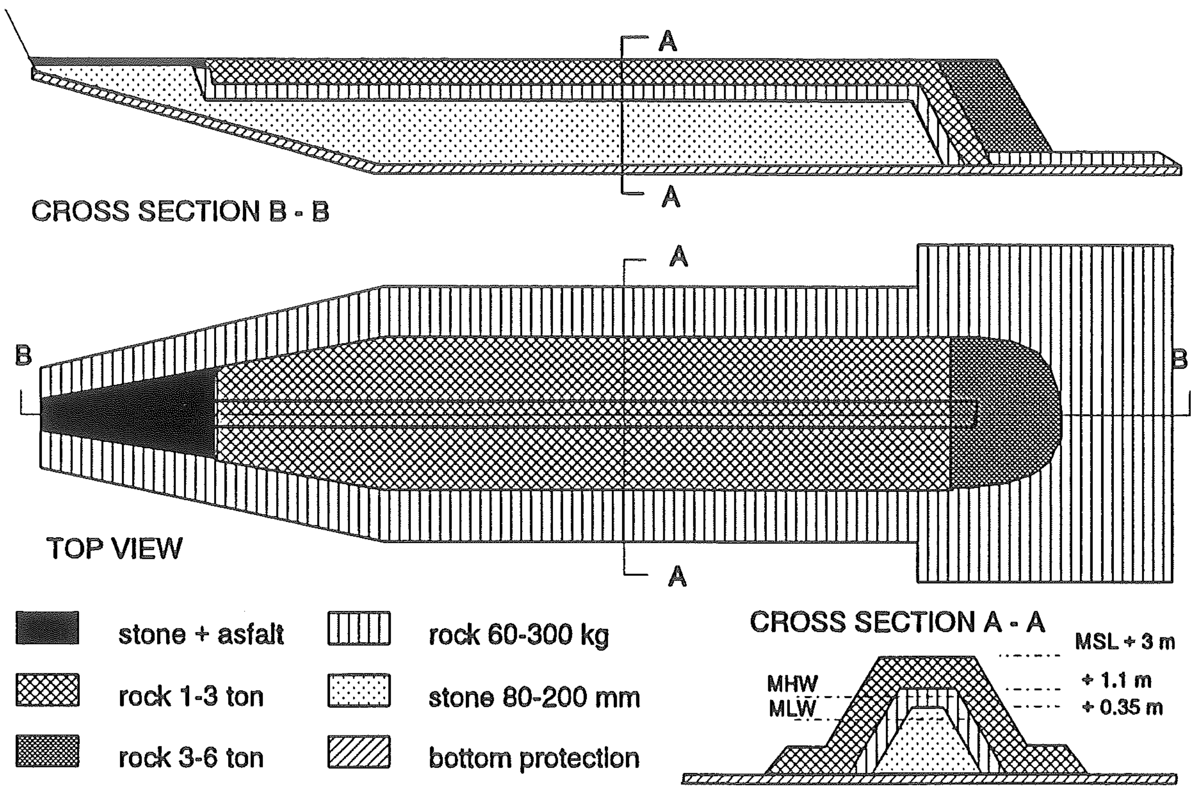
The second function can be fulfilled by making a geotextile, reinforced with willow faggots and covered with stones. These stones are necessary to keep the mattress-structure on the bottom before it is covered with the final structure. In the final stage, the top layer of the bottom protection should withstand the loads, in particular the wave shear and the tidal flow around the head.

The necessary stone size can be calculated from the modified Shields curve by Sleath, see Figure 9.13. To calculate the shear stress, we use the linear wave theory with the approach of Jonsson, see the table in chapter 4 and Figure 9.11. For 0.01/year conditions we find:

Hmax = 1.75 * 3.12 m ≈ 5.6 m (maximum wave in a wave train of 1000 waves with Rayleigh distribution, modified with shallow water correction, see figure 3.16), Tp = 12.6 s and
h = 4 + 3.25 = 7.25 m.

From linear wave theory a wave length of 103 m in this waterdepth is found, with a bottom velocity of 3 m/s and a particle displacement of 6.15 m.

Iteratively it is found that ab/k = 9.55 (with k = 2 * dn), fw = 0.073 (in equation 3.12) and dn = 0.32 m (with ψ = 0.056 in figure 10.1). This means that rock of 60-300 kg is needed as a top layer on the bottom protection .



Final design of breakwater

This figure shows the "final preliminary" design. The various layers get a thickness of about 2*d. This results in 1.9 m for the 1000-3000 kg rock and 0.75 m for the 60-300 kg rock. The top layer on the beach is made of stones 80-200 mm penetrated with asfalt in order to make crossing with cars possible.

At the head, the rock 3000-6000 kg is constructed as a homogeneous layer and acts as an additional protection of this head.

Note:
It will be clear that there is still a lot to be optimized and investigated before the final design can be constructed. Details like the transition between the beach and the dam need attention. The construction method itself has to be worked out. An average HW-level of MSL + 0.8 m, a LW-level of MSL - 0.9 m and a bottomlevel of MSL - 4 m make both land-based and water-borne operations necessary. The core will be dumped by barges and completed with land-based equipment during the low water periods of the tidal cycle. The second layer of stones 60-300 kg will be constructed with land-based equipment in the same manner. The top layer can be made completely above high water level.

9.8 Seawalls and (caisson) breakwaters

"Seawalls" stands for al types of constructions which are attacked by and reflect waves: dikes, breakwaters, (partly) protected dunes etc.

9.8.1 Erosion

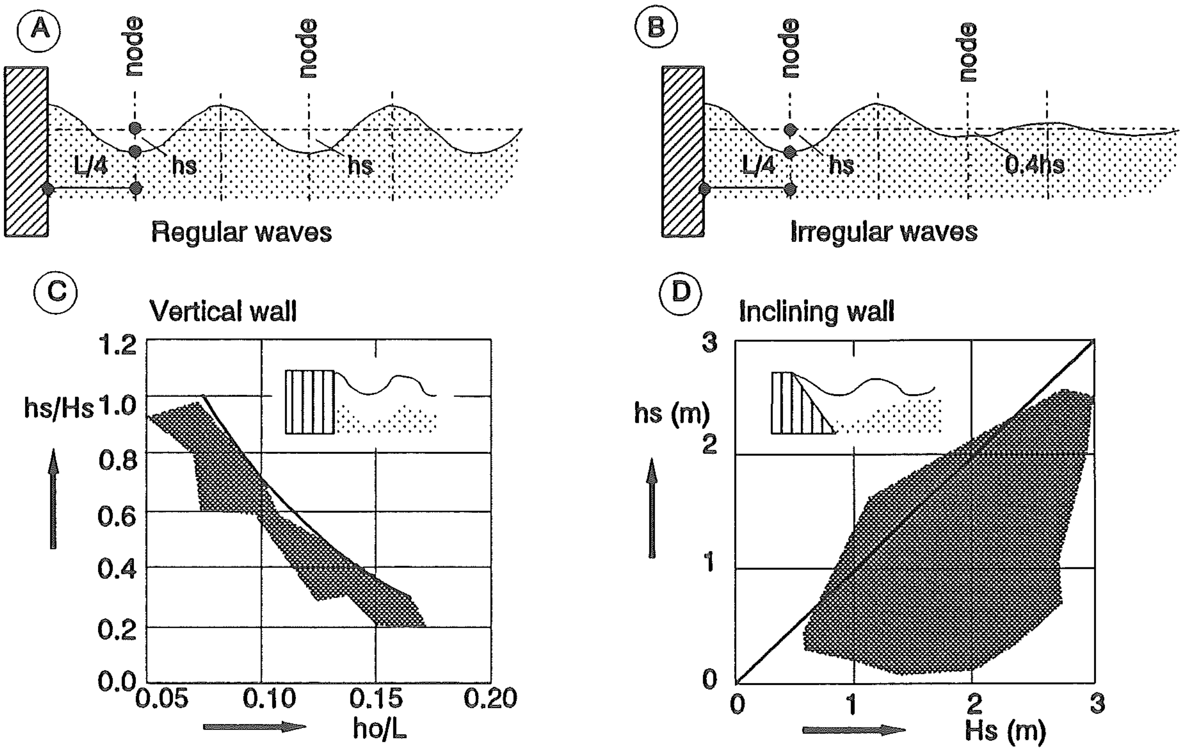


Figure 9.31 Erosion in front of seawalls due to waves

Bottom scour due to waves can be important in front of walls where a standing wave pattern is possible. From chapter 4 we know that the highest velocities occur in the nodes of such a wave and indeed, for fine sediments, the maximum scour is found there. For coarse sediments, however, the maximum scour is found between the node and antinode, probably due to another eddy and ripple pattern. The standing wave pattern for irregular waves is less distinct than for regular waves. This is also found in the scouring pattern, which decreases strongly with the distance from the wall, see Figure 9.31a and b.

The depth of the scour hole can roughly be estimated with (see Xie Shi-Leng,1981):

$$\frac{h_s}{H_s} = \frac{0.4}{\left(\sinh \frac{2\pi\, h_0}{L}\right)^{1.35}} \qquad (9.34)$$

This formula indicates that the scour depth depends on the wave height and the waterdepth related to the wavelength. Equation (9.34) is experimentally determined for fine sediment; for coarser sediment it can be seen as an upper boundary, see Figure 9.31c.

When the wall is not vertical but inclining, the maximum scour will occur at the foot of the wall, due to the backflow from the sloping wall, see Figure 9.31d. The erosion is roughly proportional to the reflection coefficient, so a vertical wall gives maximum values. Using H_S in equation (9.34), based on regular waves, overestimates somewhat the scour depth, but as a first guess for all cases, $h_s = H_S$ can be used as an upper limit. For reduction because of lower reflection or large waterdepth, this section can give some guidance for a preliminary design. In front of these constructions, an apron against scour is usually needed. The length can be about 3/8 of the significant wave length, covering the node at L/4.

9.8.2 *Stability*

The stability of a caisson is computed according to the method of Goda. The following background-text is based upon his presentation for the 1992 short course for the ICCE '92: "Design and reliability of coastal structures", published by Instituto di Idraulica, Universita di Bologna, Italy.

Design Wave

The upright breakwater should be designed against the greatest force of single wave expected during its service life. The greatest force would be exerted by the highest wave among a train of random waves corresponding to the design condition on the average. Thus the wave pressure formulas presented herein are to be used together with the highest wave to be discussed below.

(1) Wave height

$$\left.\begin{aligned} H_{max} &= 1.8 H_{1/3} && \text{for } \frac{h}{L_o} \geq 0.2 \\ &= \min\{(\beta_o^* H_o' + \beta_1^* h) \;,\; \beta_{max}^* H_o' \;,\; 1.8\, H_{1/3}\} && \\ & && \text{for } \frac{h}{L_o} < 0.2 \end{aligned}\right\} \tag{1}$$

$$\left.\begin{aligned} H_{1/3} &= K_s, H_o' && \text{for } \frac{h}{L_o} \geq 0.2 \\ &= \min\{(\beta_o^* H_o' + \beta_1^* h) \;,\; \beta_{max}^* H_o' \;,\; K_s H_o'\} && \\ & && \text{for } \frac{h}{L_o} < 0.2 \end{aligned}\right\} \tag{2}$$

in which the symbol min {a,b,c} stands for the minimum value among a,b and c, and H_o' denotes the equivalent deepwater significant height. The coefficients β_o and other have empirically been formulated from the numerical calculations data of random wave breaking in shallow water as follows:

$$\left.\begin{aligned}\beta_o &= 0.28 \left(\frac{H_o'}{L_o}\right)^{-0.38} \exp\,[\,20\,\tan^{1.5}\theta\,] \\ \beta_1 &= 0.52\,\exp\,[\,4.2\,\tan\theta\,] \\ \beta_{max} &= \max\,\{0.92,\ 0.32(H_o'\,/L_o)^{-0.29}\,\exp\,[\,2.4\,\tan\theta\,]\}\end{aligned}\right\} \tag{3}$$

$$\left.\begin{aligned}\beta_o^* &= 0.052 \left(\frac{H_o'}{L_o}\right)^{-0.38} \exp[\,20\,\tan^{1.5}\theta\,] \\ \beta_1^* &= 0.63\,\exp[\,3.8\,\tan\theta\,] \\ \beta_{max}^* &= \max\,(1.65,\ 0.53(H_o\,/L_o)^{-0.29}\,\exp[\,2.4\,\tan\theta\,]\end{aligned}\right\} \tag{4}$$

in which the symbol max{a,b} stands for the larger of a or b and tan Θ denotes the inclination of sea bottom.

The selection of the fixed relation $H_{max} = 1.8\ H_{1/3}$ outside the surf zone was based on three factors of reasoning. First, the fixed ratio was preferred to an introduction of duration-dependent relation based on the Rayleigh distribution of wave heights, because such variability in the design wave height would cause some confusion in design procedures. Second, the examination of prototype breakwater performance under severe storm wave actions yielded reasonable results of safety factor against sliding by using the above fixed relation. Third, a possible deviation of the ratio $H_{max}/H_{1/3}$ from 1.8 to 2.0, say, corresponds to an increase of 11% and it can be covered within the margin of safety factor which is customarily taken at 1.2. However, it is a recommendation and an engineer in charge of breakwater design can use other criterion by his own judgement.

For evaluation of H_{max} by the second part of Eq. 3 or within the surf zone, the water depth at a distance 5 $H_{1/3}$ seaward of the breakwater should be employed. This adjustment of water depth has been introduced to simulate the nature of breaking wave force which becomes the greatest at some distance shoreward of the breaking point. For a breakwater to be built at the site of steep sea bottom, the location shift for wave height evaluation by the distance 5 $H_{1/3}$ produces an appreciable increase in the magnitude of wave force and the resultant widening of upright section.

(2) Wave period

The period of the highest wave is taken as the same with the significant wave period of design wave, i.e.

$$T_{max} = T_{1/3} \tag{5}$$

The relation of Eq. 5 is valid as the ensemble mean of irregular waves. Though individual wave records exhibit quite large deviations from this relation, the use of Eq. 5 is recommended for breakwater design for the sake of simplicity.

(3) Angle of Wave Incidence to Breakwater

Waves of oblique incidence to a breakwater exert the wave pressure smaller than that by waves of normal incidence, especially when waves are breaking. The incidence angle β is measured as that between the direction of wave approach and a line normal to the breakwater. It is recommended to rotate the wave direction by an amount of up to 15° toward the line normal to the breakwave direction from the principal wave direction. The recommendation was originally given by Prof. Hiroi together with his wave pressure formula, in consideration of the uncertainty in the estimation of wave direction, which is essentially based on the 16 points-bearing of wind direction.

Wave Pressure, Buoyancy and Uplift Pressure

(1) *Elevation to which the wave is exerted*

The exact elevation of wave crest along a vertical wall is difficult to assess because it varies considerably from 1.0 H to more than 2.0 H, depending on the wave steepness and the relative water depth. In order to provide a consistency in wave pressure calculation, however, it was set as in the following simple formula.

$$\eta^* = 0.75\ (1 + \cos\beta)\ H_{max} \tag{6}$$

- For waves of normal incidence, Eq. 6 gives the elevation of $\eta^* = 1.5\ H_{max}$.

(2) *Wave pressure exerted upon the front face of a vertical wall*

The distribution of wave pressure on an upright section is sketched in the figure. The wave pressure takes the largest intensity p_1 at the design water level and decreases linearly towards p_1 at the design water level and decreases linearly towards the elevation η^* and the sea bottom, at which the wave pressure intensity is designated as p_2.

The intensities of wave pressures are calculated by the following:

$$\left.\begin{aligned} p_1 &= 0.5\ (1 + \cos\beta)\ (\alpha_1 + \alpha_2 \cos^2\beta)\ w_o\ H_{max} \\ p_2 &= \frac{p_1}{\cosh kh} \\ p_3 &= \alpha_3\ p_1 \end{aligned}\right\} \tag{7}$$

in which

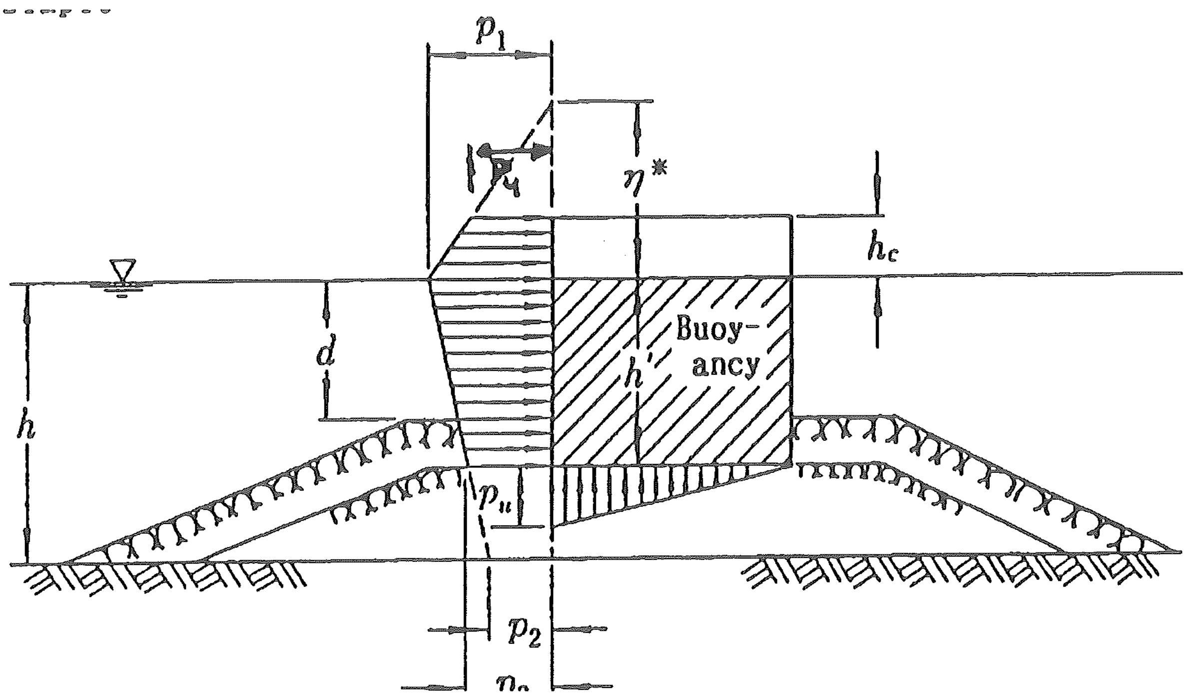


Figure: Wave presure distribution

$$\left.\begin{aligned} \alpha_1 &= 0.6 + 0.5\left[\frac{2kh}{\sinh 2kh}\right]^2 \\ \alpha_2 &= \min\left\{\left[\frac{(h_b - d)}{3h_b}\right]\left(\frac{h_{max}}{d}\right)^2, \ \frac{2d}{H_{max}}\right\} \\ \alpha_3 &= 1 - \left(\frac{h'}{h}\right)\left[1 - \frac{1}{\cosh kh}\right] \end{aligned}\right\} \qquad (8)$$

where h_b denotes the water depth at the location at a distance $5H_{1/3}$ seaward of the breakwater.

The coefficient α_1 takes the minimum value 0.6 for deepwater waves and the maximum value 1.1 for waves in very shallow water. It represents the effect of wave period on wave pressure intensities. The coefficient α_2 is introduced to express an increase of wave pressure intensities by the presence of rubble mound foundation. Both coefficients α_1 and α_2 have empirically been formulated, based on the data of laboratory experiments on wave pressures. The coefficient α_3 is derived by the relation of linear pressure distribution. The above pressure intensities are assumed to remain the same even if wave overtopping takes place.

The effect of the incident wave angle on wave pressures is incorporated in η^* and p_1 with the factor of 0.5 (1+ cos β) and a modification to the term of α_2 with the factor of $\cos^2 \beta$.

(3) *Buoyancy and uplift pressure*
The upright section is subject to the buoyancy corresponding to its displacement volume in still water below the design water level. The uplift pressure acts at the bottom of the upright section, and its distribution is assumed to have a triangular distribution with the toe pressure p_u given by Eq. 9.

$$p_u = 0.5\ (1 + \cos\beta)\ \alpha_1\ \alpha 3\ w_o\ H_{max} \tag{9}$$

The toe pressure p_u is set smaller than the wave pressure p_3 at the lowest point of the front wall. This artifice has been introduce to improve the accuracy of the prediction of breakwater stability, because the verification with the data of prototype breakwater performance indicated some over-estimation of wave force if p_u were taken the same with p_3.

When the crest elevation of breakwater h_c is lower η^*, waves are regarded to overtop the breakwater. Both the buoyancy and the uplift pressure, however, are assumed to be unaffected by wave overtopping.

Stability Analysis

The stability of an upright breakwater against wave action is examined for the three modes of failure: i.e., sliding, overturning, and collapse of foundation. For the first two modes, the calculation of safety factor is a common practice of examination. The safety factors against sliding and overturning are defined by the following:

Against sliding: $$S.F. = \mu\ (W - U)\ /\ P \tag{10}$$
Against overturning: $$S.F. = (Wt - M_u)\ /\ M_p \tag{11}$$

The notations in the above equations are defined as follows:

M_p	:	moment of total wave pressure around the heel of upright section
M_u	:	moment of total uplift pressure around the heel of upright section
P	:	total thrust of wave pressure per unit extension of upright section
t	:	horizontal distance between the centre of gravity and the heel of upright section
U	:	total uplift pressure per unit extension of upright section
W	:	weight of upright section per unit extension in still water
μ	:	coefficient of friction between the upright section and the rubble mound

The safety factors against sliding and overturning are dictated to be equal to or greater than 1.2 in Japan. The friction coefficient between concrete and rubble stones is usually taken as 0.6. The coefficient seems to have a smaller value in the initial phase of breakwater instalment, but it gradually rises to the value around 0.6 through consolidation of the rubble mound by the oscillations of the upright section under wave actions. The fact that most of breakwater displacements by storm waves occur during the construction period or within a few years after construction supports the above conjecture.

The bearing capacity of the rubble mound and the sea bottom foundation was used to be examined with the bearing pressures at the heel of upright section and at the interface between the rubble mound and the foundation. However, a recent practice in Japan is to make analysis of circular slips passing through the rubble mound and the foundation, by utilizing the simplified Bishop method. For the rubble mound, the apparent cohesion of c = 2 tf/m^2 and the angle of internal friction of ϕ = 35° are recommended.

Example of Wave Pressure Calculation

An example of calculation is given here in order to facilitate the understanding of the breakwater design procedure. The design wave and site conditions are set as in the following:

Wave:	$H_o' = 7.0$ m,	$T_{1/3} = 11$ s,	$\beta = 10°$	
Depth etc.:	$h = 18.$ m,	$d = 10$ m,	$h' = 11.5$ m	$h_c = 4.5$
Bottom slope:	$\tan \Theta = 1/50$			

The incident wave angle is the value after rotation by the amount up to 15°. The geometry of upright breakwater is illustrated in the following figure.

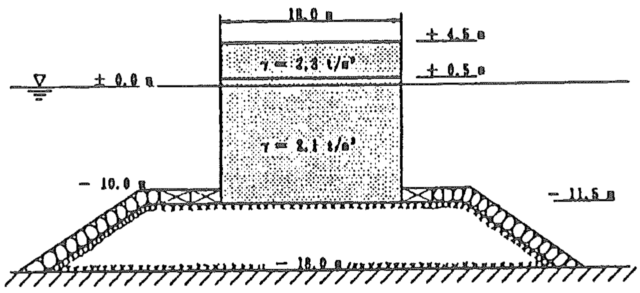


Figure Sketch of upright breakwater for stability analysis

i) ***Design wave height H_{max} and the maximum elevation of wave pressure η^****

$L_o = 188.8$ m	$H_o'/L_o = 0.0371$,	$h/L_o = 0.0953$,	$K_s = 0.94$
$\beta_o = 0.1036$,	$\beta_1 = 0.566$,	$\beta_{max} = \max\{0.92, 0.84\}$	$= 0.92$
$\beta_o^* = 0.1924$,	$\beta_1^* = 0.680$,	$\beta_{max} = \max\{1.65, 1.39\}$	$= 1.65$

Then the wave heights and the maximum elevation are obtained as

$$H_{1/3} = \min\{ 10.91, 6.44, 6.58 \} = 6.44 \text{ m}$$
$$h_b = 18.0 + 5 \text{ x } 6.44/50 = 18.64 \text{ m}$$
$$H_{max} = \min\{ 14.02, 11.55, 11.84 \} = 11.55 \text{ m}$$
$$\eta^* = 0.75 \text{ x } (1 + \cos 10°) \text{ x } 11.55 = 17.19 \text{ m}$$

ii) ***Pressure components***

The wavelength at the depth 18 m is L = 131.5 m. The coefficients for wave pressure are evaluated as

$$kh = 2\pi \times 18 / 131.5 = 0.860$$
$$\alpha_1 = 0.6 + 0.5 \times [2 \times 0.860/\sinh (2 \times 0.860)]^2 = 0.802$$
$$\alpha_2 = \min\{ [(18.64 - 10.0) / (3 \times 18.64)] \times (11.55/10)^2, 2 \times 10 / 11.55 \}$$
$$= \min\{ 0.206, 1.732 \} = 0.206$$
$$\alpha_3 = 1.11.5 / 18.0 \times [1 - 1/ \cosh (0.860)] = 0.820$$

Then, the intensities of wave pressure and uplift pressure are calculated as

$$p_1 = 0.5 \times (1 + 0.9848) \times [0.802 + 0.206 \times (0.9848)^2]$$
$$= \times 1.03 \times 11.55 = 11.83 \text{ tf/m}^2$$

$$p_2 = 11.83 / \cosh(0.860) = 8.49 \text{ tf/m}^2$$
$$p_3 = 0.820 \times 11.83 = 9.70 \text{ tf/m}^2$$
$$p_1 = 11.83 \times (1 - 4.5 / 17.19) = 8.73 \text{ tf/m}^2$$
$$p_u = 0.5 \times (1 + 0.9848) \times 0.802 \times 0.820 \times 1.03 \times 11.55$$
$$= 7.76 \text{ tf/m}^2$$

The symbol p_l denotes the pressure intensity at the top of upright section.

iii) ***Total pressure and uplift, and their moments***

$$P = 0.5 \times (11.83 + 9.70) \times 11.5 + 0.5 \times (11.83 + 7.76) \times 4.5$$
$$= 167.9 \text{ tf/m}$$
$$M_p = 1366.2 \text{ tf-m/m}^2$$
$$U = 0.5 \times 18.0 \times 7.76 = 69.8 \text{ tf/m}$$
$$M_U = (2/3) \times 69.8 \times 18 = 837. \text{ tf - m/m}$$

iv) ***Stability of upright section is assumed as in the following:***

The portion above the elevation + 0.5 m: $\gamma_c = 2.3$ tf/m^3
The portion below the elevation + 0.5 m: $\gamma_c' = 2.1$ tf/m^3

The difference in the specific weight reflects a current practice of sand filling in the cells of concrete caisson. The weight of upright section is calculated for the dry and in situ conditions, respectively, as

$$W_a = 2.1 \times (11.5 + 0.5) \times 18.0 + 2.3 \times (4.5 - 0.5) \times 18.0$$
$$= 619.2 \text{ tf/m}$$
$$W = 619.2 - 1.03 \times 11.5 \times 18.0 = 406.0 \text{ tf/m}$$

The safety factors against sliding and overturning of the upright section are calculated as in the following:

Against sliding:	S.F. = 0.6 × (406.0 - 69.8) / 167.9	= 1.20
Against overturning:	S.F. = (406 × 9.0 - 837.6) / 1366.2	= 2.06

Therefore, the upright breakwater with the uniform width of B = 18.0 m sketched in the figure is considered stable against the design wave $H_o' = 7.0$ m and $T_{1/3} = 11.0$ s

9.9 References

ANNEX

A MATERIALS

It is impossible, within the framework of these lecture-notes, to describe all properties of all materials that can be used in hydraulic engineering. In this appendix some reprints of relevant texts from the reference list are presented for the most important materials:

STONE:
Dimensions, weights, gradations. Source: CUR/CIRIA, 1991

CONCRETE BLOCKS:
Examples of shape. Source: Pilarczyk, 1990

ASPHALT:
Types and use of mixtures. Source: RWS, 1985

GEOTEXTILES:
Types and properties. Source: CUR/CIRIA, 1991
Other references:

COMPOSITE MATERIALS:
Principle and examples. Source: CUR/CIRIA, 1991

CLAY AND VEGETATION:
Gradations, properties and fitness of soil-types for vegetation. Source: CIRIA, 1990

Source: Manual on the use of rock in coastal and shoreline engineering, CUR/CIRIA,1991

3.2.2.3 Block weight and size

The relationships between size and weight of individual blocks may be defined in terms of the equivalent-volume cube (side D_n) or the equivalent volume sphere (diameter D_s) which, with weight density γ_a and block weight W, give the following relationships:

$$D_n = 1.0(W/\gamma_a)^{1/3}$$

$$D_s = 1.24(W/\gamma_a)^{1/3}$$

with $D_n = 0.806\, D_s$

As indicated in the previous section, where graded rock materials are used, size and weight relations refer to medians or averages.

Where the particles are small enough (less than 200 mm) the statistical values are most conveniently derived from sieve analyses. The median sieve size (square openings) (D_{50}) on the percentage passing cumulative curve can be related to the median weight (W_{50}) on the percentage lighter by weight cumulative curve by a simple conversion factor and γ_a or ρ_a. (This dimension D_{50} is the same as the median value of the gross shape dimension z.)

The 50% passing nominal diameter D_{n50} is the size of the cube with equivalent volume to the block with median weight, and is given by

$$D_{n50} = (W_{50}/\gamma_a)^{1/3}$$

The conversion factors relating D_{50} to D_{n50} or W_{50} have been determined experimentally by various workers. The most extensive study is that by Laan (1981), for which the following summary value was obtained and recommended for future use:

$$D_{n50}/D_{50} = 0.84 \quad \text{or} \quad F_s = W_{50}/(\gamma_a D_{50}^3) = 0.60$$

where F_s is the shape factor. As a cautionary note, Laan's study used several different rock types and sizes of stones and found that the value of F_s varied from 0.34 to 0.72, while in the study using different shape classes of limestone fragments (Latham *et al.*, 1988), F_s for all five classes fell between 0.66 and 0.70.

Samples containing blocks larger than 100 mm are difficult to analyse by sieving techniques and direct measurement becomes a more appropriate means of determining size distribution. The relationship between size and weight distributions have been noted above. With large block sizes (i.e. larger than sieve sizes of 250 mm) measurement of weight may prove more practicable than measurement of dimensions, and the size-weight conversion factor such as F_s can be used to determine the median sieve size and other geometric design parameters that may be expressed in terms of sieve sizes.

W_{50} (or M_{50}) is the most important structural parameter in the rock armour design being related to the design parameter D_{n50}, as described above. The controls on block weights were discussed under 'Discontinuities' (Section 3.2.1.4) and the changes in W_{50} from quarry to site were discussed under 'Block integrity' in Section 3.2.2.1. Gradings of rock fulfilling the class limit specification described in the following section may be expected to have standard deviations in D_{n50}, varying from 1% for heavy gradings to 7% for wide light gradings.

3.2.2.4 Grading

In a sample of natural quarry blocks there will be a range of block weights, and in this sense, all rock materials is, to some extent, graded. The particle weight distribution is most conveniently presented in a percentage lighter by weight cumulative curve, where W_{50} expresses the block weight for which 50% of the total sample weight is of lighter blocks (i.e. the median weight) and W_{85} and W_{15} are similarly defined. The overall steepness of the curve indicates the grading width, and a popular quantitative indication of grading width is the W_{85}/W_{15} ratio or its cube root, which is equivalent to the D_{85}/D_{15} ratio determined from the cumulative curve of the equivalent cube or sieve diameters of the sample. The following ranges are recommended for describing the grading widths:

	D_{85}/D_{15} or $(W_{85}/W_{15})^{1/3}$	W_{85}/W_{15}
Narrow or 'single-sized' gradation	Less than 1.5	1.7–2.7
Wide gradation	1.5–2.5	2.7–16.0
Very wide or 'quarry run' gradation	2.5–5.0+	16.0–125+

The term 'rip-rap' usually applies to armouring stones of wide gradation which are generally bulk placed and used in revetments. The phrase 'well graded' should generally be avoided when describing grading width. It merely implies that there are no significant 'gaps' in material sizes over the total width of the grading.

Standardisation of gradings

There are many advantages in introducing standard grading classes. These mostly concern the economics of production, selection, stockpiling and quality control from the producer's viewpoint. With only a few specified grading classes, the producer is encouraged to produce and stock the graded products, knowing that designers are more likely to provide them the market by referring to these standards wherever possible. The proposed standard gradings for armour are relatively narrow. This can result in increased selection costs, but these will often be completely offset by the possibility of using thinner layers to achieve the same design function. Standard gradings are not needed for temporary dedicated quarries supplying single projects where maximised utilisation of the blasted rock is required.

It is convenient to divide graded rock into:

'Heavy gradings' for larger sizes appropriate to armour layers and which are normally handled individually;
'Light gradings' appropriate to sheltered cover applications, underlayers and filterlayers that are produced in bulk, usually by crusher opening and grid bar separation adjustments;
'Fine gradings' that are of such a size that all pieces can be processed by production screens with square openings (i.e. less than 200 mm). For fine gradings, the sieve size ratio from the cumulative sieve curve ($D_{60}/D_{10} = U_c$, the uniformity coefficient) is often used to characterise the width of grading in a sample.

Standard gradings are more or less essential for fine and light gradings. However, for heavy gradings it is not difficult because of individual handling to define and produce gradings other than standard (see below). For example, if the 1–3 t grading is (just) too small for a particular application, choice of the first safe standard grading of 3–6 t will lead to an excessive layer thickness and weight of stone, and here use of a non-standard grading may well be appropriate. Again, ceiling sizes of stones in quarries arising from geological constraints may dictate an upper limit.

Box 24 Explanation of class limit system of standard gradings

Rather than using envelopes drawn on a cumulative plot to define the limits of a standard grading, the proposed system refers to either:

1. A series of weights of stones and their corresponding ranges of the cumulative percentage by weight lighter than values which are acceptable; or
2. A series of 'sieve' size of stones and their corresponding ranges of cumulative percentage by weight passing values, together with the average weight of stones in the grading.

Each weight-standardised grading class is designated by referring to the weights of its lower class limit (LCL) and its upper class limit (UCL). In order to further define the grading requirements realistically and to ensure that there are not too many undersized or oversized blocks in a given grading class, it is necessary to set two further limits, the extreme lower class limit (ELCL) and the extreme upper class limit (EUCL). The standard grading scheme then uses percentage by weight lighter than values, y (equivalent to percentage by weight passing for aggregated), 0, 2, 10, 70, 97 and 100 to set the maximum and minimum percentiles corresponding with each of the four weight values given by ELCL, LCL, UCL, and EUCL (see below and Figure 51, Table 19). Note that although straight-line segments have been drawn in Figure 51 to indicate an envelope, grading curves can go outside these straight-section envelopes and still fulfil the requirements given by the four class limits. However, the further requirement specifying that the effective mean weight, W_{em}, should fall within a set range will help to reassure the designer that there is an appropriate range of W_{50} values.

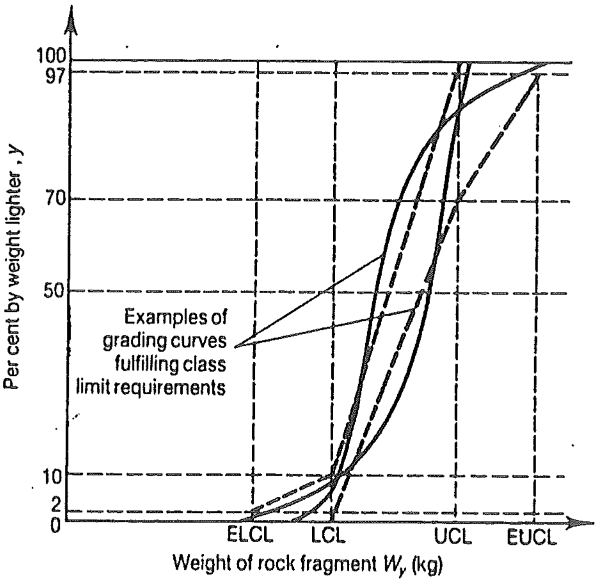
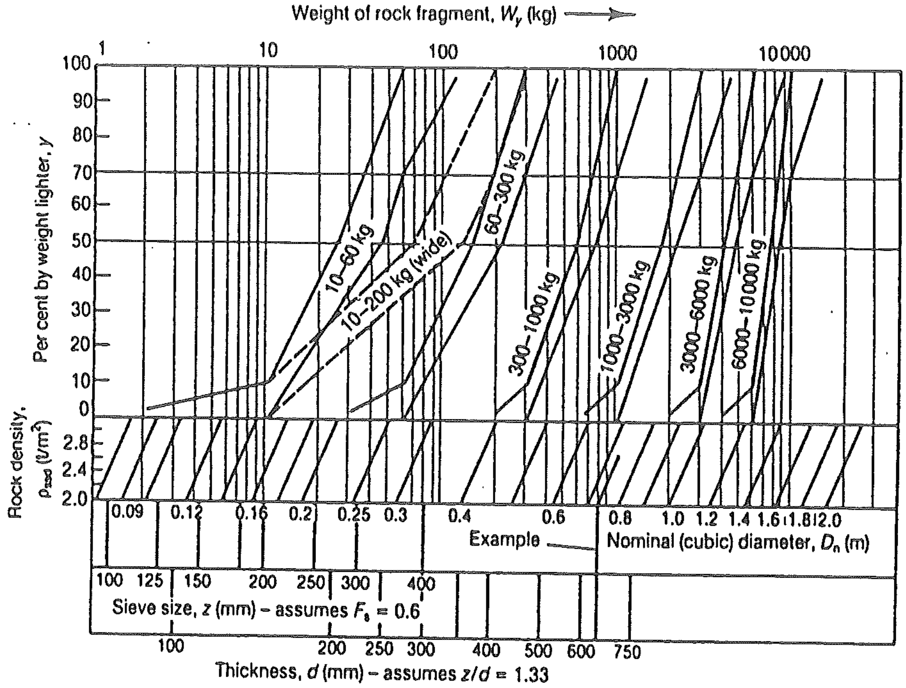


Figure 55 *Explanation of the grading class limits of a standard grading*

Size and average weight standardised gradings

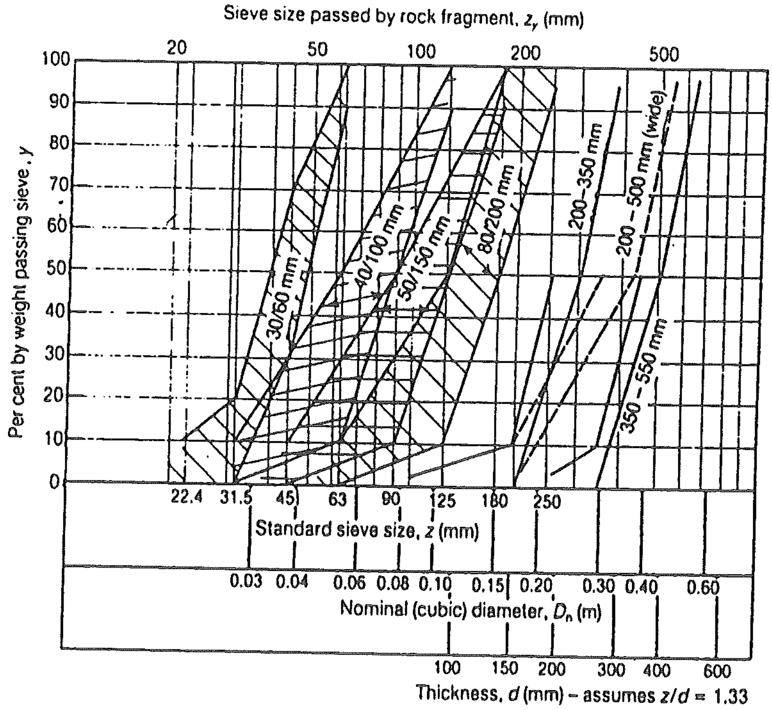
Each size-standarized grading class is defined with reference to the 'sieve' sizes of its ELCL, LCL and EUCL as explained above (no UCL specified) together with the average weight, $\bar{W}_L$, of all rocks not passing the LCL hole. However, for consistency they are designated i.e. named using the LCL and UCL hole sizes. Apart from bulk weighing to determine $\bar{W}_L$, the test verification of size is limited to gauging of blocks using three square guage holes of sizes as defined in Table 19 and imposing the requirment consistent with the weight-grading scheme that:

1. Less than 2% by weight shall pass the ELCL hole, known as the EL hole;
2. Less than 10% by weight shall pass the LCL hole, known as the L hole;
3. At least 97% by weight shall pass the EUCL hole, known as the EU hole.



Note: W_{50} values given for grading envelopes reflect estimates of W_{em} range and are not obligatory (cf. Fig 55)

Figure 56 *Weight gradings and size relationships for the standard light and heavy grading classes*



Note: z_{50} values given for envelopes of light gradings reflect estimates of W_{em} range and are not obligatory (cf. Fig 55)

Figure 57 *Size gradings and relationships for the standard fine and light grading classes*

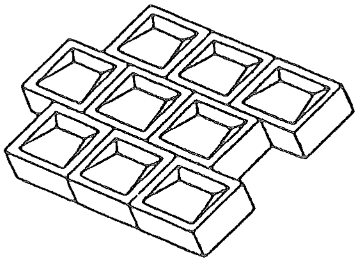
The weight envelopes for the standard light and heavy gradings are shown in Figure 56 with appropriate size-weight conversion. In this figure, rocks of any density and weight can be related to the correponding nominal diameter, D_n, sieve size, z, and minimum rock thickness, d, using the alternative horizontal scales on this chart. The 'sieve' size, z, enveloped for the standard fine and light gradings are shown in Figure 57, again with conversions to D_n and d.
The standard fine and light gradings are produced by screens and grids and sometimes with eye selection to remove oversize material in the 60–300 kg and 10–200 kg classes. The poor screening efficiency that occurs in practice means that a correction factor would be needed in addition to the given theoretical relationships for sieve size, z, and minimum stone thicknesses, d, should a producer wish to use Figures 56 and 57 to indicate the combination of screens and grids that would yield the standard gradings. For the 10–200 kg more widely graded class, experience in the UK quarries using simple grizzlies to remove undersize material and eye selection to remove oversize material suggests that for rock of average (2.7 t/m^3) density, setting the grizzly clear spacing at 225 mm and using eye selection of material above 450 mm will be a good starting point. This advice must be tempered, however, by individual quarry managers' own experience at their particular quarry.

The grading envelopes become progressively narrower in the 'heavy grading' classes, consistent with design requirements and the geological constraints on producing large sizes of blocks. However, projects requiring blocks larger than 10 t should not make the (non-standard) grading class excessively narrow because of the producer's extreme difficulty in selecting accurately and the wastage from oversize block production. For example at $\rho_r = 2.7$ t/m^3, D_n for a 15 t block = 1.7 m, and D_n for a 20 t block = 1.95 m, a difference of about 10%, which is very difficult to select precisely except by individual weighing.

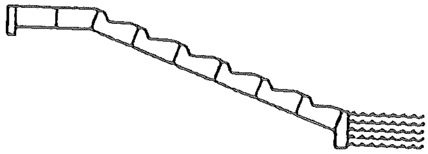
Box 25 Median and effective mean weight

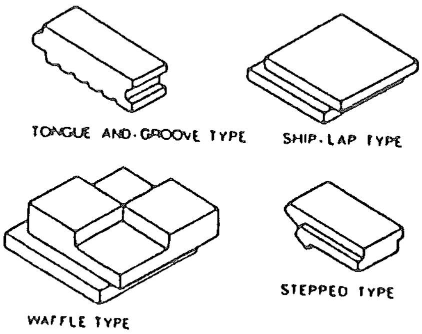
A number of parameters in addition to the designated class limits may be required for design and specification purpose. An acceptance range for the important design parameter, W_{50} (median weight), is also useful for each grading class. However, in some circumstances an effective mean weight W_{em} for a particular consignment may be more easily obtained simply by bulk weighing and counting. In order to avoid including fragments and splinters in the distribution, W_{em} is defined as the arithmetic average weight of all blocks in the consignment or sample, excluding those which fall below the ELCL weight for the grading class. An empirical conversion factor relating W_{em} to W_{50} allows an estimate of W_{50} without the necessity of weighing each piece to obtain a weight-distribution curve. As the grading becomes wider, so will W_{50} depart more from W_{em}. Approximate relations are as follows:

10–60 kg	$W_{50}/W_{em} = 1.3$
60–300 kg	$W_{50}/W_{em} = 1.15$
300–1000 kg	$W_{50}/W_{em} = 1.10$
1000–3000 kg	$W_{50}/W_{em} = 1.05$
3000–6000 kg and	
6000–10 000 kg	$W_{50}/W_{em} = 1.00$

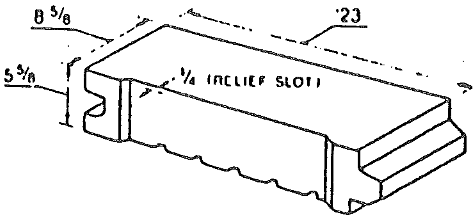


A. FREE RECTANGULAR-BLOCKS

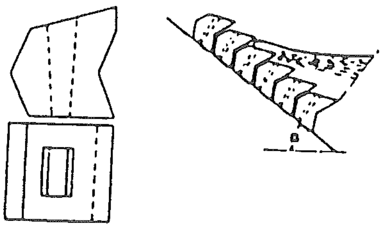




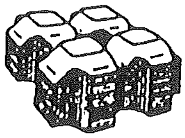
B. INTERLOCKING-BLOCKS



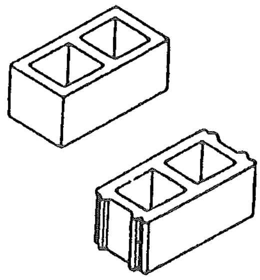
MODIFIED TONGUE AND GROOVE BLOCK



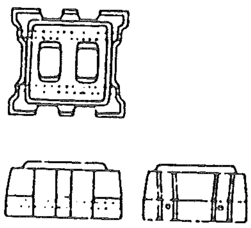
C. SVEE-BLOCKS



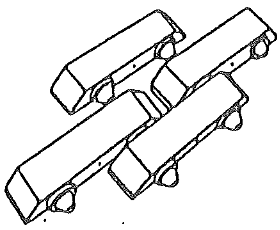
D GOBI-BLOCK



E. BUILDING-BLOCKS



F. ARMORFLEX-BLOCK



G. TERRAFIX-BLOCKS

Source: The use of asphalt in hydraulic engineering, RWS, 1985

1.1 Mix components

Asphalt is a mix of various components:

- bitumen;
- mineral aggregate;
- additives, if required.

The mineral aggregate is composed of crushed stone, gravel, sand or filler or a combination.
The choice of the most suitable composition for a particular application depends mainly on the requirements which the material has to meet and the associated mix properties, see Section 6.1.
The mix properties are specified by the composition, that is, the relative proportions of the various components, the properties of the components themselves, and the properties which result from the application and compaction method.

1.2 The degree of filling of the mix

The mineral aggregate mix contains voids. Initially, the bitumen coats and binds the various aggregate components together. If more bitumen is applied than is necessary for coating and binding then the pores will gradually be filled.
Mixes, in which the bitumen only serves as a binder, are referred to as 'underfilled' mixes, see Figure 1.1a. The properties of such a mix are directly related to the properties of the stone skeleton (4). If the proportion of bitumen is increased the voids in the mineral become filled and the influence of the bitumen on the properties of the mix is increased while that of the stone skeleton is reduced. With mixes in which the pores are almost filled with bitumen, see Figure 1.1b, both the stone skeleton and the bitumen contribute to the mix properties. This type of mix must be compacted, either mechanically or under its own weight.
'Overfilled' mixtures are those in which the volume of bitumen is greater than that of the voids in the mineral aggregate. In such a mix the properties of the bitumen predominate, the mineral providing only a certain amount of stiffening, see Figure 1.1c. This type of mix is impermeable and requires no compaction.

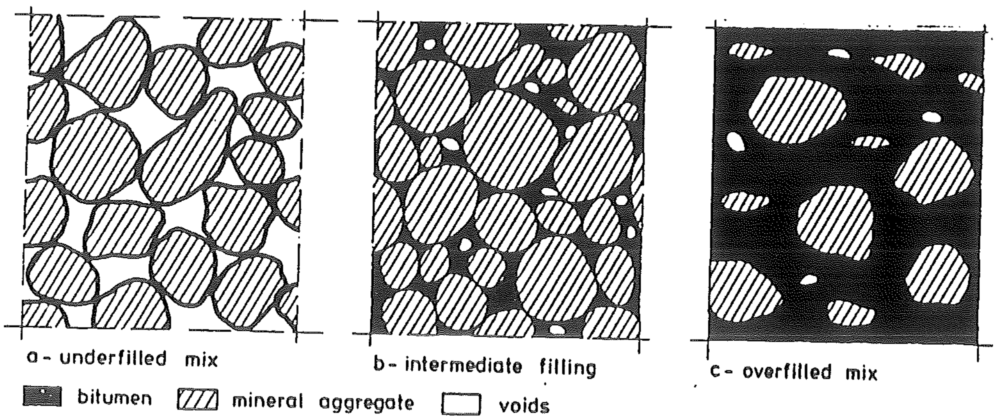


Figure 1.1 The degree of bitumen filling in the mineral aggregate.

Some material properties:

Material Parameter	Asphalt Mastic	Asphalt Concrete	Colloidal Concrete
Stiffness,S (Elasticity) (N/m^2)	1.10^9	7.10^9	2.10^{10}
Strength, σ_b (N/m^2) Correction for fatigue	8.10^6 0.1 - 0.3	8.10^6 0.15 - 0.4	1.10^6 0.5 - 0.6
Density, ρ_m (kg/m^3)	2000	2400	2400
Poisson constant, ν (-)	0.33	0.33	0.2

1.3 Voids in asphalt mixtures

The term 'voids' refers to the volume of pores in the compacted asphalt (5). The voids ratio, VIM (voids in mix), is given by:

$$\text{VIM} = 100\left(\frac{d_m - d_a}{d_m}\right) \text{ vol } \%$$

d_m = density of the mix without voids (kg/m³)
d_m = density of the mix with voids (kg/m³)

In general, the smaller the voids ratio (HR) the more resistant is the mix to erosion and the greater its durability. A mix with a small voids ratio is better 'sealed' against external influences such as oxygen, light and water. If water penetrates between the bitumen and the mineral (through the material) there is a loss of adhesion which is referred to as 'stripping'.
Exposure to the atmosphere and light ages the bitumen. In this respect, the size of individual pores and degree of interconnection between voids are also important.
The voids ratio and the distribution of voids also determine whether or not the mix is sand and watertight. Although water impermeability is not always a functional requirement it gives a good indication of the durability of a mix.

To illustrate:
- A mix containing sand with 5%, by mass, of bitumen and a voids ratio of 25% is sand-tight. A mix, however, of open stone asphalt with 80%, by mass, of stone and 20%, by mass, of mastic with the same voids ratio is not.
- An asphaltic concrete with a voids ratio of 3% can be considered as absolutely watertight. In this case the voids are not interconnected. For the same reasons a mastic with the relatively large voids ratio of 10% is also watertight.

7 Use of asphalt products in hydraulic structures

The most important mix types applied in hydraulic structures are:
1. Asphaltic concrete
2. Asphalt mastic
3. Grouting mortars
4. Dense stone asphalt
5. Open stone asphalt
6. Lean sand asphalt
7. Asphalt membranes

Mixes are defined by the constituents, the nature of asphalt mix and the binder material.

7.1 Asphaltic concrete

Asphaltic concrete is probably the best known mix type. It is a mixture of crushed stones or gravel, sand and filler in which the pores are practically completely filled with bitumen. The voids ratio is 3 to 6%.
In general the material must be compacted and is unsuitable for application under water or in the tidal zone. In view of the small voids ratio required, see Section 9.2.1., asphaltic concrete can be considered to be impermeable.
Asphaltic concrete is applied as a watertight dike revetment above the mean high water level, and as a lining for canals, reservoirs etc.

7.2 Mastic

Mastic is a mixture of sand, filler and bitumen. There is more bitumen available than necessary for filling the voids in the sand filler mixture. The mix, therefore, is naturally dense and need not be compacted. Mastic can be poured at working temperatures and is used, for asphalt slabs above and under water for lining or as bed and toe protection. When cold, mastic forms a viscous quasi-static mass.

7.3 Grouting mortars

Grouting mortars are hot-type mixes of sand, filler and bitumen of which there is more than required to fill the voids in the mineral; stone and gravel can be added if necessary. These mortars are used for grouing stone revetments above and below water-level, and also for slab construction.

7.4 Dense stone asphalt

Dense stone asphalt is a gap-graded mixture of stone, sand, filler and bitumen. The amount of bitumen slightly overfills the mixture. The material is, therefore, water impermeable.
It is used as bottom and slope protection and also in toe construction.

7.5 Open stone asphalt

Open stone asphalt is a gap-graded mixture of mastic and stone – a stone frequently used is limestone 20/40 mm. Mixing is carried out in two stages. First mastic is prepared and secondly it is mixed with limestone. The mastic binder only coats and connects the limestone particles together.
It is an 'underfilled' mix and, because of its open structure, should not be placed under water except in the form of prefabricated mattresses.

7.6 Lean sand asphalt

Lean sand asphalt is a mixture of sand, often locally obtained, with 3 to 5% bitumen. It is a greatly 'underfilled' mix and the function of the bitumen is simply to coat the sand grains and bind them together. After some time the permeability is very similar to the sand from which it is made.
It is used as a core material for reclamation bunds, filter layers and as permanent or temporary cover layer above and below water-level.

7.7 Membranes

Membranes are thin impermeable watertight layers of bitumen which are prepared in-situ or prefabricated. Membranes are used as impermeable linings for canals, banks and water courses.

A.4 GEOTEXTILES

Source: Manual on the use of rock in coastal and shoreline engineering, CUR/CIRIA,1991

Geotextiles are permeable textiles made from articifial fibres used in conjunction with soil or rocks as an integral part of a man-made project and were first used in the Netherlands. They are frequently employed as filter membranes and as the interface between differently graded layers. Geotextiles are also used as bottom protection and can be loaded with concrete blocks (block mattress), bituminous bound crushed stone and sand (fixtone mattress) and geotextile tubes filled with gravel (gravel-sausage mattress). Gravel bags have also been used for special filter requirements.

The basic functions of geotextiles may be listed as follows:

1. *Separation*: the geotextile separates layers of different grain size;
2. *Filtration*: the geotextile retains the soil particles while allowing water to pass through;
3. *Reinforcement*: the geotextile increases the stability of the soil body;
4. *Fluid transmission*: the geotextile functions as a drain because it has a higher water-transporting capacity than the surrounding materials.

3.5.4.1 Geotextile manufacture

Geotextiles are manufactured from a variety of artificial polymers:

1. Polyamide (PA);
2. Polyester (PETP);
3. Polyethylene (low-density LDPE and high-density HDPE);
4. Polypropylene (PP);
5. Polyvinylchloride (PVC);
6. Chlorinated polyethylene (CPE).

The first four are the most widely used although many variations are possible. Additives are also employed in geotextile manufacture to minimise ageing, to introduce colour and as anti-oxidants and UV stabilisers.

Comparisons of properties of the four main polymer families are shown in Figure 90. These are very broad, because there are many variants within each group. Some properties (such as strength) are also greatly influenced by the different processes of manufacture. A classification of geotextiles based on the type of production and the form of the basic elements is given in Figure 91.

The basic elements used in geotextiles are monofilaments, multifilaments, tapes, weaving film and stable fibres. Monofilaments are single, thick, generally circular cross-sectioned threads with a diameter ranging from 0.1 mm up to a few millimetres. Multifilaments (yarns) are composed of a bundle of very thin threads. Yarns are also obtained from strips and from wide films. Tapes are flat, very long plastic strips between 1 and 15 mm wide with a thickness of 20–80 μm. A weaving film is sometimes used for the warp 'threads' in a fabric.

The basic fibre is a fibre of length and fineness suitable for conversion into yarns or non-woven geotextiles. For non-woven fabrics the length is usually about 60 mm.

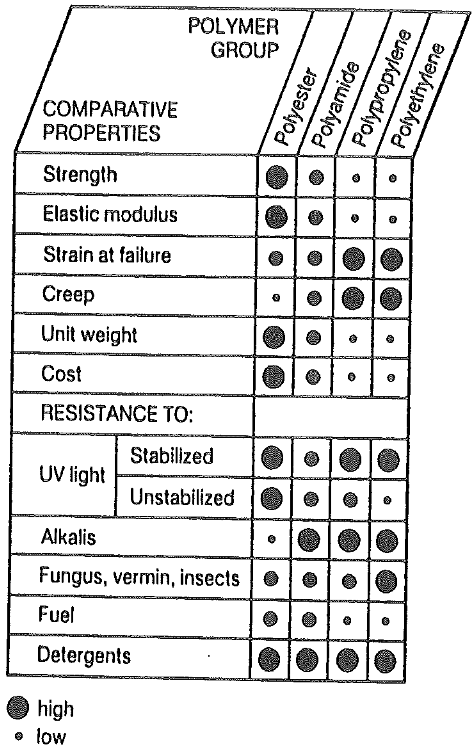


Figure 90 *Comparative properties of general polymer families*

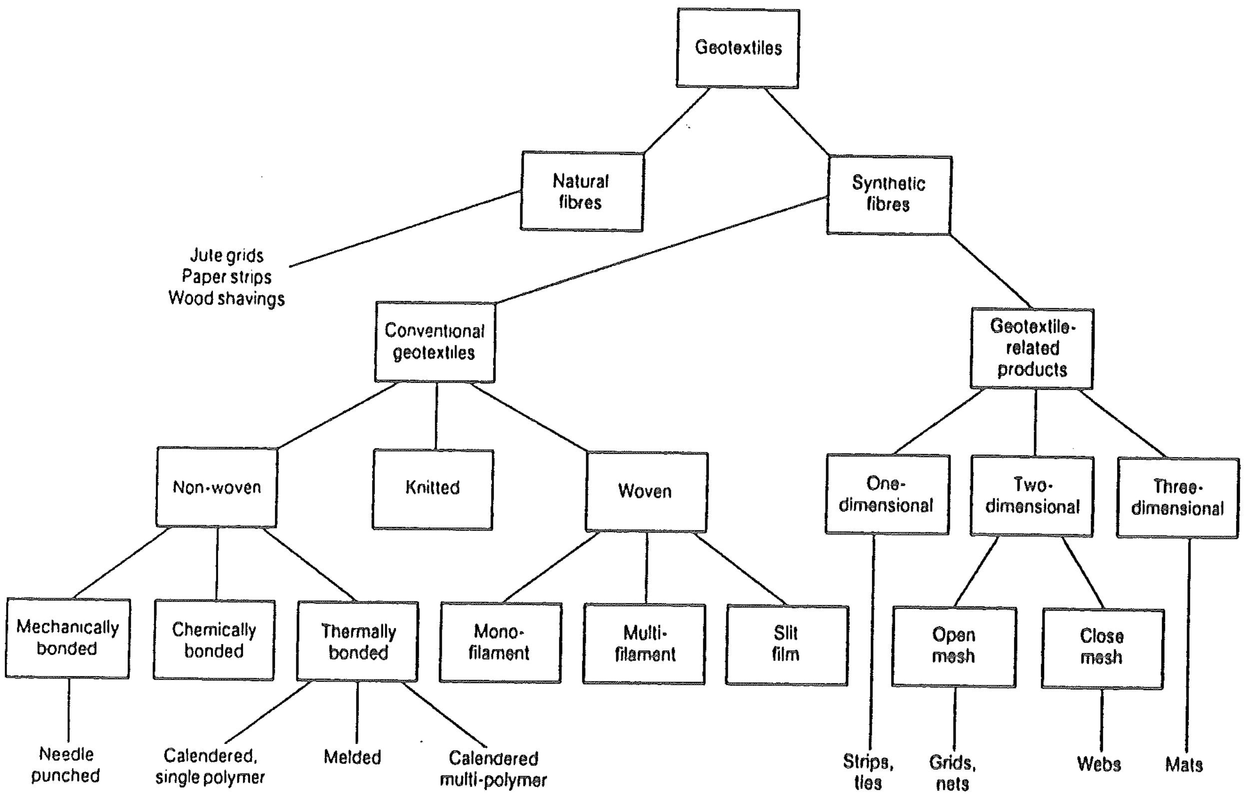


Figure 91 *Geotextile classification groups*

Woven geotextiles

A woven fabric is a flat structure of at least two sets of threads. The sets are woven together, one referred to as the warp, running in a lengthwise direction, and the other, the weft, running across. Woven geotextiles can be categorized by the type of thread from which the fabric is manufactured.

Monofilament fabrics are used for gauzes of meshes which offer relatively small resistance to through-flow. The mesh size must obviously be adapted to the grain size of the material to be retained. Monofilament fabrics are made principally from HDPE and PP.

Tape fabrics are made from very long strips of usually stretched HDPE or PP film, which are laid untwisted and flat in the fabric. They are laid closely together, and as a result there are only limited openings in the fabric (Figure 92).

Split-film fabrics are made from mostly fibrillated yarns of PP or HDPE (Figure 93). The size of the openings in the fabric depends on the thickness and form of the cross-section of the yarns and on the fabric construction. These split-film fabrics are generally heavy. Tape and split-film fabrics are often called slit-films.

Multifilament fabrics are often described as cloth, because they tend to have a textile appearance and are twisted or untwisted multifilament yarns. The fabrics are usually made from PA 6, PA 6.6 or from PETP.

Besides the above-mentioned monofilament fabrics, special *mesh-type constructions* are produced such as those with a monofilament warp and a multifilament weft which have outstanding water-permeability and sand-retention properties. Other examples include open meshes in which the woven or unwoven warp and weft threads are attached at crossing points by chemical or thermal bonding and other meshes constructed by using knitting techniques.

Non-woven geotextiles

A non-woven geotextile is a textile structure produced by bonding or interlocking of staple fibres, either monofilaments or multifilaments arranged at random, accomplished by mechanical, chemical, thermal or solvent means. Non-woven gauzes are structures with large meshes which are formed by placing threads or tapes at predetermined distances on top of one another and bonding them at the intersections by a chemical, thermal or mechanical process.

Geotextile-related products

These products are distinguished in one-dimensional (strips, ties), two-dimensional (gids, nets, webs) and three-dimensional (mats) products. Grids are lattices made from perforated and then stretched polymer sheets. Three-dimensional mats are produced by extruding monofilaments into a rotating profile roller, followed by coating so that the threads adhere to each other at crossings which are spatially arranged. The matting material itself occupies less then 10% of the mat volume. The mats are 5–25 mm thick and about 1–6 m wide.

3.5.4.2 *Characteristics and properties*

The geotextile properties stem primarily from their functional requirements. Since the geotextile can have a variety of functions, requirements are diverse. For reinforcement the emphasis is on mechanical properties such as *E*-modulus and strength, for filters it is on properties such as water permeability and soil tightness. The durability required will depend on the specific application and lifetime required. Geotextiles must also fulfil secondary functional requirements related to the execution of the work (e.g. a certain amount of UV resistance is needed) or it must have resistance to mechanical wear and tear if construction equipment is to be driven over the fabric. The suitability of a geotextile should be checked against these functional requirements during the design phase of the project.

Although specification requirement tests need only be carried out once, the following quality control tests may be required during production: tearing strength, grab test strength, tensile strength, strain at breaking load, moduli and mass distribution. These tests should be made in both the length and width directions. The thickness, the mass per unit area and the bursting strength may also need to be checked, and in some applications water permeability and sand-retaining properties will be important. A large number of national and a few international standard test methods are available covering these requirements.

Dimensions

The maximum standard width available for both woven and non-woven fabrics is 5–5.5 m. The length is limited by the available transport facilities and ease of handling on-site. Depending on the mass per unit area, the length generally lies in the range 50–200 m.

Jointing is necessary to obtain greater dimensions. In practice, large areas can be covered by overlapping sheets. Where physical continuity is required without overlap then heat welding (some non-wovens) or stiching may be used. The seam forms the weakest link in the geotextile construction and should therefore be checked thoroughly against the specifications. The thickness of most geotextiles lies between 0.2 and 10 mm when unloaded, although this may sometimes reduce under pressure.

In general, the mass of non-woven geotextiles lies in the range 100–1000 g/m^2, 100–300 g/m^2 being the most commonly used. Woven fabrics can be heavier and masses between 100 and 2000 g/m^2 are possible. The greater demand is for the lighter grades in the range 100–200 g/m^2. Generally, the lighter types of geotextiles are used as separators, the heavier woven fabrics for reinforcement and the heavier non-wovens for fluid transmission.

Chemical properties

One of the characteristic features of synthetic polymers is their relative insensitivity to the action of a great number of chemicals and environmental effects. Nonetheless, each plastic has a number of weaknesses which must be taken into account in the design and application. Specifically, the life of geotextiles can be affected by oxidation and by some types of soil/water/air pollution. Many synthetic polymers are sensitive to oxidation. The end result of oxidation is that mechanical properties such as strength, elasticity and strain absorption capacity deteriorate and the geotextile eventually becomes brittle and cracks.

Source: Manual on the use of rock in coastal and shoreline engineering, CUR/CIRIA,1991

3.5.5 COMPOSITE MATERIALS

Composite systems are normally adopted where improved stability of the rock materials being used is sought (see Section 5.1.3.9) or where ease of placing the protection system is important.

3.5.5.1 Gabions

A gabion is a box or mattress-shaped container made out of hexagonal (or sometimes square) steel wire mesh strengthened by selvedges of heavier wire, and in some cases by mesh diaphragms which divide it into compartments (Figure 94). Assembled gabions are wired together in position and filled with quarried stone or coarse shingle to form a retaining or anti-erosion structure. The wire diameter varies but is typically 2–3 mm. The wire is usually galvanised or PVC-coated. PVC-coated wire should be used for marine applications and for polluted conditions.

The durability of gabions depends on the chemical quality of the water and the presence of waterborne attrition agents. The influence of the pH on the loss of the galvanic zinc protection is small for pH values in the range 6–12 and there are examples of gabions with negligible loss over 15 years. Grouting of the stone-filled gabions or mattresses can give some protection to the wire mesh against abrasion and corrosion, but this depends on the type of grout and the amount used.

The dimensions of gabions vary, but typically range in length from 2 to 4 m (mattress, 6 m), with widths about 1 m (mattress, 2 m), and height 0.3–1.0 m (mattress, max. 0.3 m). The mesh size is typically 50–100 mm.

The units are flexible and conform to changes in the ground surface due to settlement. Prefabricated gabions can be placed under water. Gabions can thus be used in a wide variety of marine works: groynes, dune and cliff protection, protection of pipelines and cables, and as toe protection. Mattress-shaped gabions are flexible and are therefore able to follow bed profiles both initially and after any scouring which may take place. Gabions can also be piled up to the form retaining walls or revetments (see Section 6.2, Box 80). In order to prevent migration of solid through the structure they may be used in conjunction with geotextile filter layers.

In certain applications the gabion structure needs impermeability or weight to counter uplift. To give these characteristics, the stone is grouted with mastic or a cement-bound grouting. The weight of the structure can also be influenced by the choice of the density of the stone blocks with which the gabion or gabion-mattress is filled.

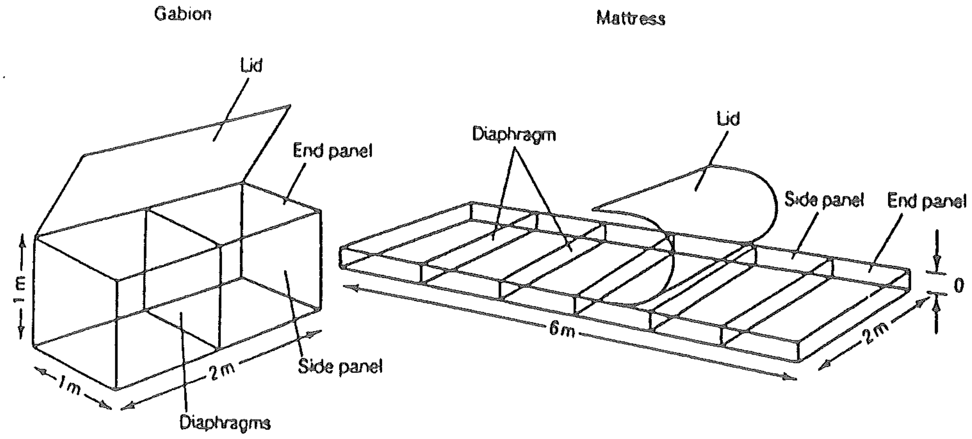


Figure 94

3.5.5.2 *Mattresses*

Prefabricated mats for hydraulic constructions have been used extensively as a medium for combating bottom and embankment erosion. The range of mat types have increased considerably during the last decade. Formerly, use was made of handmade mats of natural materials such as reed, twigs/branches (willow), rope, quarry stone and rubble. Today, geotextiles, bituminous and cement-bonded materials, steel cables and wire gauze are used as well as the more traditional materials.

Functions and principles

A distinction must be made between mattresses used for the protection of continually submerged beds and foreshores and mats employed for slope or front face protection of embankments. Bottom protection mats are concerned with:

- The mitigation of groundwater flow in the underlying seabed to prevent horizontal transport and erosion:
- Prevention of piping of soil particles;
- Assisting in the overall geotechnical stability of a larger structure;
- Offering resistance to loads imposed by anchors, trawler boards, etc. and thereby protecting pipes and cable lines;
- Providing a protection system with the flexibility to allow for anticipated settlements.

Mattresses for embankment retention have similar functions, i.e:

- Prevention of soil transport;
- Resistance to wave action;
- Provision of flexibility to allow for settlement.

Additionally, they may be required to:

Not trap refuse; and
- Support vegetative growth.

Mats are manufactured from the following elements:

1. *The carrier*: Fabric, gauze, cables and bundlings by which the necessary strength is obtained for its manufacture, transport and installation.
2. *The filter*: A geotextile which satisfies the conditions with respect to permeability, soil density, strength, etc. In some embankment-mat constructions the geotextile is applied separately.
3. *The ballast*: Concrete blocks, packaged sand, gravel, stone or asphalt necessary for the stability of the protection. Quarry stone can be used to serve as a material to sink the mat.
4. *The connections*: Pegs, wires, etc. by which the ballast material is fixed to the filter and the carrier materials.

In addition to the mat proper, a further deposit of gravel, aggregates, quarry stone, slag, etc. may be necessary when the ballast material stability in itself is insufficient or when extra protection is demanded.

The carrier and the filter can often be combined in the form of a single geotextile with the necessary mechanical and hydraulic properties. Sometimes the filter is combined with the ballast, which is then dimensioned as a granular filter.

Manufacture and placing

Fascine mats are typically up to 100 m long and 16–20 m wide. Historically, these mats were manufactured from osiers which were clamped between the so-called 'raster' of bundles of twigs/branches bound together to a circumference of 0.3 m.

'Plugs' were driven into the crossing-points of the bundles to serve as anchor points for towing and sinking the mats (for bottom protection) or collar-pieces (for foreshore protection). Because of the destruction of the osiers by pile-worm attack, a reed layer was incorporated into the mats. These fascine mats were made on embankments in tidal areas or on special slipways.

The finished mat or collar-piece was then towed to its destination and sunk using quarry stones which were placed in position by hand, with the uppermost rasterwork of bundles serving to maintain the stone in place during sinking. After sinking, more and heavier stones were discharged onto the mat if this was found to be necessary.

Modern mats are made in accordance with the same principles. The twigs/branches and reed are replaced by a geotextile into which loops are woven and to which the rasterwork of bundles are affixed. A reed mat may be affixed onto the geotextile in order to prevent damage to the cloth during the discharge of stones onto the mat. The geotextile extends out beyond the rasterwork with the aid of lath outriggers so that, on sinking, the mats overlap. Sometimes a double rasterwork is applied to give the mats a greater rigidity and more edge support to the quarry stone load. The mat construction can serve as a protection for the bottom, a foreshore and even as an embankment protector. In the last case the mat is hauled up against the embankment itself.

Modern alternatives to fascine mats include mats where the quarried stone is replaced by open-stone asphalt reinforced with steel mesh, or by 'sausages' of cloth or gauze filled with a ballast material such as sand, gravel, bituminous or cement-bound mixtures.

Source: Use of vegetation in civil engineering, CIRIA,1990

Box 2.3 summarises the important physical parameters of the soil. The soil characteristics which affect vegetation establishment and growth, their principal determinants and the ways in which they can be modified are given in Box 2.4. Soil contains water, air, fine earth, stones and organic matter.

Box 2.3 Soil physical parameters (definitions relating to plant growth)

Parameter	*Definition*	*Assessment*
Soil texture	Description based on proportions by weight of sand, silt and clay as percentages of fine earth fraction <2 mm in size	Field estimations or laboratory measurement
Stoniness, % vol	Proportion of large particles, >2 mm	Direct measurement, or field estimation
Dry bulk density* (ρ_d) Mg/m^3	Apparent density of soil *in situ* (on a dry basis)	Field measurement, either removal of undisturbed core or replacement method (sand or water)
Particle density* (ρ_s) Mg/m^3	Density of the soil particles	Laboratory measurement by displacement. Most soils are consistent with a value about 2.65 Mg/m^3
Void ratio (*e*)	Ratio: volume of soil voids to volume of solids	$e = \rho_s/(\rho_b - \rho_s)$
Porosity (ε) %* (total pore space, *T*)	Volume of soil voids expressed as a percentage of total *in situ* soil volume *note* – voids occupied by air and water.	$\varepsilon = (1 - \rho_b/\rho_s) \times 100$
Soil erodibility factor	The risk of erosion by air or water due to the nature of the soil itself	Direct measurement or estimation based on soil texture, *see* Sections 6.3.2 and 6.5.2
Packing density (L_d)* (rooting potential)	A more reliable indicator of the effects of compaction than bulk density alone: allows for clay content	$L_d = \rho_b + (0.009 \times \%\ \text{clay})$

* see Hall *et al.*, (1977), Jarvis and Mackney (1979).

Box 2.4 Soil physical characteristics

Important soil characteristics	Principal determinants				Modifiers					
	Particle size	Packing density	Porosity	Organic matter	Vegetation cover	Topography	Cultivation	Compaction	Additions	Time
Texture	●									
Soil structure	●		●	●			○	○		○
Rooting potential	●	●	●	●			○	○		
Soil water capacity	●		●	●				○	○	
Permeability and water acceptance			●		○	○	○	○		○
Ion exchange capacity	●			●	○				○	
Erodibility	●		●	●	○	○	○	○		
Ease of cultivation	●	●	●			○		○	○	

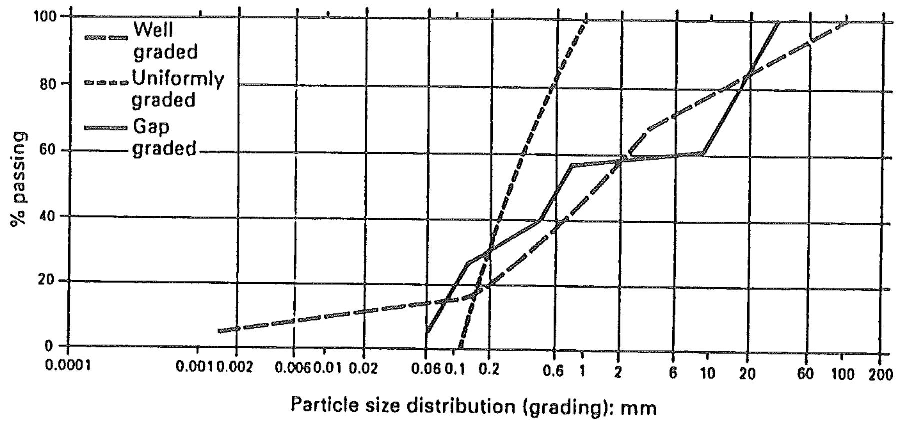
Rooting potential indicates the resistance of the soil to root penetration, which depends mainly on the soil's bulk density and on mechanical strength. Roots have great difficulty penetrating soil with strengths greater than 2.0 to 2.5 MN/m^2, though higher limiting values have been suggested for coarse-textured soils. Generally, root growth is enhanced by greater moisture content and voids, and is retarded by higher bulk density and clay content. Critical dry bulk densities for soils, above which root growth is severely restricted, are about 1.4 Mg/m^3 for clay soils and 1.7 Mg/m^3 for sandy soils. As clay content is so important in determining the rooting potential, a term *packing density* (L_d) is often used to determine the maximum density to which a soil can be compacted and still permit root growth (*see* Box 2.3).

Soil texture describes the particle size distribution and gives an indication of the likely behaviour of a soil in respect of handling, root growth or drainage. Descriptions such as sandy loam or clay are based on measured proportions and mixtures of clay, silt and sand in the fine earth (<2 mm) fraction, as shown in Box 2.5.

Soil structure is a characteristic which describes the arrangement and size of particle aggregates or 'peds'. Structure develops over time, as fine soil particles aggregate into crumbs and blocks. This increases the number of large pore spaces and thus the permeability and rooting potential of the soil. The presence of organic matter and plant roots plays a major role in developing and maintaining soil structure. Structure is easily damaged by handling or cultivation during wet conditions, when the soil is weaker.

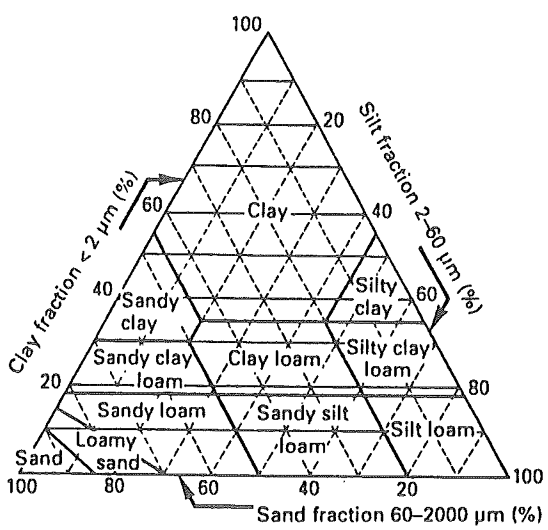
Box 2.5 Soil texture

The usual particle grading curves prepared to BS 5930:1981 are familiar to most engineers. Soils are described according to the British Soil Classification System for Engineering Purposes.



Clay fraction	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	Cobbles
	Silt fraction			Sand fraction			Gravel fraction			
Fine earth							Stones			

For soil survey work, texture descriptions are based on the fine earth fractions, that is <2 mm size. The overall proportion of gravel and cobbles defines the *stoniness*. The proportions of sand, silt and clay in the fine earth soil matrix define the texture classes as given in the triangular diagram.



The proportions of sand, silt and clay can be obtained from the particle grading curve, calculating the quantity of each size fraction as a percentage of the <2 mm fraction.

2.4.5 Soil potential

The physical, water and chemical characteristics of the soil can be combined into an overall assessment of soil potential for plant growth. A scheme for this is given in Box 2.10.

Box 2.10 Assessment of soil potential

Class A is the highest quality and suitable for situations where good quality fertile topsoil is necessary. However, class C, whilst of poorer quality, would still be suitable for many situations (*see* Section 4.2.1). In many cases it would be possible to modify or manage a class B or C soil to improve its quality.

Parameter	*Unit*	*Suitability class*			*Unsuitable*
		A	*B*	*C*	
Soil type					
Texture	description[1] and clay%	fLS,SL SZL,ZL	SCL,CL, ZCL,LS	C<45% SC,ZC,S	C>45%
Stoniness	% vol	<5	5–10	10–15	>15
Available water capacity (at packing density 1.4–1.75)	% vol	>20	15–20	10–15	<10
pH		5.7–7.0	5.2–5.5	4.7–5.5	<4.7
			7.0–7.3	7.3–7.8	>7.8
Conductivity	mmho/cm	<4	4–8	8–10	>10
Pyrite	% weight	–	<0.2	0.2–3.0	>3.0
Soil fertility					
Total nitrogen	% weight	>0.2	0.05–0.2	<0.05	
Total phosphorus	mg/kg	>37	27–37	<27	
Total potash	mg/kg	>360	180–360	<180	
Available phosphorus	mg/kg	>20	14–20	<14	
Available potassium	mg/kg	>185	90–185	<90	

Notes: 1. f = fine, S = sand, C = clay, L = loam, Z = silt

4.2.1 Selection of soil materials

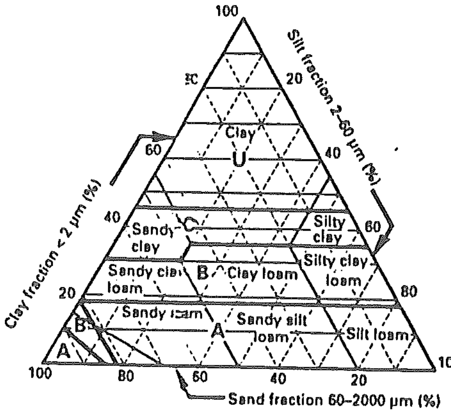
The simple system for assessing soil potential described in Box 2.10 can be used as a general guide to classify all material which the engineer intends to use as soil, regardless of origin, according to its potential for plant growth. Classes are allocated as follows.

Soil type

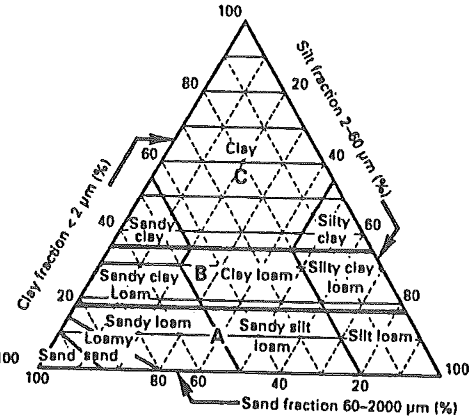
Class A Highest growth potential, important when it is necessary for soil to have minimal restrictions on plant growth. Suitable for final soil covering or topsoil.
Class B Where growth potential is not critical, but reasonable growth is still required. Also suitable for subsoil layers beneath Class A.
Class C Will still support good growth if managed properly, but susceptible to handling problems which may restrict growth. Can also be used as subsoil layers.

Suitability cannot be based on soil texture alone and the classes proposed in Box 2.10 have only general application. For some uses special soil characteristics may be required; examples are given in Box 4.3.

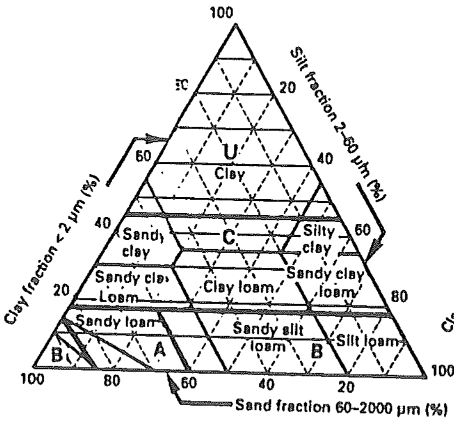
Box 4.3 Soil suitability for specific purposes based on texture



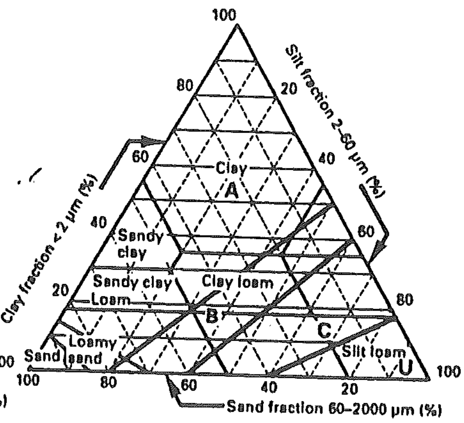
(a) Suitability classes given in Box 2.10



(b) Ease of cultivation and handling



(c) Heavily trampled and trafficked areas



(d) Erosion risk (erodibility)

Notes
A – most suitable/least susceptible
B – moderately suitable
C – just suitable/most susceptible
U – unsuitable

Soil fertility

Class A Highly fertile, will produce dense vigorous growth, requiring higher maintenance and leading more quickly to successional changes. This is not always necessary nor desirable, however, and the group is best used for intensively managed areas and for grazing.

Class B Moderate fertility; fertilisers may be required to support very productive growth.

Class C Minimal fertility, suitable for low-maintenance vegetation but fertilisers will be necessary. Swards should have good legume component.

Soil physical properties and fertility are defined separately so that different classes can be selected for each, depending on the situation.

10 CONSTRUCTION/MAINTENANCE

10.1 Construction Methods

10.1.1 The concept

Copying concepts from the established industrial countries for application in countries with a strongly developing economy, like Indonesia, is generally not the best solution. The main reason is the difference in the available resources. In the established industrial countries there is a strong tendency to solve problems with minimal required labour. Thus, a capital-intensive solution is searched for. The reason is the very costly social system and the high standard of living. This causes a large difference between hourly income and hourly costs of labour, which is much less in countries with a developing economy. On the other hand, these countries have often difficulties in importing industrial products from abroad. It is also difficult to have sufficient financial resources available. For countries like Indonesia it is economic to search for solutions which hardly require any investments, but are relatively labour-intensive. These solution often require more maintenance. However, increased maintenance costs may even be advisable, provided the initial investment is very low. The total cost of the solution, i.e. investment plus maintenance, can be spread over a longer period without financial indebtedness. The above described philosophy is valid in many sectors, especially in the coastal zone. Works in the coastal zone are generally of a large scale, require long planning and should be constructed using capital intensive methods.

10.1.2 Low investment solutions

Sometimes so-called "low-cost solutions" are presented, especially developed for use in countries with a fast developing economy. The argument to use these is that these countries cannot afford high-cost solutions. This reasoning is not correct! Also the industrialized countries are very much interested in cheap solutions. Even in cases when enough money is available, why choose a more expensive solution if a low-cost solution is feasible ?
In practice however, the cost of these "low-cost solutions" are not really low. The total costs for every structure comprise the initial investment and maintenance cost. In fact the total costs are of the same order of magnitude for the low as well as for the high cost solutions. Only the distribution of the costs are different. Current practice to compare various structures is to capitalise the costs. This means that all costs are transformed to one single year using a formula which takes both the rate of interest and inflation into account. As an example in the table a cost comparison for a groyne is presented. It is a groyne with relatively low wave-attack, but located in an area with strong tidal current, which is typical for groynes in an estuarine environment. To reduce tidal currents wooden piles are place on top of the groynes. A bed protection with boulders around the groyne is required to prevent scouring.

Table 1 gives the overview of the cost in the Netherlands. Such a groyne requires an investment of 725,000 M.U. (=Monetary Units, this can be dollars, rupiahs, or some other currency; the absolute value is not relevant for this comparison). The yearly maintenance cost of the structure amounts 2300 M.U. The economic lifetime of a good-quality groyne is approximately 30 years, after which the residual value is assumed at zero. Applying an

Table 1: Construction of a groyne (In the Netherlands)

Element	No/Units		NL rate	NL cost	Ind rate	Ind cost
Construction cost						
Wooden piles 3.5 m	193	pc	40	7720	20	3860
Wooden piles 5 m	252	pc	70	17640	35	8828
Wooden piles 6 m	12	pc	85	1020	42	504
Wooden piles 7 m	24	pc	100	2400	50	1200
Wooden piles 8 m	4	pc	120	480	60	240
Wood. sheetpile 2.5m	608	m	170	103360	85	51680
Wood. sheetpile 3m	206	m	200	41200	100	20600
Geotextile	6390	m^2	2	12780	10	63900
Placing geotextile	500	hr	40	20000	4	2000
Concrete blocks	3483	m^2	40	139320	30	104490
Placing concr. bl.	2200	hr	40	88000	4	8800
Hydraulic asphalt	194	t	190	36860	150	29100
Stones 25/80	820	t	60	49200	40	32800
Stones 80/200	930	t	55	51150	55	51150
Phosph.slag 40/250	310	t	35	10850	60	18600
Bitum. grouting	460	t	250	115000	250	115000
Extra labour	50	hr	40	2000	4	200
Use of shovel	25	hr	60	1500	10	250
Use of vibration crane	20	hr	70	1400	12	240
Contingencies				5000		5000
	Total			706880		518434
Unforseen, risk	3%			17672		1296
	Grand total			724552		531395
Maintenance costs per year						
labourers	50	hr	40	2000	4	200
Use of shovel	10	hr	60	600	10	100
Extra stones, etc.				1000		1000
	Total			3600		1300
Interest rate 4%, 30 years, multiplier 17.29:						
Capitalized maintenance				62251		22480
Total project cost				786803		553874

interest rate of 4% the total maintenance costs during 30 years are capitalized to the present value, which is 17.3 times the required maintenance cost, arriving at 62,000 M.U. This is only 8.5 % of the total cost.
The discounting factor can be determined using the following formula:

$$Df = \frac{(1+i)^n - 1}{i\ (1+i)^n}$$

in which:

- i interest rate (in this example 0.04)
- n number of years regarded (in this example 30)

Table 2: Construction of a groyne (In Indonesia)

Element	No/Units		NL rate	NL cost	Ind rate	Ind cost
Construction cost						
Wooden piles 3.5 m	193	pc	40	7720	20	3860
Wooden piles 5 m	292	pc	70	20440	35	10220
Wood. sheetpile 2.5m	608	m	170	103360	85	51680
Wood. sheetpile 3m	206	m	200	41200	100	20600
Jute filtercloth	6390	m^2	3	19170	1	6390
Placing jute filter	500	hr	40	20000	4	2000
Pitched stone	3483	m^2	60	208980	25	87075
Placing pitched stone	5000	hr	55	275000	6	30000
low graded asphalt	250	t	150	37500	75	18750
Stones 25/80	2500	t	60	150000	40	100000
Extra labour	200	hr	40	8000	4	800
Use of shovel	25	hr	60	1500	10	250
Contingencies				5000		5000
	Total			897870		336635
Unforseen, risk	2.5%			22447		8416
	Grand total			920317		345041
Maintenance costs per year						
labourers	200	hr	40	8000	4	800
Use of shovel	20	hr	60	1200	10	200
Extra stones, etc.				3000		3000
Replacement of piles	50	pc	85	4250	42	2100
	Total			16450		6100
Interest rate 4%, 30 years, multiplier 17.29:						
Capitalized maintenance				284453		105481
Total project cost				1204770		450522

In a developing country a labourer costs much less, for example only 10% of that in an industrialized country. However, geotextile is much more expensive. Making the same calculation for Indonesia gives quite a different result. An alternative design has been made, see table 2. Geotextile has been replaced by jute, the concrete blocks by pitched stone, etc. Phosphorus slag is easily available in the Netherlands, because it is a rest product of nearby industries. It has been replaced by quarry stone. The stone-weight of the various stones is somewhat lower. Also less asphalt and bituminous material is used. The consequence of his alternative is more damage to the groyne, and thus significant higher maintenance costs.

In an industrialized country the alternative design is absolutely not attractive. Apart from the higher initial costs (920,000 M.U. vs. 725,000 M.U.), the maintenance costs are a real problem (284,00 M.U. vs. 62,000 M.U.). For Indonesia the situation is completely different. There the alternative is on the long run 18% cheaper (450,000 M.U. vs. 554,000 M.U.). Very important and attractive is the 35% lower initial investment (345,000 M.U. vs. 531,000). Of course this is a very rough example. Differences in productivity of a labourer are not accounted for. The price of the boulders depends on the available quarry in the vicinity. Further, the price of the wooden piles may be different.

Note: A practical problem with low investment solutions in countries like Indonesia, is that the organisation of the maintenance is often a problem. This is caused by the fact that infrastructural works often are regarded as "projects" with a begin and an end. Maintenance is a continuous activity, and is therefore difficult to put in a typical project approach. It is therefore very important to pay a lot of attention to the quality of maintenance. It pays off.

As can be seen from this example, on the long run the "low-cost solution" would be (in the Netherlands) more expensive than the "high-cost solution". Therefore it is better to speak about Low-investment Solutions. When the availability of money is no problem, the cheapest solution on the long run should be selected.

As showed above, low-investment solutions require generally more maintenance that capital intensive solutions. Therefore, the coastal structures have to be designed in such a way that maintenance can be conducted as easily as possible with local means, i.e. local material, local labourers and local equipment. These requirements are not new an obviously not very special and can usually be met easily. The snag is to be aware of these requirements during the design phase.

10.2 *Monitoring of condition and inspection*

10.2.1 *Maintenance*

As discussed above, low investment solutions generally require more maintenance than capital intensive solutions. Therefore the construction has to be designed in such a way that maintenance can be performed easily, with local means. Thus, with local material, local people and with local equipment. These requirements are not very special and one can meet these requirements easily. The main problem is that one has to realize these points during the design phase.
For example in case of a rubble shoreline protection, it has to be designed in such a way that repair works can be done directly from the shoreline. So a maintenance road has to be available. The stones have to be placed with local cranes, generally with a short boom. Also the weight of the individual stones should not be too big. This stones have to be available from local quarries. When the construction is designed with concrete elements (because no big stones are available in the neighbourhood), these elements should be simple. Concrete cubes are therefore preferable above fancy elements, like dolosses, akmons and tetrapods.

Maintenance has to be done on the right moment. The main problem with construction requiring maintenance is that often there is no organisation with is responsible for maintenance on an active way. There has to be an maintaining agency and there has to be a maintenance plan. In this chapter the elements of the maintenance plan with emphasis on the inspection and monitoring system will be worked out for a dike system.

Example: A dike system
A dike system forms part of a total water retaining system. consisting of different type of structures. If one part of the system fails, flooding of the inland area may occur. In order to guarantee an acceptable safety level under extreme loading conditions the contribution of the

total length of the dike sections to the overall probability of failure of the potential flooding area amy not exceed a specified value.

Each of the dike sections is composed of a number of structure components, termed as elements. These elements can be related to the detailed water retaining function as derived from the main function.
Examples are:

dike element	*function*
slope revetment:	*hydraulic erosion control*
dike crest:	*control overflow and overtopping*
slope:	*ensure geotechnical stability.*

The elements can be described by a number of characteristic condition parameters, mainly geometrical and material (=strength) parameters. For example the dike crest can be described just by the geometrical parameters height and width of the crest, whereas the description of a slope revetment necessitates specifications of the type, geometry, weight an strength of the successive layers of the revetment.

In common engineering practice of dike design for each of the relevant condition parameters functional requirements have to be specified that should meet the overall safety requirements of the total dike system. After construction of a new dike of enlargement of existing dikes, the actual value of the condition parameters should at least satisfy the specified range.

However, during the service life of the dike a number of deterioration mechanisms may occur that will affect the strength of the structural elements. I other words these deterioration processes may change the values of the condition parameters.
Examples of these processes are gradual settlement of the dike crest due to consolidation of underlying soft soil layers, washing out of finer particles of a stone revetment due to wind, rain and temperature effects and damage effects of slopes due to biological activities such as vegetation and animal burrowing.

These alterations caused by deterioration mechanisms, which will be recognized as characteristic damage patterns, can be of such significance that the dike-safety will no longer be sufficient. This means that the actual quality of the condition parameters may decrease to a level at which the extreme loading condition cannot be sustained.

Figure 10.1 shows the relationship of the damage patterns and the deterioration mechanisms (at daily loading conditions) on one hand and the ultimate failure mechanisms (at extreme loading conditions) on the other hand.

```
deterioration mechanism ====> damage pattern ==> alteration of condition parameter ======> ultimate failure mechanism
```

Figure 10.1: Life cycle of a project

By means of an adequate maintenance and control system of the dike the managing authority aims to secure that the actual quality of the relevant condition parameters do not decrease to a level lower that the acceptable failure limit.
For this purpose the actual state of the condition parameter has to be assessed by periodical

inspection and monitoring of the dikes. In addition based on these observations an supported by theoretical interpretations predictions have to be made of the expected behaviour in the near future. From these two conditions, i.e. the actual state and the expected behaviour, the necessary maintenance measures can be planned.
These measures may vary from "do nothing (=zero option)" or "increase inspection frequency" till "repair or replacement of structural elements".

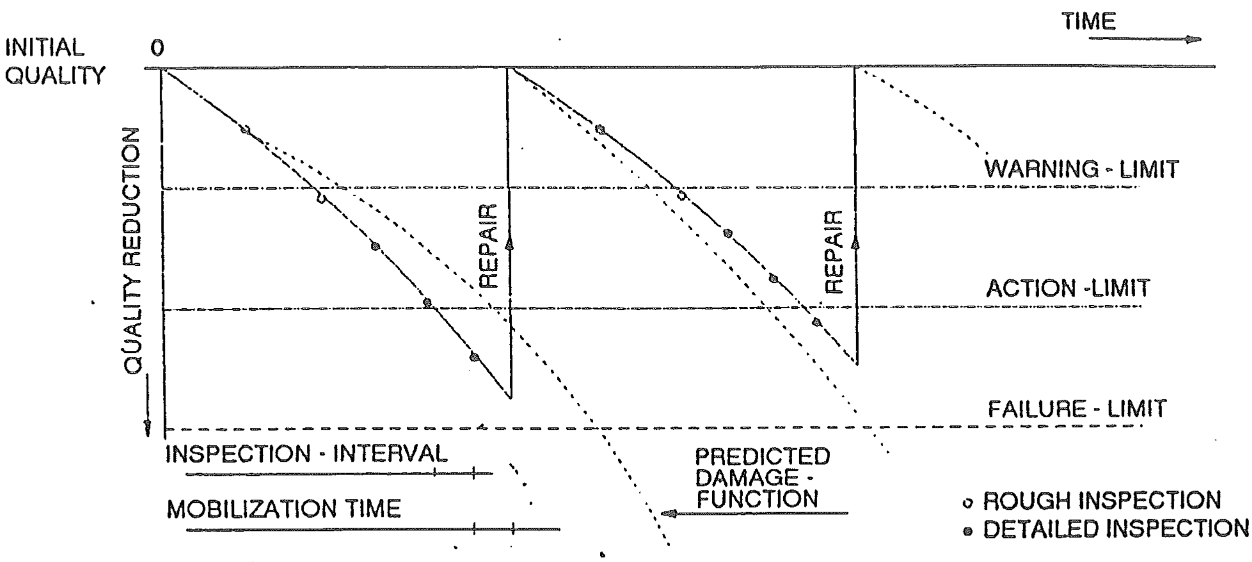


Figure 10.2: Preventive condition-based maintenance

The so-called preventive condition-based maintenance strategy distinguishes three limits of quality levels of condition parameters (see Figure 10.2).

These three limits are:

- warning limit: quality level at which a more intensive control of the condition parameter is needed (higher inspection frequency).
- action limit: quality level at which repair measures should be prepared and carried out before the failure limit has been reached.
- failure limit: quality level that is just acceptable from the safety requirement. If the condition decreases below this level the dike system will not provide sufficient safety.

The margin between the action limit and the failure limit will depend on the inspection frequency and the mobilization time for the execution of repair measures.
An optimum maintenance and control strategy will be obtained by considering the minimum costs of repair and inspection, on the condition that the probability of exceeding the failure limit is sufficiently low.

10.2.2 Organization of maintenance

From the above example follows that a construction, requiring more maintenance, needs to be inspected regularly. The only way to organize this in a proper way is to establish local

based agencies with sufficient independence in order to take care of this maintenance.
It is very difficult and very cumbersome to work out the above system of rational maintenance in all details on paper. Very often, maintenance is obvious. When there is a stone removed from a shoreline protection of pitched stones, this stone has to be replaced. When not, the damage will be progressive. No advance science is needed to decide upon this.

However, the local authority responsible for maintenance should:

* monitor the slope protection, so that one realises that a stone is missing;
* have the direct power to send immediately a men to the place to do the work, at not be enforced to ask permission of the headoffice (e.g. in Jakarta).

The above points seems very clear. But in reality very often, no one is looking after a civil work like a slope protection. An when there is someone doing it, it is very often extremely difficult to have the job done because of red tape. Often permission is required from a head-office before maintenance can start.

The last problem can be solved by creating a floating budget for local authorities. The size of the budget can be determined based upon the experiences from previous years and based upon standard values for maintenance costs in comparable situations in the country. Of course, expenses have to be accounted for.

As an annex to this manual chapter 10 "Maintenance" of the CUR-report 169 on the use of rock in hydraulic engineering [CUR/RWS, 1995] is included.

10.3 Calculation of costs

<< to be filled in at a later stage >>>

10.4 References

CUR/RWS [1995] Manual on the use of Rock in Hydraulic Engineering, CUR, Gouda, ISBN 90.376.0060.3

QUELLERIJ, L. DE AND E. VAN HIJUM [1990] Maintenance and monitoring of water retaining structures. In: *K.W. Pilarczyk, ed. Coastal Protection, Balkema, Rotterdam, Netherlands.*

VERHAGEN, H.J. AND YAP, J.T.L. [1992] Coastal zone management with relation to low investment solutions. *Paper presented at the 2nd PIANC seminar for developing countries, Surabaya, Indonesia*

10. MAINTENANCE

Maintenance of a structure involves a variety of decision problems, flow of information and activities. These are outlined in the flow chart of Figure 10.1.

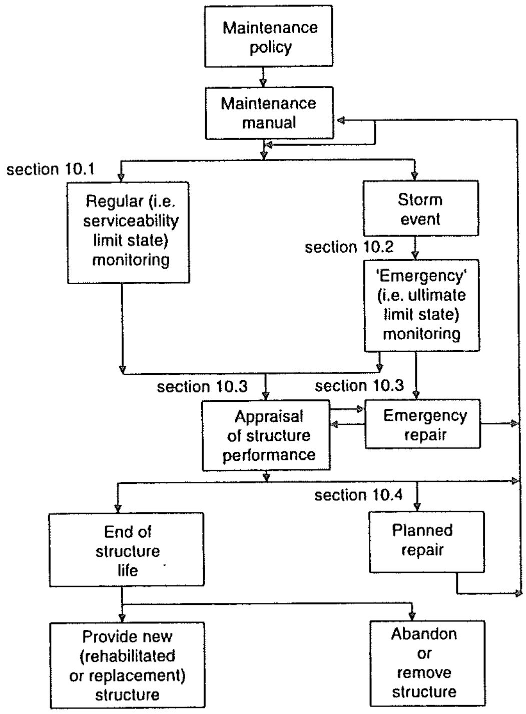


Figure 10.1 *Maintenance program flow chart*

Maintenance of rock structures in hydraulic engineering should be considered as comprising all activities that are required to be carried out on a periodic basis after construction to ensure that a structure performs to an acceptable standard during its lifetime. A maintenance programme will, therefore, in principle include the following essential elements:

i) Inspection and monitoring of environmental conditions and condition or state of the structure;

ii) Appraisal of monitoring data to predict structural deterioration and to assess compliance of performance with pre-determined standards, such as service level and planned lifetime. These standards may by themselves vary through the lifetime of the structure, for instance due to:
- secular trends in water level or wave climate;
- unexpected structural response;
- economic developments;
- unforeseen change of functions assigned to the structure;
- change in criteria used to define the acceptable maintenance level;

iii) The repair or replacement of components of a structure which' life is assessed to be less than the overall structure, or of a localised area which is assessed to have failed.

10.1 Maintenance policy and strategy

The activities to be carried out will be based on a maintenance policy developed at the design stage as part of a management policy for the whole-life performance and costs of the hydraulic structure under consideration. The conceptual framework linking the design, maintenance and the risk of failure is the minimal lifetime cost:

$$\text{Minimize}\{\, I + \text{PV}(M) + \text{PV}(R) + \text{PV}(P_F\, C_F) \,\} \tag{10.1}$$

where:

I = investment in the structure;
PV = present value operator;
M = cost of monitoring;
R = cost of repair or replacement;
P_F = probability of failure;
C_F = cost involved with failure.

With regard to the investment (I) it is noted that possible subsidies must be taken into account. The present value operator is demonstrated in Box 10.1 and an outline of the effect of the PV operator is also discussed in Section 2.1.7 (Box 2.5).
The extent of monitoring required depends on the selected maintenance strategy, which is discussed in the following.

Box 10.1 *Discounting periodic cost to present values*

Let C_r be the expected cost of damage per year, assuming this is spent imediately on repair in order to bring the structure back to its initial strength. For a rock armour layer, C_r relates directly to the damage expressed by the parameter S_d, defined in Sub-Chapter 2.2 and Section 5.2.2. The present value (PV) of an expenditure (C_n) made in the n^{th} year of a structure's design lifetime (N_L) is defined as the ratio C_o/C_n. Here, C_o is the initial capital, which, when reserved at the start of the structure's life cycle, just returns (due to interest) the amount C_n. With a real interest rate r the present value operator reads:

$$\text{PV}(C_n) = \text{PV}(n,r) = \frac{C_o}{C_n} = e^{-r\,n}$$

Then the total reservation for damage (C_{rL}) during the entire lifetime of N_L years can be calculated as the sum of PV's of all C_n for n= 1 to N_L:

$$C_{rL} = C_r \frac{1}{r} \left[1 - \text{PV}(N_L,r) \right]$$

An example is shown in the Figure for a range of values for N_L and different values of r.

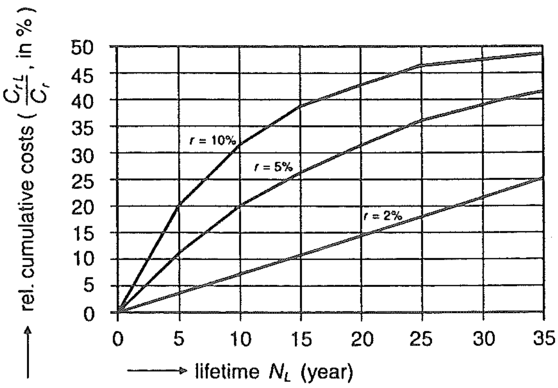


Figure 1 *Relative cumulative cost as a function of N_L for three different interest rates r*

Within this framework several positions may be taken, which leads to the selection of one of the following maintenance strategies (see Figure 10.2):

- failure-based maintenance;
- time-based maintenance;
- use-based maintenance;
- load-based maintenance;
- state-based maintenance.

In case of ***failure-based maintenance***, repair is only undertaken if the stucture or a part has failed. This type of maintenance is advisable if the consequences of failure (risk) are very limited.

Time-based maintenance assumes that the structural state deteriorates according to a known function of time. Repair is due after a certain time has elapsed.
Use-based maintenance reckons that the structural state deteriorates as a known function of the number of times the structure was used (e.g. a spillway). Usage has to be monitored and repair is due after a certain number of times that the structure was operating.
Load-based maintenance attributes the structural deterioation to heavy loading (e.g. storms). Loading has to be monitored and repair is due after a certain number of heavy loadings took place.
State-based maintenance depends on the inspection of the physical state of the structure. If this state seems no longer adequate repair is neccessary.
Given the above given definitions, except for the first, all strategies can be considered more or less preventative and as such monitoring is essential.

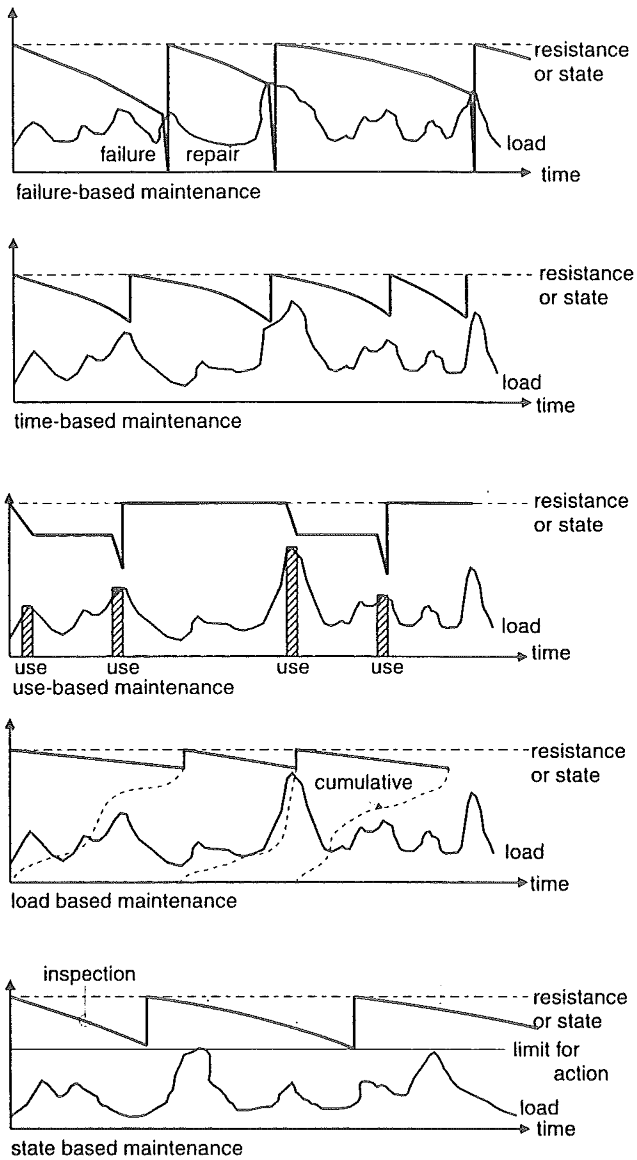


Figure 10.2 *The five principal Maintenance Strategies*

From this classification it may be clear that the choice of the maintenance strategy depends on:

- predictability of the structural deteriation (SLS);
- cost of inspection and monitoring;
- cost of repair;
- consequences of failure (risk);
- availability of methods to measure the physical state accurately.

In the design, as already discussed in Chapter 2, maintenance shall have been taken into account with due regard to safety requirements, available inspection methods and costs for plant, labour and materials needed for repair. The design developed shall also have considered access for monitoring and maintenance and likely as well the engineering and financial resources required for carrying out both the required monitoring, appraisal and repair activities.

It should be realized that, due to the vastness and the often poor (under-water) visibility of hydraulic structures, inspection becomes so costly that an inspection strategy along the same lines as mentioned above for maintenance is developed. Time, use and load-based inspection are introduced in the strategy of state-based maintenance.
The maintenance policy developed at the design/construction stage should be recorded in a maintenance manual. This is always warranted for rock structures in view of their flexible -and sometimes even dynamic- nature. In many situations, particularly where the structure has been designed to minimise maintenance, the manual may be very simple. However, it must contain an indication of the adopted maintenance strategy and the minimum necessary guidance on techniques and criteria for the three basic elements of the maintenance programme listed above. It shall also set out the interrelationships of the various activities involved, a guide to which is given in the flow chart in Figure 10.1, which also shows where in this Chapter information on the three principal types of activity in a maintenance programme can be found.

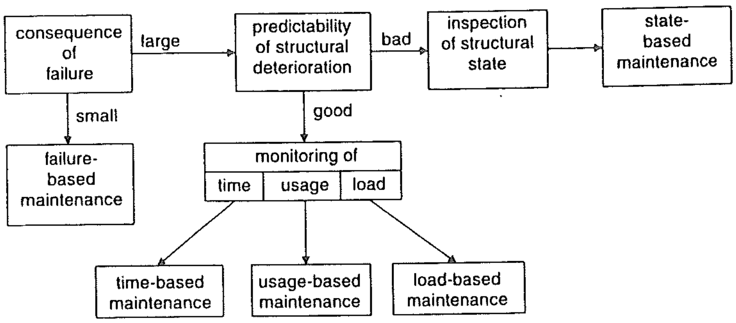


Figure 10.3 *Selection of Maintenance Strategy*

10.2 Monitoring

Hydraulic structures respond to the destructive mechanisms of wave and current action by changes in the profile of the structure and/or changes in the size and shape of its component parts. The changes in shape can also be due to loss or changes to foundation or core material. Typically, armour rock may be displaced, abraded, fractured or even dissolved. Any quantitative description of the state of the structure must be able to identify these different responses, which may either take place *gradually* (SLS) or *catastrophically* (ULS) during a major storm. It must also be able to link the environmental forces to the responses and in Chapter 5 it is shown that for a number of responses this is indeed possible.

Further it should be realized that each failure mode has its own characteristic probability of failure and the consequential cost of damage. This should be considered when setting up a program for monitoring and maintenance. It is essential not to arrive at a too costly schedule. This is demonstrated by means of some examples in Box 10.2.

It should be realized that monitoring is only necessary in preventative maintenance strategies. Failure-based maintenance does not require a monitoring program.

The monitoring methods selected should relate to the potential failure modes for the structure considered and, in particular, to those which have been identified as the most likely (see Section 2.2.2 for a discussion on failure modes and fault trees).
The SLS are most appropriate for monitoring as they show slowly developing phenomena like settlement, erosion, abrasion, etc. Sometimes these SLS are extremely important because they influence the resistance in a connected ULS, that may lead to failure of the structure.

Regarding an ULS, only failure or severe damage can be observed.

A regular monitoring programme for both structure and environment allows the designer or owner to plan repair and replacement activities with a good understanding of slowly acting mechanisms and damage trends. Without the aid of a monitoring programme, deterioration of the armour layers or foundations may go unnoticed and faster than expected, which may ultimately result in the failure of the armour layers or unacceptably large settlements. A management strategy basically comprises four elements: (i) *Measurements* shall be provided of the condition or state of a hydraulic structure at a number of points in time, which can be (ii) *compared* with the design predictions, made with regard to the development as a function of time for the state parameters concerned. Following these steps (iii) a *structural monitoring programme* will then enable unfavourable deviations to be identified at an early stage, which enables the manager to carry out the appropriate remedial *action* (iv).

10.2.1 TYPES OF MONITORING

From the limit-state analysis of failure mechanisms it follows that there are basically three principal areas of monitoring:

a) measurement of structural state (see sub a1) and sub a2) below);
b) measurement of environmental loading conditions;
c) measurement of scour developments (in fact part of b) above).

Structural state monitoring concerns the resistance or strength of the structure and its foundation against loading. Preferably the variable defining the resistance should be measured directly (D_{n50}, thickness of the compressible layer). In many cases however, this is difficult to realize and a proxy is chosen (average stone size, crest level of the structure or overall geometry).

sub a1) The simplest *structural state monitoring* consist of walkover surveys carried out to record, with the aid of photographs, the overall condition of the structure including any obvious responses such as rock movements, changes in geometry or volume etc. This type of survey is always very subjective but has great value if it is performed on the basis of a thorough knowledge of the failure mechanisms of the structure. Here, a maintenance manual provided with the structure can be of great help. An experienced engineer may pick up minor signs of impending failure that escape a formal measurement campaign. It has however only limited use in a detailed quantitative assessment of a structure.

sub a2) A conventional *survey*, taking profiles of the structure above as well as under water, forms the second class of state monitoring methods. The results of these surveys may provide experienced personnel with sufficient data for a detailed quantitative assessment of the actual behaviour of the structure.

sub c) A conventional survey, supplemented with *special measurements* and *soundings* of details of the structure that -according to the design predictions- are prone to deformations related to limit states, is the third class. Besides normal profiles, special soundings in the area where scour holes are predicted, stereogrammetric photography of armour layers and sonar surveys of toe structures should be envisaged, all related to specific limit states (Table 10.1). Fuller details of the survey techniques mentioned are given in the CUR/CIRIA Manual, Appendix A6.

The monitoring of environmental loading conditions may concern the following three and, possibly, four items:

- the environmental conditions; water level, current velocity, wave climate, wind climate, temperature, humidity, live loads;
- the external loading on the structure: differential head, wave load, wave pressure, wave run-up, wave overtopping;
- the internal responses of the structure; soil stresses, pore pressures, accelerations;
- the effect the structure has on the total environment: bathymetry, topography.

Details of environmental conditions, loadings or loading effects which may be monitored, Details of environmental conditions, loadings or loading effects which may be monitored, together with appropriate monitoring techniques, are given in Table 10.2. For a much fuller discussion of environmental data collection, however, the reader is referred to Chapter 4 and, for information on instrumentation, to Appendices B and C.

Table 10.1 *Measurements of the state of hydraulic structures*

Aspects of structure state measured	Output from comparison of structure state at a number of points in time
Walk over	Settlement of foundation Change in alignment
Geometry, profiles	Consolidation of structure Comparison of slope profiles enables overall armour damage parameter (S_d, see Chapter 5), to be determined Scour damage (see Sub-Chapter 5.3)
Profiles + details	Loss or movement of armour rocks Overall sliding of armour layers if this has occurred Voids requiring emergency planned repair
Profiles details and special variables	Rounding of rocks and loss of material, enabling revised assessment of D_{n50} with the design wave climate, or measured wave climate, or revised design wave climate from wave measurements, allows re-assessment of armour stability parameters H_s, ΔD_{n50} using equations in Chapter 5. Comparison with design and measured damage parameters, S, is also possible.

Table 10.2 *Measurements of environmental conditions or loadings*

Environmental condition or loading	Monitoring survey technique (full details in Chapter 4 and Appendices B and C)
Water level	Tide board, visually inspected Data from nearest local tide recording stations Use of surface elevation monitor (step gauge or resistivity gauge) recordings, if available
Wave climate	Seabed pressure meter (robust and cheap) Surface elevation monitor mounted on robust support (e.g. pile, or triangulated scaffold tube arrangement) Wave rider buoy or similar (will be expensive to maintain for long periods) Hindcasting analysis for storm events using wind records
Wind climate	Standard anemograph device (depending on correlation between wind and wave direction may be a useful way of assessing directionality of wave climate)
Wave run-up	Parallel steel wire resistivity guage (survival a problem)
Wave transmission (for breakwaters)	Appropriate wave gauge at reat of breakwater
Mound pore pressures	Piezometers installed within mound with automatic recording facility
Bathymetry and beach topography	Below high water, standard bathymetric techniques possible. Above low water, conventional land-survey techniques may be used or photogrammetry from aerial photography Land-based photography of waterline from fixed positions gives useful assessment of low to high water beach form
Stress in foundation	Pressure pads
Pore pressure in foundation	Piezometers (simple standpie or, for continuous measurements, vibrating wire electronic recording devices may be used)

10.2.2 FREQUENCY OF MONITORING

It is important to commence the monitoring of new structures by establishing a "baseline" set of information regarding the structure and its environment at the time of construction and during the defects liability period. This information should be stored in a form which allows for its retrieval for comparison purposes in future years. The recording of details about the structure during the construction stage is also required as a check against the assumptions and details established at the design stage. Information recorded should not only include basic geometric survey data of profiles etc., but also records of any failures of the rock elements during the construction period. The basic monitoring information should be documented by the Contractor and handed over to the Owner or Designer by the end of the defects liability (or guarantee) period.

The method and the frequency of monitoring is in principle governed by the minimum cost requirement stated above. A balance should be established between investment in the structure, cost of inspection and monitoring, cost of repair and the consequences of failure (risk), all in view of the predictability of the deterioration process of the structure. This may differ considerably from structure to structure and even for mechanisms within a structure. So a practical approach is advized.

loading

The monitoring of the environmental loading conditions should be performed on a continous basis. In some cases the detailed measurements may be limited to severe conditions, to reduce the amount of data to managable quantities, but a complete record in qualitative terms should be kept under all circumstances. Such logs of the wheather, tidal levels, river discharges, etc. are already kept on a routine basis (e.g.) by Meteorological Offices or Water Boards and the Owner of the structure may rely on them. Also hindcasting using proven numerical models may be a economical option.

state monitoring

Structural state monitoring generally puts a considerable strain on the resources of the Owner in terms of finance and personnel due to the vastness the difficult access and the poor underwater visibility of many hydraulic structures. An optimum can be found by developing a monitoring and inspection strategy following the same classification given above for maintenance viz. time, use, load and state-based inspection.

time-based inspection

Time-based inspection concerns the most simple approach, where inspections are performed at regular intervals. The rational minimum interval, based on the changing of the seasons, is 6 or 12 months. Longer intervals up to several years may be chosen if the detoriation process is mainly a function of time and well known (e.g. settlement). An inspection is planned some time before the state is predicted to reach a certain minimum value. If the inspection confirms the prediction, repair will be planned.

use

If the deterioration depends mainly on the usage or the loading, the cumulative use or the cumulative *loading* should form the basis for inspection. Now inspection is due after a specified number of uses (e.g. spillways: load=discharge) or storms (e.g. revetments: load=flood level+wave height). Scour phenomena in river engineering seem very suitable for a load-based inspection scheme, asking for extensive bathymetric surveys after a number of floods.

State-based inspection might be of an incremental nature:
Walkover surveys, carried out by experienced personnel, can form the basis of a decision to perform a suitable in depth monitoring programme involving more resources.
The incremental approach (i.e. more monitoring after a first inspection shows the necessity) may also be based on the increasing knowledge of a previously *insufficiently known* deterioration process (i.e. a matter of uncertainty, $\sigma_{\Delta R}$, see Box 10.2).
Alternatively, deterioration may appear to have been assessed too pessimistic, which then justifies relaxation of the monitoring requirements.
After the first few years of the lifetime of a structure, satisfactory performance may indi-

cate that adequate monitoring will still be achieved if the detailed surveys are reduced in frequency from say every 12 to 24 months or even longer. However, the practical experience of authorities responsible for monitoring is that there is a danger of surveys being forgotten if they are not carried out every year. Related to the *human failure* to inspect is the danger that such infrequent surveys overlook rapid deterioration near the end of the lifetime of a structure, i.e. that such deterioration is not detected before a failure occurs.

Box 10.2 *Example of maintenance assessment*

The following example concerns the cost assessment of a rock bed protection. The bed protection is exposed to a random current attack, causing a transport and consequent loss of stones. In terms of the concept of loading *vs.* strength (*S-R*, Sub-Chapter 2.3) the ***loading*** is represented by the transport of stones. The ***strength*** (R) is represented by the weight of the (armour) stones per unit surface (W_a in kg/m²), which corresponds to a layer thickness ($t_a = W_a/\gamma_a$).

Loading
The distribution $f(U)$ of current velocities is assumed to be Gaussian and characterized by μ_U=2 m/s and σ_U=0.4 m/s. To calculate the transport of stones (S) as a function of velocity a practical transport formula is used:

$$S(U) = 1260 * (U-2)^2 \qquad \text{(kg/m}^2\text{ per year)}$$

Although $S(U)$ is non-lineair and as a result its distribution $f(S)$ deviates from Gaussian, for practical reasons only $f(S)$ is schematized by a mean value and a standard deviation, which are determined by integration of $f(U)*S(U)$. This results in μ_S=101 and σ_S=226 kg/m² per year respectively.

Strength reduction
Since the strength (R) of the bed protection is the stone weight per unit surface (kg/m²), the reduction of the strength (ΔR) of the bed protection is directly associated with the transport (S). Also ΔR is described in terms of a mean value and a standard deviation. Using the characteristics of S, these $\mu_{\Delta R}$ and $\sigma_{\Delta R}$ will be a function of time, for which it is assumed that $\mu = \mu_S * t$ and $\sigma = \sigma_S * t$ (lineair model). Based on this model, the probability densities of ΔR have been calculated for t=1, 5 and 10 years respectively (see Figure 1).

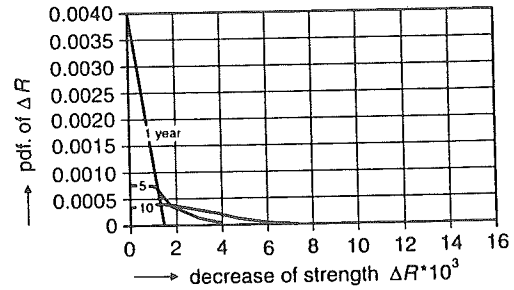


Figure 1 *Probability density function of strength reduction*

Cost of repair
An important aspect of a maintenance strategy is the choice of the length of the maintenance interval. The volume of stone associated with repair after a period t is assumed to be equal to the expected loss of stones, $\Delta R(t)$. The initial strength at t=0 is R_o=3000 kg/m². The loading (S) is characterized by μ_S=100 kg/m² and σ_S=50 kg/m². Further it is assumed that the corresponding cost consist of a initial part (C_i) and a unit rate (C_u). Costs are discounted to present values with an interest rate, r. The example, for which the resulting cost as a function of maintenance interval is shown in Figure 2, is made with the following cost figures:

- loading, (loss) of stones	: μ_S, σ_S=101,226 kg/m² year;
- strength decrease	: $\mu_{\Delta R}$=101, $\sigma_{\Delta R}$=226;
- initial strength	: R_o=3000 kg/m²;
- initial cost (e.g. mobilisation)	: C_i=100 \$;
- unit cost of stone	: C_u=0.01 \$/m³;
- interest rate	: r=0.04.

to be continued

Box 10.2 *Example of maintenance assessment*

continued

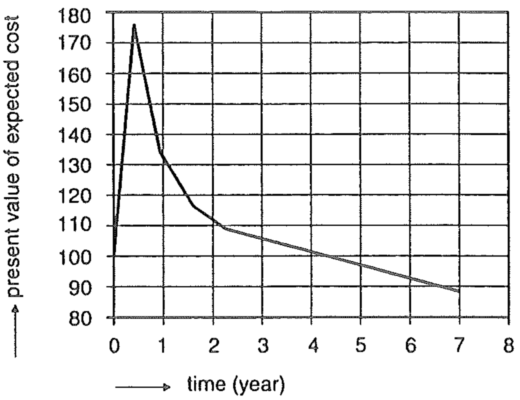


Figure 2 *Present value of maintenance cost*

Probability of failure

As a result of the reduction with time of the strength (μ_R) under constant loading conditions, the probability of failure increases. This can be evaluated using the reliability function Z and the reliability index β_Z (Sub-Section 2.3.2.2), taking $\mu_Z=\mu_R-\mu_S$ and $\sigma_Z=\sqrt{(\sigma_R^2+\sigma_S^2)}$. Subsequently the probability of failure (P_f) is calculated, the result of which is shown in Figure 3.

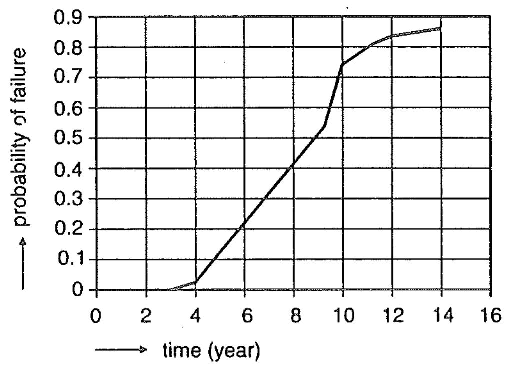


Figure 3 *Increase of probability of failure with time*

Effect of inspection

Finally, the effect of inspection should be mentioned, which mainly implies a reduction of the *uncertainty* with regard to the actual strength (σ_R) of the structure. This is illustrated in Figure 4, where it is assumed that 50% of σ_R is inherent to the physical deterioration process itself. The remaining 50% due to untercainy and is assumed to develop lineairly after inspection to drop back to 0 after the next inspection etc. The inspection interval is chosen to be 2 years. After inspection thus σ_R reduces instanteneously with 50%, which causes an simultaneous (significant) reduction of the actual value of P_f.

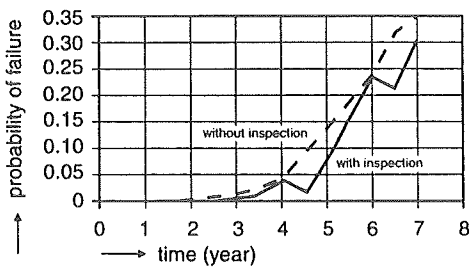


Figure 4 *Effect of inspection on probability of failure*

be continued

Box 10.2 *Example of maintenance assessment*

continued

Conclusions

First it is illustrated that with increasing time, the distribution of ΔR shows more resemblance with a Gaussian one. Further, repair of damage corresponding to this ΔR shows a peak in cost at relatively short time (actual peaks depend largely on ratio C_f/C_u).

Second, as a result of ΔR the probability of failure may increase considerably (of course depending on actual transport S and initial layer thickness R_0). Combining repair cost with the risk associated with failure another (longer) optimal maintenance period will be found. Here a quantification of the risk must be made to transfer the graph of Figure 3 into a risk curve and subsequently find the actual optimum.

Finally, the benefit of inspection (in actual cases to be compared with its cost) lies mainly in reducing the risk of failure. As a result, the optimal maintenance interval will be longer, possibly even at a lower cost.

Two basic frequencies of monitoring can be recognized, one related to normal conditions or the serviceability limit state (SLS) and the other related to extreme conditions or the ultimate state (ULS). Monitoring related to the serviceability state should take place on a planned basis at moments identified at the design stage. Ultimate-state monitoring will only take place during and following severe storm or flood events, for which the minimum wave conditions and/or water levels should be stated at the design stage.

As with the methods of monitoring, the time of monitoring should be pre-determined in relation to the cost of inspection, the cost of repair and the risk associated with particular failure mechanisms or structural elements. The time of those events may well be different for different mechanisms and a practical solution may be necessary to optimise the number of inspections by formulating sensible combinations.
It should finally be realized that the interval between inspections may increase during the lifetime of the structure, reflecting the structure's degressive detoriation process (i.e. a physical reality instead of statistical uncertainty). On the other hand, decreasing intervals between inspections may be required in case of progressively reduced resistance to failure as it degrades with time (Figure 10.4).

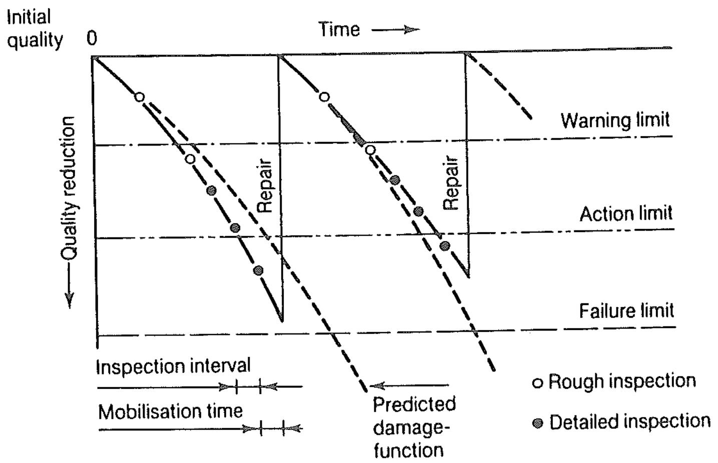


Figure 10.4 *Preventative condition-based maintenance*

10.3 Appraisal of Structure Performance

Having initiated a comprehensive monitoring programme, a designer or owner needs to document all the monitoring information and carry out a review and/or analysis of the data which will form the basis for decisions regarding the need for and extent of maintenance works.

Each structure-state monitoring report should be assessed in relation to the environmental conditions recorded during the interval between two monitoring reports and compared with the predictions that were made for each mechanism in the design. For *ULS* *mechanisms*, the appraisal is relatively simple as damage is only expected when the design conditions have been exceeded. If damage is observed due to less severe conditions, a thorough analysis of the design calculations and the as-built structure is advized.

In the case of *SLS mechanisms*, the pace of the detoriation process can often not be predicted accurately and therefore the first monitoring report is primarily used to calibrate the models used.

During the ensuing life of the structure the results of inspections and monitoring are compared with the predictions of the calibrated model. In this way the data of previous structure-state monitoring reports should be assimilated in the prediction model for the SLS.

It is essential, after each monitoring survey, to make an assessment of the remaining life of the structure in relation to each potential damage pattern (SLS or ULS), in order to monitor the overall safety. The remaining lifetime can be estimated by an extrapolation of the graphical representation of the state variable as a function of time (Figure 10.2). Typical assessments which may be made are listed in Table 10.3 and these will enable a continuous assessment of design and performance criteria.

Table 10.3 *Outputs from comparison of measures of the state of rock structures over a period of time*

Aspect of structure state measured (see Table 45 for details	Output from comparison of structure state at a number of points in time
Level 1: Location	Settlement of foundation Change in alignment
Level 2: Geometry	Consolidation of structure Comparison of slope profiles enables overall armour damage parameter, S_d (see Chapter 5), to be determined Scour damage
Level 3: Composition	Loss or movement of armour rocks Overall sliding of armour layers if this has occured Voids requiring emergency/planned repair
Level 4: Element composition	Rounding of rocks and loss of material (Section 3.3.4), enabling revised assessment of D_{n50} with the design wave climate, or measured wave climate, or revised design wave climate from wave measurements, allows re-assessment of armour stability parameter $H_s/\Delta D_{n50}$ using equations in Chapter 5. Comparison with design and measured damage parameter (S_d) is also possible

The scale of the deterioration effect can also usefully be presented by constructing a kind of "measuring watch". On this watch (see Figure 10.5) both the actual damage level and the damage limits (warning limit, action limit and failure limit) can be presented on a time scale (de Quelerij and van Hijum, 1990).

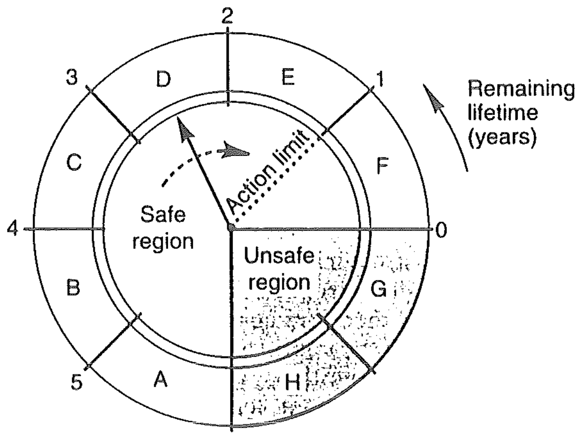


Figure 10.5 *Measuring watch for a selected damage pattern*

To provide a good presentation, the time functions or the "measuring watches" of all relevant failure mechanisms or damage patterns for the hydraulic structure can be collected together into one control panel as illustrated for a sea-wall in Figure 10.6, thereby providing a complete overview of its overall actual and near-future safety condition.

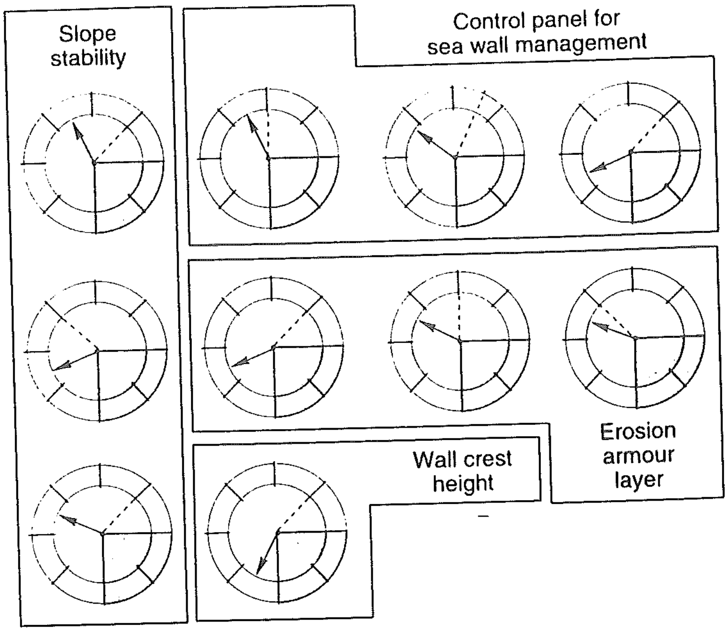


Figure 10.6 *Safety control panel for sea-wall management*

Having completed the assessment, the Owner or Designer may decide to opt for one or more of the maintenance actions indicated in the flow chart (Figure 10.1). His options may be briefly listed as:

1) do no repair/replacement work and await next planned monitoring report;
2) do no repair/replacement work but initiate additional future monitoring of structure state and/or environmental conditions;
3) carry out further detailed inspection before making decision;
4) undertake temporary or 'emergency' repair/replacement works;
5) undertake permanent repair/replacement works;
6) initiate development of a new (rehabilitated or replacement) structure;
7) initiate abandonment/removal of the structure.

The decisions made in response to a monitoring report should be set against performance and failure criteria which have been established at the design stage. It must be recognized, however, that performance and failure criteria can change as technical understanding develops and as the requirements for the function of a structure change, due to changes in use, safety standards, or standards for Environmental Assessment (see Sub-Chapter 2.4).

10.4 Repair/replacement construction methods

The repair construction methods adopted for any particular rock structure will be strongly influenced by decisions taken during the design and project appraisal phase prior to implementation of the structure. Thereby, financial considerations are an important aspect an in particular the maintenance budget reserved for the structure. Generally, it will be uneconomic to underdesign the structure such that, when damage occurs, the armour layer is removed, leaving the underlayers or core exposed. Repairs in this situation are more of the order of a major -and therefore costly- rehabilitation. Economic maintenance procedures will therefore gradually be confined to repairs to the armour layer.
Having set the financial boundary conditions, the main considerations then become availability of and access to materials and plant, leaving basically three options: (i) re-use of present rock, (ii) use fresh rock or (iii) stability improvement of present rock.

sub (i) Since armour stone is a re-usable commodity, in many cases of shoreline protection structures where suitable access is possible, repair works will only require dislodged stone to be *retrieved and placed back* into the face of the structure. Certainly in these cases it can therefore be stated that rock armour is easy to maintain.

sub (ii) In other cases, *fresh armour stone* will be required for repairs. In this case, if provision for suitable access has been made at design stage and there are no financial constraints, the stone can be imported as required. More usual however, to import small quantities of additional armour stone is very expensive, particularly if the source is remote. In addition, access for haulage trucks after construction has been completed may be difficult or impossible. In this case, stockpiling of spare material at the site -as part of the main initial construction operation- should be considered. This rock can be either placed in a mound, or if necessary to soften the impact of a stockpile of large pieces of rock, be buried beneath fine material or used to mark access roads.

sub (iii) Occasionally, none of the above options is practically or financially feasible, and it may be appropriate to consider asphaltic grouting or one of the other *stability improvement* systems discussed in (Sub-)Sections 3.6.7, 3.6.8 and 3.6.10 and 5.2.2.10.

access

Access to plant and along and around the structure, for handling and placing armour stone for repairs, is of prime importance and should be considered carefully when detailing or dimensioning new rock structures. Access for initial construction is very often provided by the underlayers of a partly completed structure or by purpose-built and expensive temporary works, but these forms of access are no longer available for repair works. Even concrete crown walls, sometimes included partly to make provision for future maintenance, are only of value in this regard if it can be guaranteed they will not be damaged by storms.

equipment

Equipment commonly used for repair of armour layers is summarized in Table 10.4 and discussed in Sub-Chapter 9.2, also addressing suitability and potential constraints to accessibility. Handling attachments are also discussed and it is worth highlighting that for repair work the positive pick-up and placing capability of a fixed-arm grab often makes this the favoured attachment. This is because the grab tends to allow more rapid working by enabling easy picking up of rock from a stockpile or from a position already in the structure. It can also be used to push rock into position, if necessary.