ROOSTER

cursus Innovatief denken en doen in kustverdediging

datum 19, 20, 26 en 27 april 2005

Dinsdag 19 april, zaal 0.96

IK 1	09.30 – 10.30	Introductie, overzicht kustverdedigingssystemen *Krystian Pilarczyk, HYDROpil en Rijkswaterstaat DWW*
IK 2	10.45 – 11.45	Kustgedrag op grotere tijd- en ruimteschaal *Marcel Stive, TU Delft*
IK 3	12.00 – 13.00	Hydrodynamica en morfodynamica van het zeegat van Texel *Edwin Elias, TU Delft*
	13.00 – 14.00	Lunch
IK 4	14.00 – 15.00	Kustverdediging (1) Probleem formulering versus oplossing en selectie technieken. Kustmorfologische processen en respons in relatie met het type constructie. Ook zandsuppletie (en in combinatie met); grindstranden, sand-bypassing. *Jan van de Graaff,TU Delft*
IK 5	15.15 – 16.15	Kustverdediging (2)Kustmorfologische processen respons in relatie met het type constructie; stand van zaken met betrekking tot ontwerptechnieken/gereedschappen en (mathematische) modelleringtechnieken. *Jan van de Graaff,*

Woensdag 20 april, zaal 0.96

IK 6	09.30 – 10.30	Zandsuppletie; ontwerp, ontwikkelingen en ervaringen *Ruud Spanhoff, Rijkswaterstaat RIKZ*
IK 7	10.45 – 11.45	Europese regelgeving: Belemmering of uitdaging *Marian Geense, Zelfstandig adviseur*
IK 8	12.00 – 13.00	Algemene principes van functioneel en technisch design, selectie, kosten–baten analyse en integrale aanpak *Henk Jan Verhagen, TU Delft*
	13.00 – 14.00	Lunch
IK 9	14.00 – 15.00	Ecologisch ontwerp kustverdedigingsconstructies *Mindert de Vries, WL/Delft Hydraulics TU Twente en Martin Baptist, TU Delft*
IK 10	15.15 – 16.15	Kustverdediging van de koppen van de Waddeneilanden *Jelmer Cleveringa, Rijkswaterstaat RIKZ*

Dinsdag 26 april, zaal 0.96

IK 11	09.30 – 10.30	Interacties met constructies *Jentsje van der Meer, INFRAM Interntional b.v.*
IK 12	10.45 – 11.45	Functioneel en technisch ontwerp 'beach-connected structures' *Henk Jan Verhagen, TU Delft*
IK 13	12.00 – 13.00	Functioneel en technisch ontwerp offshore breakwaters en reefs *Jentsje van der Meer,*
	13.00 – 14.00	Lunch
	14.00 – 14.30	Introductie IK 14 en 15; situatie Dubai *Mark Lindo, Van Oord n.v. en Ronald Stive HASkoning Nederland b.v.*
IK 14	14.30 – 15.30	Palm Island Jumeirah: ellipsvormig golfbreker (eiland) biedt bescherming aan palmboom/The World: kunstmatig rif pakt "wereld" in *Ronald Stive,*
IK 15	15.45 – 16.45	Palm Island Jumeirah: Uitvoeringsstrategie Palm Jumeirah/Ontwerp en uitvoeringsaspecten van andere kustwaterbouwkundige werken in Dubai *Mark Lindo,*

Woensdag 27 april, zaal 0.96

IK 16	09.30 – 10.30	Andere (innovatieve) systemen en materialen *Krystian Pilarczyk*
IK 17	10.45 – 11.45	Nieuwe ontwikkelingen in dimensionering van golfbrekers en bekledingen *Marcel van Gent, WL/Delft Hydraulics*
IK 18	12.00 – 13.00	De rol van vegetatie bij stabilisatie en destabilisatie van zachte kusten (strand, duinen, intergetijdengebieden) *Frank van der Meulen, Rijkswaterstaat RIKZ*
	13.00 – 14.00	Lunch
IK 19	14.00 – 15.00	Case Maasvlakte II (integrale aanpak) *Tido Vellinga, Havenbedrijf Rotterdam*
	15.15 – 15.30	Video "Goedemorgen Nederland" Marcel Stive
IK 20	15.30 – 16.30	Nederlandse en Europese ontwikkelingen *Moniek Loffler en Hugo Niesing, Rijkswaterstaat RIKZ*
	16.30 – 16.45	Evaluatie en afsluiting

Content:

HYDRAULIC AND COASTAL STRUCTURES IN INTERNATIONAL PERSPECTIVE

Krystian W. Pilarczyk[1]

Abstract: The objective of this paper is to bring some international perspectives on the policy, design, construction, and monitoring aspects of Hydraulic and Coastal Structures in general, and whenever possible, to present some comparison (or reasons for differences) between the experiences of various countries and/or geographical regions. This chapter reviews the *trends* of our hydraulic/coastal engineering profession and presents an overview of the *miscellaneous* aspects, which should be a part of the entire design process for civil engineering structures. This overview ranges from initial problem identification boundary condition definition and functional analysis, to design concept generation, selection, detailing and costing and includes an examination of the construction and maintenance considerations and quality assurance/quality control aspects. It also indicates the principles and methods, which support the design procedure making reference as appropriate to other parts of the chapter. It must be recognized that the design process is a complex iterative process and may be described in more than one way. Some speculation on the possible *future needs and/or trends* in hydraulic and coastal structures in the larger international perspective is also presented briefly.

INTRODUCTION

To place hydraulic and coastal structures in international perspective of users one has at first to define what we understand by this term. In general, each man-made structure in contact with marine environment can be treated as a coastal structure, and when in contact with fresh water (river, reservoir, estuary) as a inland or hydraulic structure. Applying this definition, some traditional/standard civil structures (e.g. a sheet pile, a bulkhead, a concrete wall, etc) will become coastal structures when placed in contact with marine environment, or hydraulic structure when placed in contact with fresh water. Sometimes, the term hydraulic structure(s) is used as an umbrella covering both, inland and coastal structures.

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Usually, by coastal structures we imply a number of typical structures such as breakwaters, jetties, groins, seawalls, sea dikes, sea revetments, etc. Usually the heavy hydraulic loading associated with the marine environment distinguishes these structures from more conventional land and inland applications (hydraulic structures). Therefore, when discussing coastal structures the same items are also valid for hydraulic structures.

The discussion, however, cannot remain limited to the technical subjects only. Most engineering structures have a large impact on the environment, and Society forces the engineering world to mitigate the negative effects of works (if any) on the environment.

When discussing the subject of coastal structures, it is useful to indicate briefly the type of structures and their terminology, and where coastal structures play a role in the marine technology (see Figure 1).

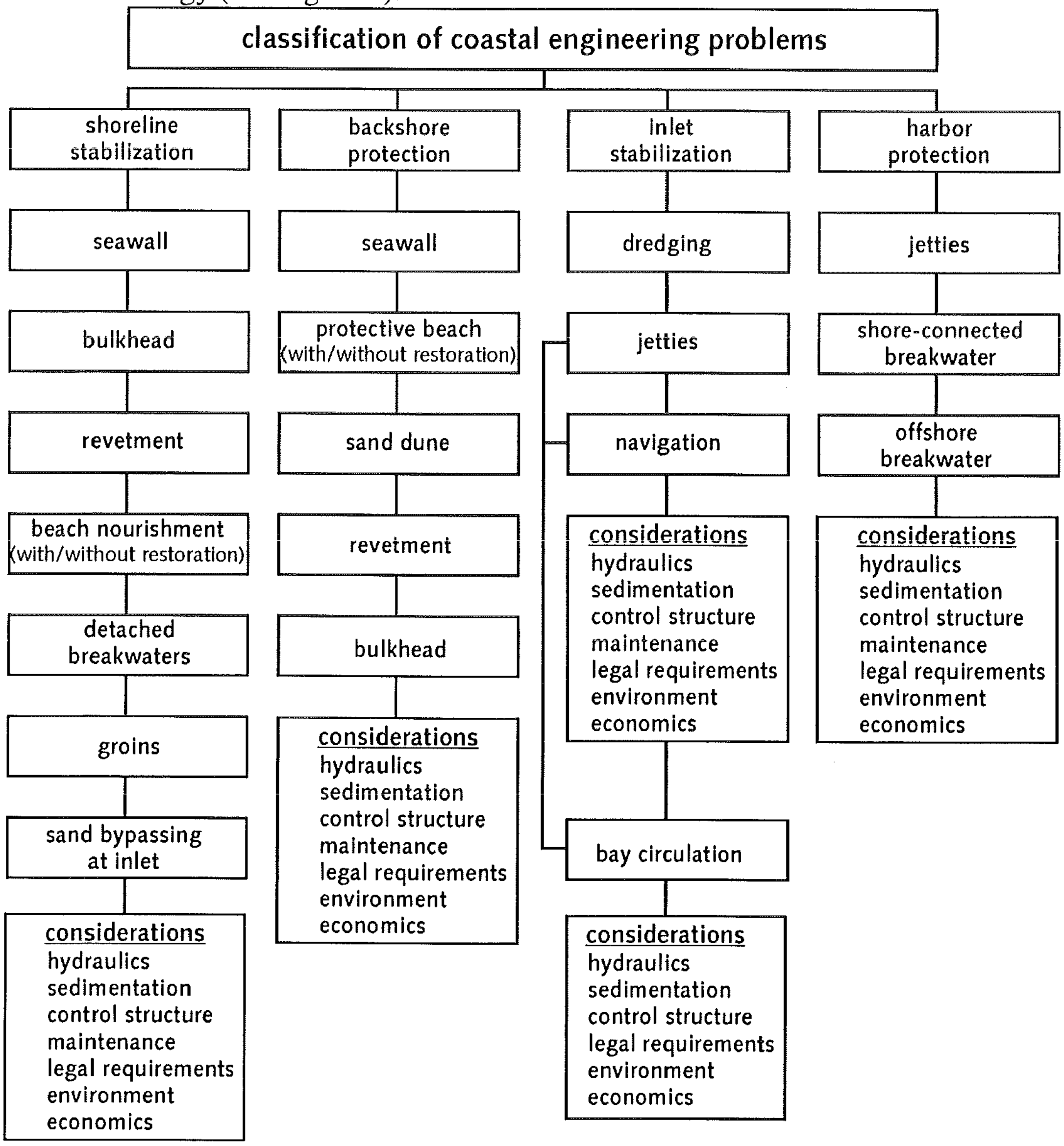


Fig. 1a. Classification of coastal structures (SPM, 1984)

As mentioned above, there are many ways of classification and presentation of coastal structures. For a quick orientation in the scope of functional selection, the classification proposed by Van der Weide (1989) can be applied (Figure 1b). Rock structures can be classified using the ratio between the hydraulic load (e.g., wave height, H_s) and the strength (e.g., ΔD, where Δ = relative mass density of material and D = representative size, i.e., stone diameter); $H_s/\Delta D$. This classification is shown in Figure 1c.

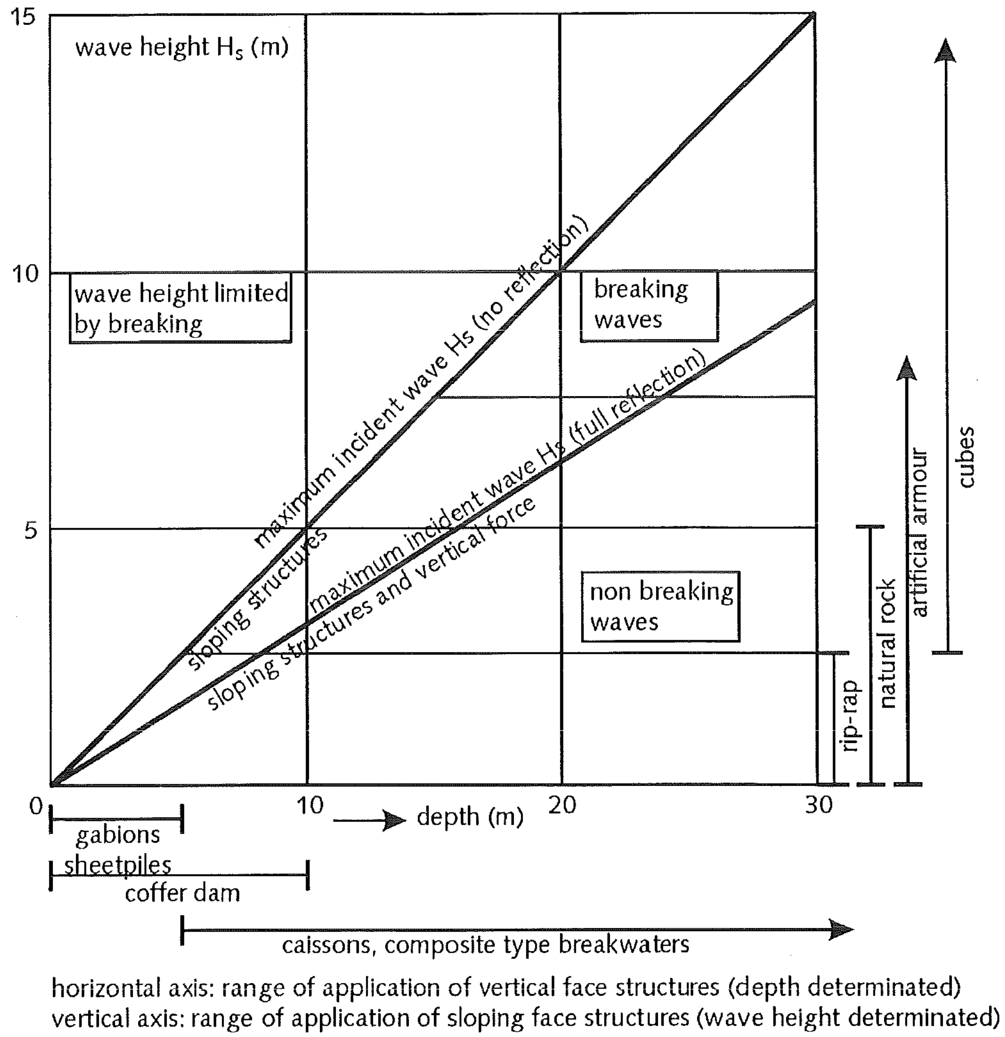


Fig. 1b. Classification of coastal structures according to Van der Weide (1989)

Whenever possible, geographical differentiation and international comparison (various safety standards, use of local materials, equipment and labour, etc.) will be taken into account when discussing this topic. In general it is worth noting that each region/country has its own problems and its own solutions related to technical and economic ability of the country.

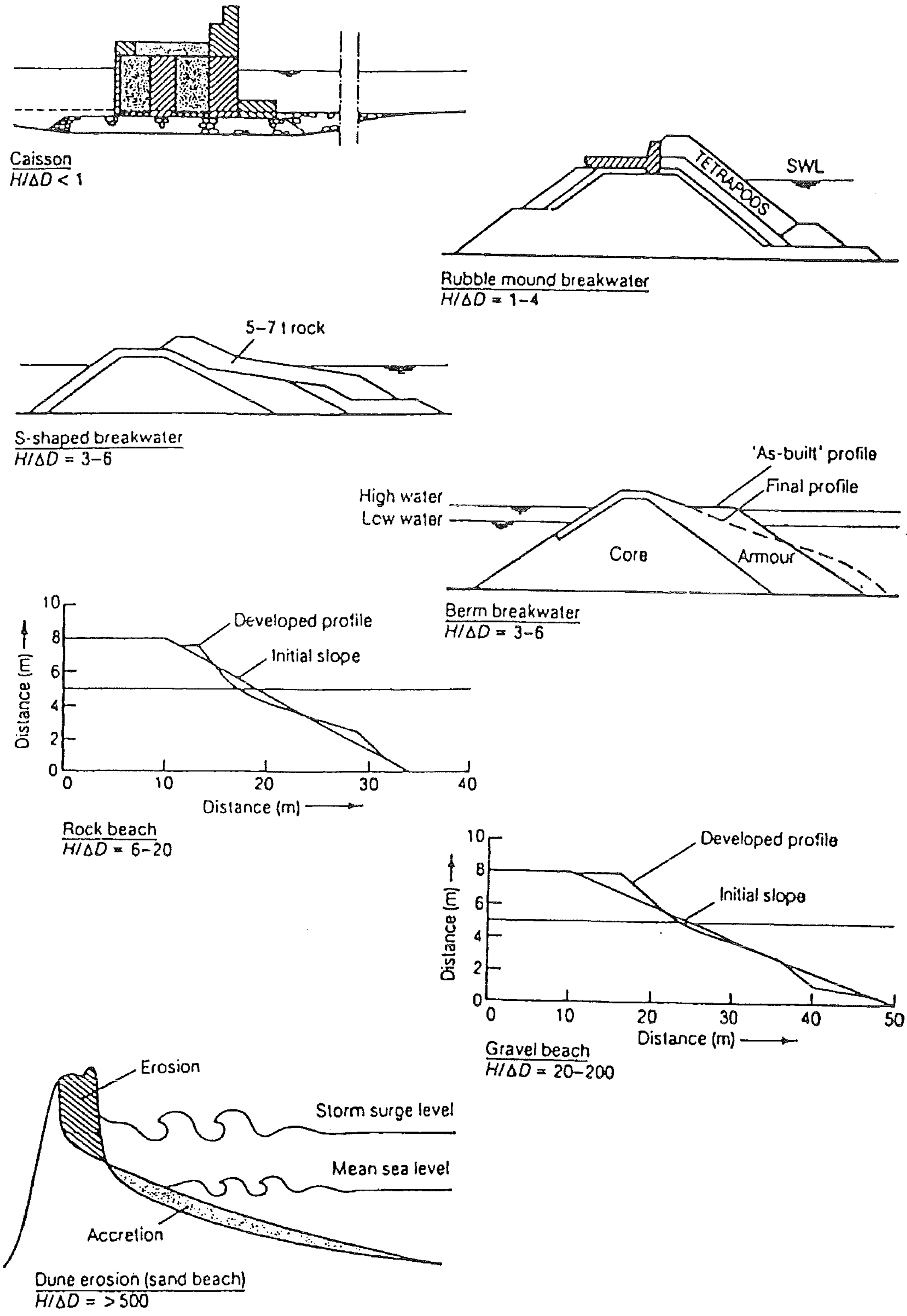


Fig. 1c. Classification of rock structures using $H_s/\Delta D$-parameter (Van der Meer, 1988)

PROBLEM IDENTIFICATION AND DESIGN PROCESS

In general, a coastal (or hydraulic) structure is planned as a practical measure to solve an identified problem. Examples are seawalls and dikes, planned to reduce the occurrence of inundation due to storm surges and/or flooding, or a shore or bank protection structures to reduce erosion.

Coasts and banks appear in many landforms, yet all coasts and banks have one element in common: they form the transition between land and water. With the water being a dynamic element, it is clear that coasts or banks also act dynamically. The design of erosion control structures is one of the most challenging activity for an engineer because of multifunctional character and multidisciplinary interactions and responses namely, interaction between complex hydraulic loading, morphology, foundation (geotechnical aspects) and structural elements (stability) (Przedwojski et al., 1995). It should be stressed that the integrated, multifunctional and multidisciplinary approach to planning and design of hydraulic or coastal structures is still not yet a common approach. Even in the education process of engineers this approach is not always followed.

As indicated earlier, hydraulic and coastal structures are one of the means to solve a water management or a coastal problem. Coastal erosion is one of the most frequent coastal problems. Erosion of the part of the coast, which is often considered to be the most valuable part, viz., beach and dunes (or mainland), is an example of such a problem. To understand the problem and to find a proper control measure one must understand the hydraulic and morphological processes involved. Morphological processes cover those physical processes, which eventually result in the modification of the shape of a coast. The hydraulic and morphological processes in the coastal zone are governed by two primary phenomena, namely wind and tide. The winds are directly responsible for the generation of waves, currents and water-level fluctuations and as a result, for the transport of sand onshore and on the dry beach, while the tides express themselves in a periodic rising and falling of the water and in tidal currents. Strong winds result in extreme storm-surges and high waves, which are the dominant factor for structural stability of structures, flood protection, etc. They belong to "short" duration phenomena although the duration can be even in order of days.

Coastal erosion can be due to two fundamentally different processes (RIKZ, 2002): (i) erosion during a severe storm surge, and/or (ii) structural erosion. Process (i) can be considered as a typical (often heavily, but temporary) redistribution phenomenon. Sand from the dunes and upper part of the beach is transported during the storm surge to deeper water and settles there. Under ordinary conditions, the sand will usually return partly to its pre-storm position. Assuming that there is no gradient in longshore transport, the total volume of sand between some limits of a dynamic cross-shore profile practically does not change due to the storm surge. Process (ii), structural erosion, is quite different from erosion due to a storm surge. This erosion is in most cases due to morphological gradients (mainly, gradients in longshore currents) along the coast. The volume of sand within a cross-shore profile reduces gradually with time. Without additional measures, the upper part of the profile (i.e. dune area) may also be lost permanently. Changing the gradient in longshore current provides a way to reduce the erosion. When an erosion control scheme to a structural erosion problem is designed, one always needs to take into account the consequences of the selected alternative for the erosion process during storm surges.

The methods of interference differ from each other in a way they interfere with the coast. Where the hard methods aim at reducing the sediment transport along the coast or to try to contain the sand on the beach or in the dunes, beach nourishment merely supply sand which consequently will be eroded again. The latter implies that in most cases beach nourishments will have to be repeated regularly in order to protect an eroding coastline in the long term. When these beach nourishments can be reduced, for instance by applying offshore breakwaters, the investment of constructing the offshore breakwaters may be paid back by the (long term) reduction of the beach nourishments (CUR, 1997). All these factors must already be included in the design of project scheme.

There is still much misunderstanding on the use of dikes and seawalls and their possible disadvantages related to the disturbance of the natural coastal processes and even acceleration of beach erosion. However, it should be said that in many cases when the upland becomes endangered by inundation (as in The Netherlands, Bangladesh, Vietnam, and other countries) or by high-rate erosion (possible increase of sea-level rise) leading to high economical or ecological losses, whether one likes it or not, the dike or seawall can even be a 'must' for survival. The proper coastal strategy to be followed should always be based on the total balance of the possible effects of the counter measures for the coast considered, including the economical effects or possibilities. It is an 'engineering-art' to minimise the negative effects of the solution chosen (Kraus and Pilkey, 1988).

In general, designer has always to remember that an effective application of (hard) measures to stop or reduce the gradual erosion in the area under consideration always will result in a reduced input of sediments to the lee-side area. Often this reduced input leads to (increased) erosion in the lee-side area compared to the previous situation. Whether this is acceptable or not depends on the particular case. The lee-side consequences should always be taken properly into account in studying solutions for erosion problems (Pilarczyk, 1990, Pilarczyk and Zeidler, 1995). In conclusion, before making a final choice of a specific measure, the effectiveness and consequences of applying such a measure should be investigated with all available means. Some of these means (models) can give probably only a qualitative answer (show tendencies), but still can be a very useful tool in helping to take a right decision.

Substantial developments have taken place in hydraulic and coastal engineering design over recent years. These have been due principally to an improved scientific understanding of the river and coastal environment and to the development of better analytical and predictive techniques – particularly through mathematical modelling. Although a number of calculation methods have been developed and are applied, the mathematical description of the hydro-morphological processes and the consequent quantitative assessment of the influence of structures on the behaviour of the coastline still are in a first stage. Further developing the description of these processes and incorporating them in computer programs will be required in order to have the tools available to design coastal structures and predict their impact more reliably. A number of promising attempts in this direction are already done in various research centres in Europe, USA and Japan (Van Gent, 1995, Hanson et al., 1996, Larson et al., 1997, Hsu et al., 1999). The closer international co-operation in this field is needed to accelerate these developments, including a proper validation using different geographical site data. However, from the viewpoint of coastal structures design (groins, offshore breakwaters, sea walls, etc.), considerable further developments will still be necessary before the design can be carried out on a fully analytical basis. Experience and engineering judgement still therefore form major elements of good design practice for hydraulic and coastal structures.

Despite of these new developments, a large number of new designs all over the world have not always been successful and the money spent on existing systems has not always yielded the anticipated benefits. As an example, inventory of functioning of groins applied along Dutch coast has indicated that about 50% of them do not fulfil the functional requirements, and sometimes even have an adverse effect. However, due to the relatively high investment involved, the designers and local authorities (to avoid to be blamed for a wrong choice) often defend a certain choice of insufficient or even non-functioning structures. But this wrong choice reflects often only a real state of our knowledge and not an inadequacy of a designer. There is an obvious need for guidelines on the use and effectiveness of coastal structure systems to assist engineers working in coastal engineering planning and management and to give them a basic understanding of design practice.

At the problem identification stage (see example in Figure 2), the presence of an existing or future problem is acknowledged and defined. The acknowledgement of an existing or future problem is generally accompanied by a determination to find an appropriate solution to that problem. In the context of this book, this solution will probably consist of a coastal or shoreline structure, bed protection, or any kind of maritime works. Future problems may be foreseeable as a result of predictable changes or may be generated by proposed engineering works. A simple example of the latter would be the need of protecting the down drift part of a planned shoreline protection (flank protection). Where several options exist, the preferred solution should always be determined as a result of cost/benefit analysis and consideration of environmental impact.

Starting with identification of the problem (e.g. inundation or shoreline erosion), a number of stages can be distinguished in the design process for (and life cycle of) a structure, the subsequent stages of which are determined by a series of decisions and actions cumulating in the creation of a structure (or structures) to resolve the problem. Post-design stages (to be considered during design!) are the construction and maintenance (monitoring and repair) of the structure and, finally, its removal or replacement. An overall formulation in flow chart form is given in Figure 3.

From the design process/life cycle of a structure, one must be aware that the design of a structure may easily develop into a multidisciplinary process, including social conditions, economics, environmental impact, safety requirements, etc.

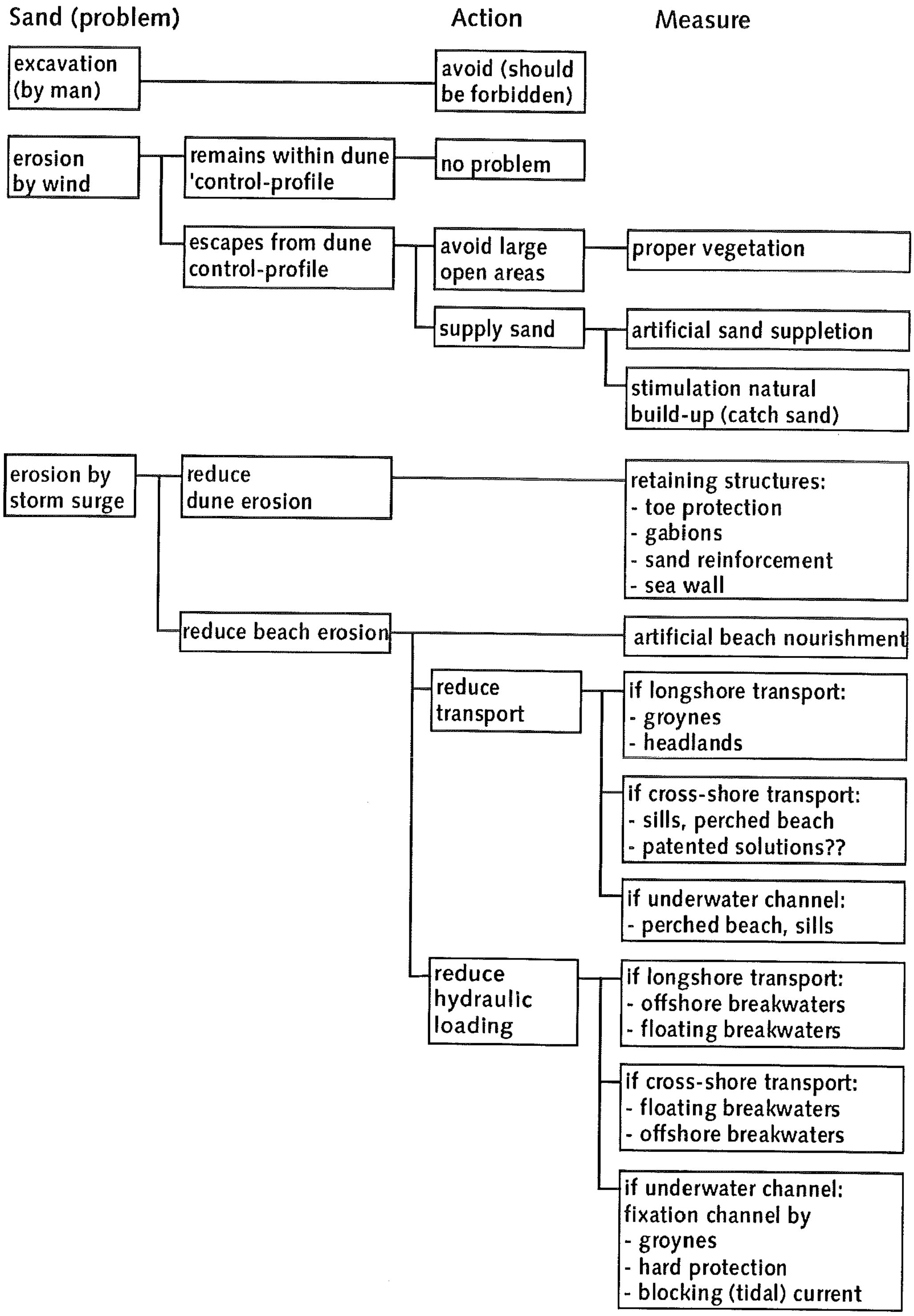


Fig. 2. Identification of coastal problem

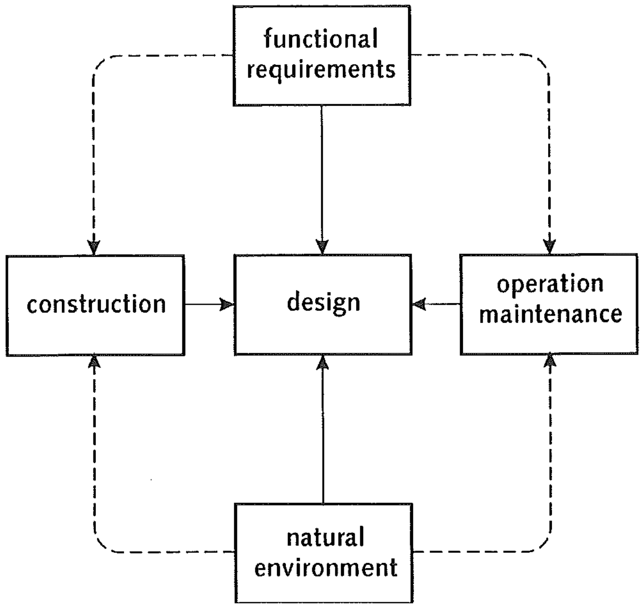


Fig. 3. Main stages in design process (Pilarczyk, 1990)

In conjunction with identification of the problem, all of the boundary conditions, which influence the problem and its potential solutions, must also be identified. These boundary conditions are of various types and include aspects of the following:
- planning policy (including environmental impact aspects);
- physical site conditions;
- construction and maintenance considerations.

Planning policy aspects involve political, legislative and social conditions and include a definition of acceptable risk of failure/damage/loss of life and acceptable/desirable environmental impacts. Therefore, the functional design of hydraulic and/or coastal structures, which aims to combine all these functions and requirements, can be often a very heavy task for the designer.

FUTURE NEEDS IN POLICES AND DESIGN PHILOSOPHIES

Design and construction of hydraulic and/or coastal structures were for many centuries (actually, up to mid- 20th century) based mainly on the system of trial and error, with little scientific backgrounds. However, there has been an increasing need in recent years for reliable information on design methodology and stability criteria of revetments exposed to wave and current action. This need arises partly from an increase in the number and size of applications which have to be realised accordingly to the higher safety standards, and partly from constructing structures at specific locations where they are exposed to more severe wave and current attack (artificial islands, offshore breakwaters, river and sea dikes, waterways and entrance channels with increased intensity and loading due to navigation, etc.).

In the past we have seen only too often, that local experience determined the selection of type and dimensions of the protection system. A satisfying structure of the neighbours was copied, although hydraulic loads and subsoil properties were different. This led to designs, which were unnecessarily conservative and consequently too costly, or were inadequate and thus leading to high maintenance costs. Actually, the technical feasibility and the dimensions of protective structures can easily be determined on a sounder basis and supported by better experience than in the past. Often, the solution being considered should still be tested in a scale model since no generally accepted design rules exist for all possible solutions and circumstances.

Applied design methods, usually site- and material-specific, require often different design parameters, and vary considerably in reliability. As a result, engineers experience particular difficulties when comparing alternative options for new structures and are very restricted in calculations of failure risk and residual life. Bringing more worldwide uniformity in design approaches is a very important factor for overall improvement of reliability of coastal structures. However, proper functioning of hydraulic and coastal structures as an instrument in solving water management and coastal problems is even more important aspect. Both of these components include risks. Managing these risks, equally when there is a strong man-made (e.g. structure) or nature-made component (e.g. flood protection), basically means assessing alternative options under uncertainty. The possibility of multiple fatalities is one of the factors that can vary between options. There is a number of publications which help to increase the awareness of societal risk, and show how to aggregate risks from major hazards, disseminate available knowledge of existing approaches, and exchange information of applications from various domains (e.g., TAW, 2000). The basic components of this integrated approach are shown in Figure 4.

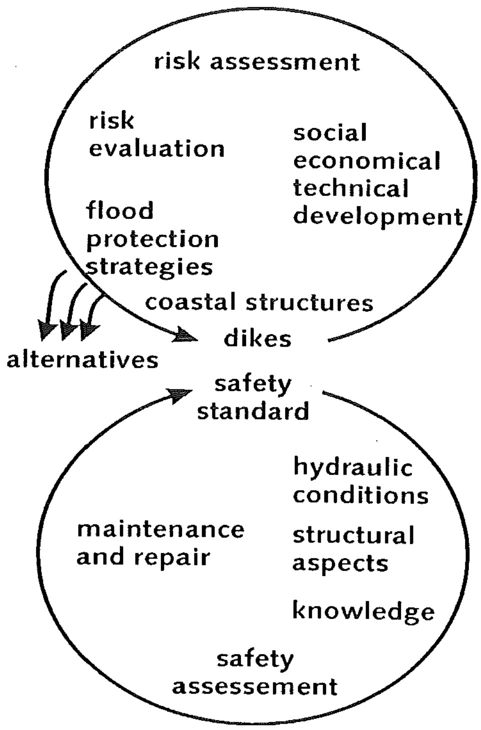


Fig. 4. General approach (TAW, 2000)

This is very important when structures have to function as flood protection, especially for low-lying areas. The higher, stronger and more reliable the flood defences are, the lower the chance they will collapse. Reducing the possibility of consequent damage is the essential benefit of the level of safety inherent in the flood defences. To provide these benefits strengthening the flood defences demands major investment from society. This covers not only the money for flood construction and maintenance. In many cases such construction or improvement of the flood defence means damage to the countryside, natural life or local culture. The demands that are made on the level of protection against high waters also have to be based on balancing of social costs against the benefits of improved flood defences (Jorissen and Stallen, 1998). However, the balance between costs and benefits can also change as a result of changing social insights, at last but not at least, the actual occurrence of floods and flood damage, or the future climate change. To include all these aspects in the design, it is necessary to have the new design techniques cantered on risk-based approach.

In future practice, the results of (much) improved calculations should give rise to the discussion whether the local standards have to be increased (that means also further strengthening of flood defences or other risk reduction measures) in order to comply with existing standards for Group Risk or that the present situation is to be accepted as good practice. However, this discussion can only take place based on an extensive policy analysis. At present, such policy analysis cannot be fully drawn up yet. A lot of technical and non-technical data has to be collected and models have to be developed, but there are more than technical problems. This also implies the new requirements concerning the education of engineers and/or the need to work in multidisciplinary design teams.

Level of protection

Most design manuals are based on a deterministic design philosophy assuming a design water level and a design wave height of predicted return period (say 1 in 50 or 100 years), and the structure is designed to resist that event with an acceptable degree of safety. Probabilistic design methods, applied firstly on a large scale (in coastal engineering) during design and construction of the Eastern Scheldt Barrier in the Netherlands in 1980's, are still not yet a common design philosophy in coastal engineering. However, their use is highly increasing in recent years in western countries. In probabilistic approach, the reliability of the structure is defined as the probability that the resistance of the structure exceeds the imposed loads. Extensive environmental (statistical) data is necessary if realistic answers are to be expected from a probabilistic analysis, and it is mainly for this reason that the procedures have not been frequently used in the past. However, the more uncertainty one has on environmental data and on structure response calculations, the more important it is to use a probabilistic approach. By using this approach one can estimate the uncertainties and their influence on the final result.

For the return period of environmental events used in the design of hydraulic and coastal structures, the actual value or values selected are generally considered both in relation to the level of protection required and the design life of the structure. Both have an important bearing on the subsequent benefit cost study. Where high risk is involved and/or where scheme has a disproportionately high capital cost (i.e. flood barrier scheme, dikes protecting low-lying high density housing/population) extreme return periods of up to 1 in 1000 (or even 1 in 10000) years are chosen to ensure an adequate factor of safety. In case of projects of national importance (i.e. flood protection scheme), usually very costly, grant aid is sough from central government (or international aid agency), in which case agreement is reached early in the design through consultation with the appropriate authority.

Design life

Existing Codes of Practice or Design Guidelines often provide some information on the minimum requirements for the design life of hydraulic and coastal structures (usually as 20 years for temporary or short term measures, 50 to 100 years for shore protection structures and 100 to 1000 years or more for flood prevention structures). However, the proper choice of return period should be carefully investigated base on type and required function of the structure. Also, the probabilistic approach allows to carryout the calculation with respect to cost optimization, which can be a reasonable base for the proper choice of the design return period. Whatever the level of protection, there is always a risk of damage by storms more extreme than the design event. Unless the structure is maintained in a good state of repair, the risk of damage is increased in time.

Similarly, the limitations in the serviceable life of some materials used in the construction (i.e. concrete subject to abrasion, steel subject to corrosion, timber subject to deterioration, etc.) means that they cannot be expected to last the overall life of the structure and, repairs and replacement must be allowed for (must be planned for already in the design stage).

It is unrealistic to expect to design any hydraulic or coastal structure such that it will be free of maintenance or repair during its lifetime. Nevertheless, the ever-increasing requirement to minimise maintenance costs in line with some (national) economic restrictions has a considerable influence on the type of solution ultimately accepted. Cost optimization often shows that it is beneficial to use heavier rock (often only with a little increase of cost) than normally used in a rubble structure, to reduce the risk of damage suffered and so reduce the maintenance requirement (especially in case when mobilizing of material and equipment can be a problem). Conversely, where access and maintenance are relatively easy and where the result of failure is less serious, low capital-cost works are often an economical and acceptable solution.

To continue the functioning of the hydraulic or coastal structures during the prescribed lifetime, their renovation/rehabilitation will be usually needed. In general, in designing rehabilitation or upgrading works the engineer is restricted to a much more greater extent than in new works by the existing conditions. In some cases, complete demolition and reconstruction of the structure (or its part) can be considered as an optimum solution. The design of this type of works is primarily conditioned by the inadequacy of the previous structure to fulfil its original purpose or to meet the requirements of a new and more demanding standard or new boundary conditions (i.e. due to the climate change). In some cases, the wrong functioning of the structure can be proved (i.e. a loss of beach in front of the sea wall). In such a case, a radically different type of structural solution may be evolved, which is more compatible with coastal (or river) processes and the needs of conservation and amenity, (for example, introduction of a shingle beach, sometimes in combination with a groin system, instead of a sea wall, as it was often applied in United Kingdom).

Failure modes and partial safety factors (based on Burcharth, 1999 and others)

For the majority of coastal (or hydraulic) structures, however, like breakwaters, groins and revetments, there seems to be no generally accepted safety or risk levels, and very few design standards comprise such structures. Prof. Burcharth, a driving force in Europe for reliability standards for breakwaters, made in 1999 an interesting attempt to discussion on the safety levels and ways of implementing them in the design procedure of breakwaters, at least in conceptual design stage. In this stage, we basically are evaluating alternative designs and it is of course important that we compare designs with equal functional performance and equal safety.

A (coastal) structure can fail or be damaged in several ways (Figure 5). Consequently, it is very important that the designer considers all the relevant failure modes and assures a certain safety level for each of them. The safety of the whole structure can then be calculated by a fault tree analysis (see Figure 6).

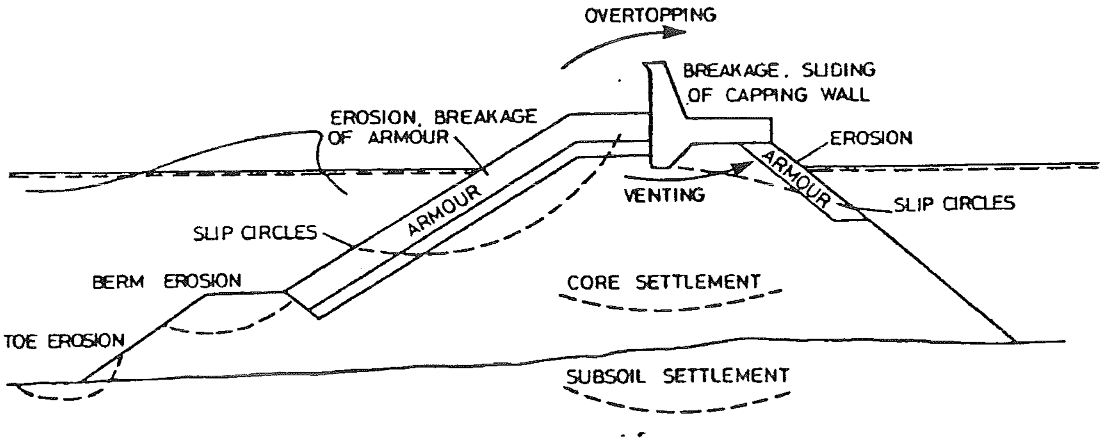


Fig. 5. Potential failure modes for breakwaters (Burcharth, 1994)

It is generally accepted that for each failure mode, implementation of safety in design should be done by the use of partial safety factors linked to the stochastic variables in the design equation, rather than by an overall safety factor on the design equation. An increasing number of national codes (for concrete structures, soil foundation, etc.) and the Euro Code are based on partial safety factors, because this allows a more precise consideration for the differences in parameter uncertainties than an overall safety factor.

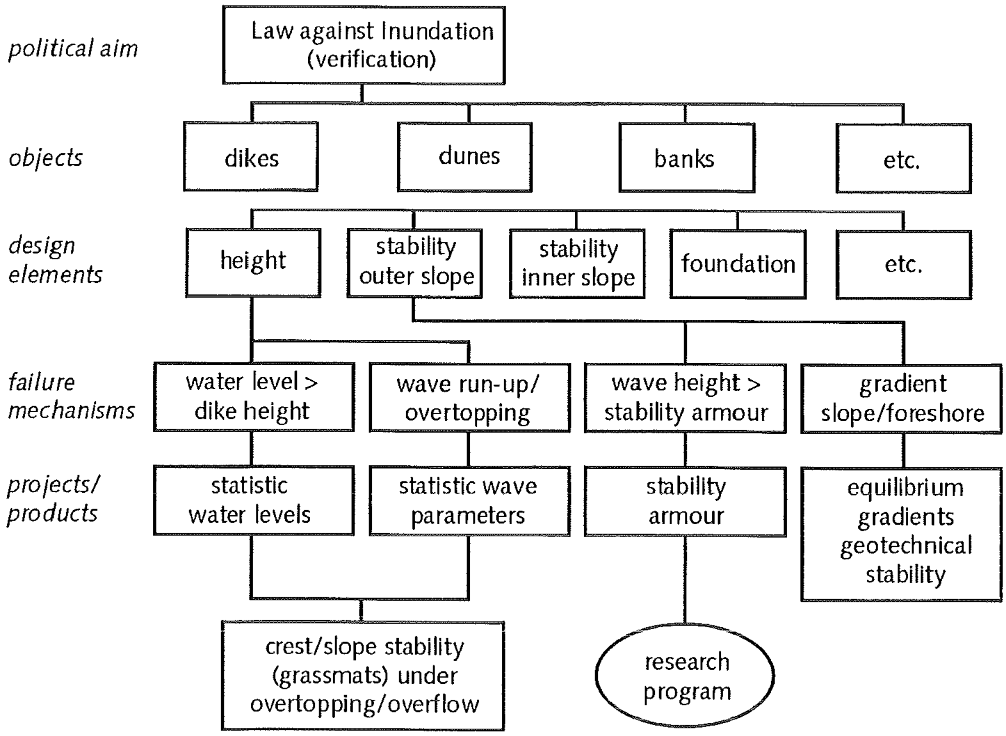


Fig. 6. Example of an event) tree for dikes and correlation between aims and products

The principle of partial safety factors and fault tree analysis related to flood defences is explained in (CUR/TAW, 1990), and for coastal structures is explained in Burcharth (1994, 1999). The formats of partial safety factor systems in existing codes and standards differ, but have the same shortcoming in that the specified coefficients are not related to a specific safety level. They are tuned to reflect the historically accepted safety of conventional designs, and are organized in broad safety classes for which the actual safety level is unknown. Such format is not suitable for breakwaters and coastal defence structures for which no generally accepted designs and related safety levels exist.

On this background, and using probabilistic techniques, a new partial safety factor system was developed for breakwaters, originally in the scope of PIANC Working Group 12 on Rubble Mound Breakwaters (PIANC 1992). The new approach was subsequently used also for vertical wall structures in the PIANC Working Group 28, and further expanded in the EU-MAST 2 and 3 projects Rubble Mound Breakwaters and PROVERBS, respectively. The system allows design to any wanted target safety level (probability of failure) and structure lifetime. This means that structures can be designed to meet any target safety level, for example, 20% probability of certain damage within 50 years.

The PIANC partial coefficient system provides the partial coefficients for any safety level, but no recommendations about safety levels are given, as it is left to the designer to decide on this. Decision can and should be made on the basis of cost benefit analysis (risk analysis). However, because such analysis can be rather complicated and uncertain, there should be defined in the national or international codes and standards some design target safety classes for the most common types of coastal structures. This will certainly accelerate the use of partial safety factors. The safety classes and related failure probabilities could, for example, be formulated as given in Table 1 (Burcharth, 1999a). The useful background information can be found in (Burcharth, 1994).

Table 1. Example of format for safety classes for permanent breakwater structures (numbers only illustrative)

Safety classes:	failure implies:
Very low	no risk of human injury and small environmental and economic consequences
Low	no risk of human injury and some environmental and/or economic consequences
Normal	risk of human injury and significant environmental pollution or high economic or political consequences
High	risk of human injury and significant environmental pollution or very high economic or political consequences

Limit states: Ultimate Limit State (ULS) and Serviceability Limit State (SLS)

Limit State	Failure probability within 50 years structure lifetime			
	Safety class			
	Very low	Low	Normal	High
SLS	0.4	0.2	0.1	0.05
ULS	0.2	0.1	0.05	0.01

Flood protection and management; comparative study for the North Sea coast

Increasing population and development have left coastal areas more vulnerable to a variety of hazards, including coastal storms, chronic erosion and potential sea level rise. Development of coastal areas not only can create increased risk for human life, it also can create a substantial financial risk for individuals and the involved governmental agencies Absolute flood prevention will never be possible although its impact upon human activities can be mitigated in areas of flood hazard. The challenge of flood prevention therefore is to provide an acceptable degree of protection by physical infrastructure combined with alternative means of risk reduction against the most severe floods. A broad range of coastal management functions require good understanding of flooding in order to determine effective policy and response.

In most countries, the provision of flood defenses is undertaken by public authorities (national, regional or local). Thus, the funding flood defense infrastructure forms part of public expenditure and vies with other services and budgets for a share of national and local revenue both for expenditure on new works and on maintenance of existing defenses. Public expenditure on flood defense may be judged on economic return at a national or regional level and is often constrained by political judgments on the raising and distribution of public finances. The time-scales for such political judgments are driven by many factors including public opinion, national and international economic cycles, etc. It may be argued that the provision of effective flood defense can become a "victim" of its own success, with increase pressure to reduce expenditure on flood defense when the defenses appear to remove the flood hazard and the impacts of the previous flooding recede in the public and institutional memories. Thus we may hypothesize a cyclic variation of flood hazard determined by responses to major flood events (Samuels, 2000). Superimposed upon this cycle will be increases in vulnerability from economic and social development within (coastal) flood plains and changes in the climatic forcing and hydrological response.

The problem of flooding is too complex for a complete review. However, there are a large number of excellent publications where useful information on these problems and associated techniques can be found (CUR/TAW, 1990, Przedwojski et al., 1995, Meadowcroft et al., 1995, Vrijling, 1998, Jorissen and Stallen, 1998, Koch, 2000, Oumeraci, 2001). Also, as an example, the results of comparative studies on coastal flooding for some countries along the North Sea are presented below.

The countries along the North Sea coast enjoy both the advantages and disadvantages of this shared neighbour. All countries face the threat of coastal floods to some extent, although the potential consequences of a flooding disaster vary significantly. Each country has developed a system of flood protection measures according to the nature of the threat, potential damages, and its historical, social, political and cultural background. These measures may range from coastal zone planning to evacuation in emergency situations. In all cases, however, construction and maintenance of flood defence structures is the core of these measures.

Recently, the North Sea Coastal Management Group (NSCMG) has agreed to conduct a joint study on the different approaches to safeguarding against coastal flooding. The primary goal of research is to improve communication between the various countries on this subject. The study is limited to coastal defence structures in the five participating countries, namely: Belgium, the United Kingdom, Germany, Denmark and the Netherlands (Jorissen et al., 2001).

The safety offered by flood defence structures, generally expressed as return periods of extreme water levels, seems to vary quite a lot in the different countries. In the United Kingdom, no safety levels are prescribed. Indicative safety levels range from less than 200 years to 1,000 years. In The Netherlands, on the other hand, the legally prescribed safety standards range from 2,000 to 10,000 years. The return period of an extreme water level however, is only one indication of the actual safety provided by the flood defence structures. In practice, the applied data, design procedures, criteria and safety margins determine actual safety. In addition to all this, significant historical, social, cultural and political differences contribute to the variety of flood protection policies, especially with regard to the authorities involved and responsibilities. Table 2 compares some specific aspects of flood protection policies in the five countries.

Table 2. : Overview of flood protection policies

	Netherlands	United Kingdom	Denmark	Belgium (Flanders)	Germany
flood-prone areas	The coastline is 350 km long. Two-thirds of the country (25,000 km^2) is at risk of coastal flooding. The flood-prone area comprises densely populated polders. The capital value at risk is estimated at 2,000 billion euros (1992).	The coastline is 4500 km long. 2,200 km^2 (with 5% of the population), is at risk of coastal flooding: some large urban and agricultural areas, but also very many small areas. The capital value at risk is estimated at 250 billion euros (2000)	The coastline is 7,300 km long. A few towns and some agricultural areas are at risk of coastal flooding.	The coastline is 65 km long and about 3% of total area of Belgium is at risk of coastal flooding.	11,240 km^2 (17,5% of the area) of land at risk of coastal flooding in the coastal states Niedersachsen, Bremen, Hamburg, Schleswig-Holstein.
types of sea defence	• Dunes (70%) • Dikes • storm surge barriers	• sea walls • dikes • dunes, beaches, some shingle • gates and storm surge barriers	• dikes • beaches some sandy	• dikes • dunes and beaches	• dikes • dunes • combination dikes and dunes
Organisation / responsibilities	Centralised policy framework, decision making and engineering, decentralised operational management	centralised policy framework, decentralised engineering and decision making	centralised	centralised (at the level of Flemish region)	centralised (at the level of coastal states)
Legislation	Prescriptive legislation (Flood Protection Act, 1996)	permissive legislation	permissive legislation Act of Reinforcement of Ribe dike, 1976; Fremskudt Dige, 1977	permissive legislation Regionalisation Act, 1988	permissive legislation (State Water Act)
decision criteria	Legal safety standards	economic efficiency, indicative standards	size of the population at risk	absolute standard	absolute standard
safety levels	Statutory standards by dike ring area. Standards are expressed as return periods of extreme water levels. Safety standards in the coastal area range from 2,000 to 10,000 years.	No target risks or flood defence standard; general aim of reducing risks to people and the environment, and requirement to achieve value for money spent. Indicative standards range from less than 200 to 1,000 years.	Safety levels are proposed by the DCA and approved by the Ministry. Safety levels are based on a cost/benefit analysis. Safety standards range from less than 50 to 1,000 years.	A minimum safety level of at least 1,000 years is prescribed according to the Dutch metho-dology.	Safety levels expressed as a combination of design water level, design wave run-up and slope criteria. In practice this standard will exceed a 100 years.

MANUALS AND CODES

Unlike the majority of engineering designs, the design of hydraulic and/or coastal works is not always regulated or formalised by codes of practice or centralised design and construction. In some countries (e.g. Japan, China) the design of hydraulic and coastal structures is based upon a national code of practice, but usually it is based upon a design manual or standardised design guidelines. Such publications may have mandatory effect or be simply advisory. Design Codes of Practice are more useful for less developed countries (or countries with less maritime engineering tradition) where, due to a certain arrears in technological development, too much freedom can lead to the unreliable designs. However, such design codes must be prepared by experts (or at least verified by experts) and periodically upgraded. In general, it should be recommended to upgrade the codes every five years.

In Europe, most countries are using design guidelines instead of design code. A formal code of practice (or a strict formalised design manual) are usually considered to be inappropriate for coastal engineering in view of the somewhat empirical nature of the present design process, the diversity of factors bearing on the design solution (often site dependent), and still the major role that engineering judgement and experience plays in the design process. The term 'guidelines' implies that guidance is given to the engineer responsible for planning and designing coastal structures in that steps in the design/planning process are described; and the considerations involved are discussed, alternative methodologies are set out and their present limitations explained. Usually these guidelines are officially formalized, but still they include a certain freedom in their use; designer may deviate from these guidelines when reasonable arguments are provided or when better (more recent) approved design techniques are used. In this way, designers can follow the actual worldwide developments.

The Netherlands probably provides the best example of construction and use of design guidelines. As a low-lying country, dependent on reliable water defence system, it requires high level of safety and thus, also reliable design and construction techniques. The responsible departments of the Dutch government, under supervision of the Technical Advisory Committee for Water Defences (TAW), have supported the production of a number of overall guidelines and Technical Reports giving guidelines on a general strategies and design philosophies, and on specific technical subjects. These guidelines cover not only the general design philosophy and methodology, but also technical details on failure modes and calculation methods (often developed in own research programs when not available, or not reliable enough on the market). These guidelines were often used as a reference by other countries (especially, countries around the North Sea) for establishment of their own guidelines. The usual period of upgrading these guidelines is about 5 years. There are national standards in the Netherlands on specification of materials (rock, concrete, timber, steel, geotextile, etc., which are gradually replaced by European Standards (EuroCodes).

In Germany, the Committee for Waterfront Structures has produced the design recommendations (EAU 1996, 2000). These are not mandatory regulations, and so can be simply up-dated annually if required. However, Germany is known as a country with a long standardisation tradition in civil engineering applications (German DIN's); it concerns specifications and design methods in a wide range of various materials (concrete, steel, timber, geosynthetics, earthworks, etc.). Also, structural safety is treated by one of these codes. Most of these codes will be actually replaced by Euro codes. When designing coastal structures or their components, reference is usually made to these DIN standards.

In the UK there is little centralised design or construction of coastal structures. In 1984, the British Standards Institution (BSI) issued a Code of Practice for maritime structures. This Code of Practice is not, however, intended to be of direct use in the design of coastal structures. While it considers some subject areas in detail, some other aspects of importance in the design of coastal structures receive very little attention and/or need updating. More information on organisational aspects (policy responsibility) and standards and technical guidelines in UK can be found in (Fowler and Allsop, 1999).

In Spain, design of coastal structures is regulated by a recent document: Recommendations for maritime structures, ROM 0.2-99. The ROM documents gather the leading state of the art knowledge, as well as the extensive experience in maritime engineering in Spain. The objective of the ROM is to define a set of RULES and Technical Criteria, that must be followed in the project design, operation, maintenance and dismantle of Maritime Structures, no matter the materials and methods used in each of the project stages. Concerning the structural safety, the ROM proposes different levels of reliability analysis, for each of the mutually exclusive and collectively exhaustive modes of failure, depending on the general and the operational nature of the maritime structure. Structures with small values of the "nature" (definition of the importance of structure and consequences of failure) can be verified with a "partial safety coefficient level", while those with high nature values are enforced to be verified with the application of a Probabilistic Level II method. An overall procedure is set up in order to guide the designer to fulfil the recommendations prescribed in the program ROM. A software program has been written in order to help designers to follow the ROM. A new revised version of ROM is planned for 2002.

Japan is known as a country working with rather strict design standards. The history and recent developments on design standards for maritime structures in Japan are extensively outlined in the three papers at Coastal Structures'99 (Takahashi et al., Mizuguchi et al., Yamamoto et al., 1999). Originally, depending on the designation and usage of a particular region, coasts were managed by four governmental agencies (construction, transport, fishery, and agriculture), each with its own standards and regulations. This situation was very confusing for everybody, especially because of different design approaches and criteria. Recently, the Ministry of Construction and the Ministry of Transport are combined to the new Ministry of Land, Infrastructure and Transport (MLIT) which fact clarifies the present coastal management situation in Japan. Actually, extensive revision of technical standards on coastal facilities is under way. Already in 1999, it was decided to revise the old 'Technical Standards for Port and Harbour Facilities' and 'Design Standard of Fishing Port Facilities' (see OCADI, 1999/2002). Following that, the Japanese Committee on Coastal Engineering is actually preparing Design Manual on Coastal Facilities, which will be a base for a new Japanese standard (Mizuguchi and Iwata, 1999). Also, the Japanese Coastal Act (dated from 1956), is under revision. The purpose of these revision activities is to harmonise the different approaches and to reflect progresses in coastal engineering from the recent years. The neutral body like Japanese Committee on Coastal Engineering is asked to guide all these revision activities, and in this way to help to resolve the differences within the Ministries involved.

In United States, where the U.S. Army Corps of Engineers is responsible for many coastlines, the most frequently used guide is the Shore Protection Manual (SPM, 1984). This is not an official formalised national Design Code, but in practice it is treated in that way, especially within the U.S. Army Corps organization. This guide was a very modern tool in the 70's and was used worldwide for the design of coastal structures. The advantage of this guide was its completeness, clear style and calculation examples. The disadvantage was its conservatism and limited upgrading in new editions; upgrading was only accepted when the faults become very evident or when much experience with some new techniques was gained, usually outside the U.S.A. As a result, the new design techniques developed often in US were at first applied in Europe or other countries where European consultants were active. Currently, SPM is being replaced by the new and updated Coastal Engineering Manual (CEM, 2002), which becomes a very modern guide reflecting latest developments and which, in combination with recent Manuals on Rock (CUR/CIRIA, 1991, CUR/RWS, 1995) can be recommended for worldwide use as reference.

It should be mentioned here that during the Coastal Structures'99 Conference, a special session on "Guidelines, Standards and Recommendations on Maritime Structures" was held. Speakers invited from different countries around the World (Denmark, France, Germany, Holland, Italy, Japan, UK, USA and Spain) were invited to present the state of the art and the level of development of guidelines in their respective countries. Also, the PIANC Safety Factor System for Breakwaters was discussed. From the presentations, it seems that a large variety of codes, manuals and guidelines with different scopes and objectives are now available; moreover, it is apparent that each country is writing their standards without too much connection with other countries. However, there are many similarities between these documents.

Concerning the structural safety, most of the standards are using the Method of the Limit States as a standard method for the verification of the failure modes. Many countries are still using overall safety factors, while others are developing or using partial safety factors. In order to facilitate the design of breakwaters to any target safety level, the PIANC PTC II Working Groups on breakwaters developed a system of partial safety factors corresponding to any wanted safety level which can be considered as a practical engineering way of using a probabilistic level II method. The PIANC "method" is independent as such of the level of environmental data quality. The partial safety factors given are related to the data quality (poor or good data sets). More details on this subject can be found in the Proceedings of this conference (Losada, ed., 1999).

Future design requirements (codes)

Technical developments will always go on resulting in one or other way in further upgrading of our knowledge and improvement of our design standards. However, each new period brings some new elements and problems, which should explicitly be taken into account and planned in more structural way. The type of design that will need to be undertaken in the 21st century will reflect on one side the type and magnitude of existing work including their ageing and continuing deterioration, which is seen to be mainly in the field of upgrading, extension, rehabilitation and maintenance (CIRIA, 1986). On the other side, the new problems arising due to the long-term changes affecting the coastal regimes, such as the structural erosion in front of coastal structures and the trend towards rising sea levels, steeping of foreshores, land subsidence and continue reduction of sediment supply from the rivers. Some new flood elevation schemes in low-lying areas will probably be required due to new safety standards.

The upgrading and extension of existing structures to meet new defence standards may require the engineer to determine the structural stability of the existing structures in order to determine whether the increased loading is capable of being accommodated. To support the engineer in his new task, new techniques for safety assessment and new criteria for dealing with upgrading of existing structures should be developed. In this respect, the ability to benefit from lessons learnt from the previous works is of paramount importance. The Dutch guide on safety assessment of dikes can be seen as an example of such development (Pilarczyk, 1998, TAW, 1996)

Taking a long-term view, the nature of the requirements will partly depend on the increasing demands that might be made on the coastline due to the continuing upward trend in leisure pursuits, or a greater emphasis on conservation (CIRIA, 1986). Such development may well necessitate a radical change of strategy in coastal defences but it is impossible to predict the type of changes that may result. However from the viewpoint of design, it is already recognised that there is a need to consider coastal engineering strategy over much greater lengths of coastline and over a longer period than at present. Some examples in this direction can be found in the Netherlands (RWS, 1990, TAW, 2000) and in UK (MAFF, 1997, 2000).

Strategic planning on this scale would be helped, if a comprehensive and detailed database of all the existing hydraulic and coastal defences all over the world existed. National authorities and international organisations should initiate some actions for development of such a database (including the lessons learnt from failures) and preparing new guidelines. The state-of-the-art review, which follows, should be based on (international) discussion with design engineers, contractors, research scientists and administrators to present a balanced presentation of a wide range of views. These guidelines should set out the state of the art in each subject area and comment on limitations in actual knowledge.

The introduction of (internationally recognised) guidelines should bring about an increase of reliability (reduction of risk) and a (possible) reduction in the overall cost to the nation of works by (CIRIA, 1986):
- helping the designer to identify the most effective design solution;
- improving the overall level of design practice, so as to reduce the number and cost of over-designed and under-designed works;
- promoting common standards of planning and design, thus improving the effectiveness and co-ordination of coastline control nationally.

The future guidelines should serve a valuable role in setting out a common framework for future planning and design, and for helping to identify the most effective of a number of alternative design approaches with reference to the differences in geographical conditions and economic developments (abilities). Production of the guidelines must not reduce the need to carry out further research into the key areas of design but in opposite, it should stimulate a new research in areas where our knowledge is still limited. Moreover, not all situations can be covered by guidelines, which refer more to standard cases, and there will always be need for additional research (i.e. model investigation) for special problems and high-risk projects. In order to maintain an overall coherence, the design guidelines should be reviewed (internationally) periodically (say, every 5 to max. 10 years) to introduce the advances in the state-of-the-art, and incorporate new experience.

DESIGN TECHNIQUES

The design of hydraulic and coastal structures subjected to currents and wave attack is a complex problem. The design process and methodology are summarised in Figures 7 and 8.

Design methodology

When designing these structures, the following aspects have to be considered:

- the function of the structure
- the physical environment
- the construction method
- operation and maintenance

The main stages, which can be identified during the design process, are shown in Figure 7. The designer should be aware of the possible constructional and maintenance constrains. Based on the main functional objectives of the structure a set of technical requirements has to be assessed.

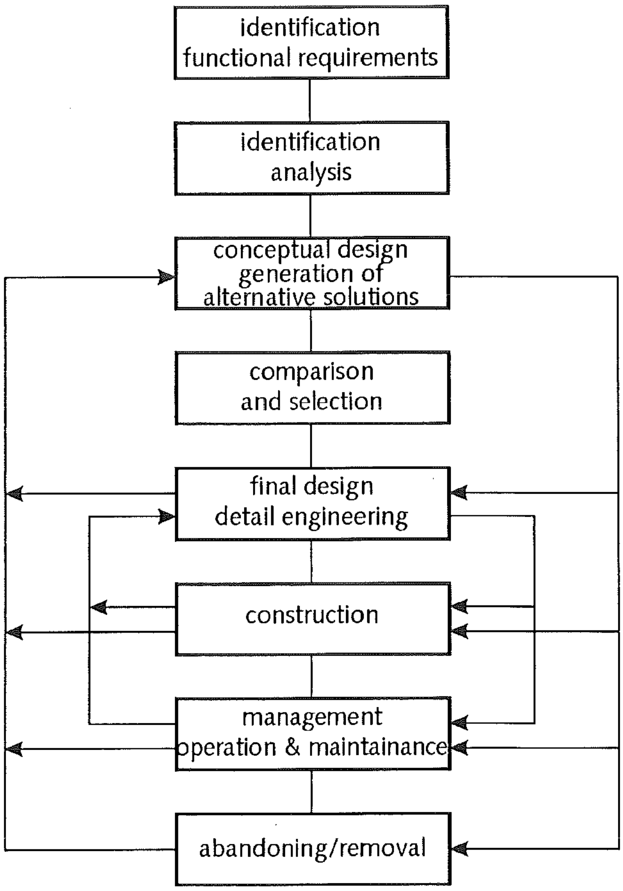


Fig. 7. Design process and integrated approach

When designing a hydraulic or a coastal structure (dike, seawall), the following requirements to be met can be formulated:

1. the structure should offer the required extent of protection against flooding at an acceptable risk,
2. events at the dike/seawall should be interpreted with a regional perspective of the coast,
3. it must be possible to manage and maintain the structure,
4. requirements resulting from landscape, recreational and ecological viewpoints should also be met when possible,
5. the construction cost should be minimised to an acceptable/responsible level,
6. legal restrictions.

A very important stage in the design process is that of making alternatives, both, in case of a conceptual design (making choice of a solution) as well as in a final design (optimisation of the structural design). These are the moments in the design where the cost of project/structure can be influenced. Elaboration of these points mentioned above depends on specific local circumstances as a type of upland (low-land or not) and its development (economical value), availability of equipment, manpower and materials, etc. The high dikes/seawalls are needed for protection of lowlands against inundation while lower seawalls are often sufficient in other cases. The cost of construction and maintenance is generally a controlling factor in determining the type of structure to be used. The starting points for the design should be carefully examined in cooperation with the client or future manager of the project.

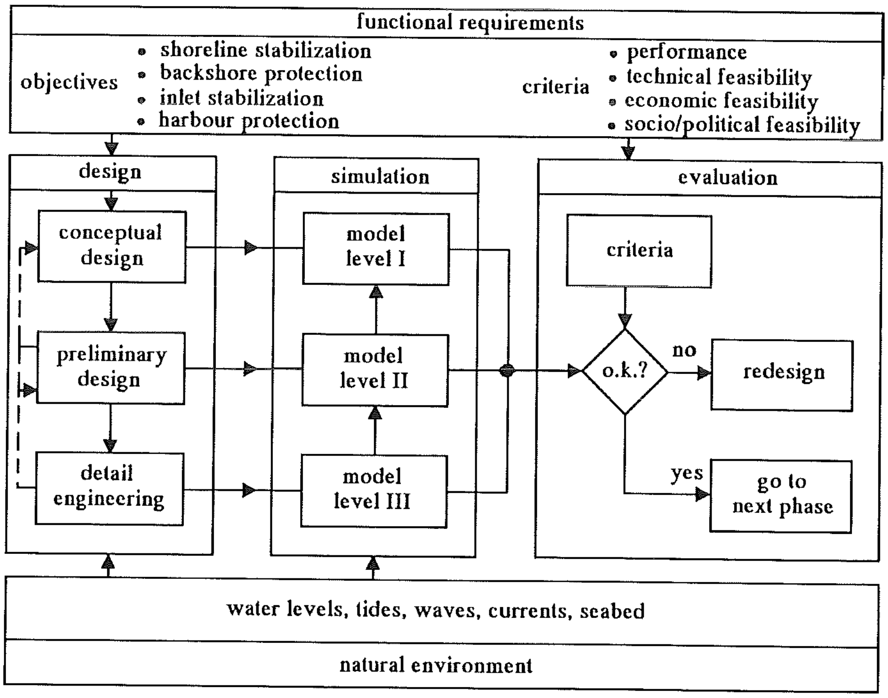


Fig. 8. Design methodology and tools

Note: the meaning of design levels I, II, and III in Figure 8 is different from the terminology related to the design levels using probabilistic approach as discussed in previous Sections (Level I: overall safety factors, Level II: only mean value and standard deviation for stochastic parameters, and Level III: actual distribution of stochastic parameters used).

This design methodology is shown schematically on Figure 8, including also various simulation models (design tools) required to evaluate the behaviour of the structure in the various stages of design (Van der Weide, 1989, Pilarczyk, 1990). In general, it can be stated that in the course of the design-process more advanced methods are used. The actual choice, however, is dependent on the complexity of the problems, the size of the project and the risk-level, which is acceptable.

Depending on the objective, simulation can vary from crude approximations and rules of thumb (usually applicable at level I/conceptual design), through accepted empirical design formulae with their limitations (usually applicable at level II/preliminary design), to sophisticated reproductions of reality, using physical models, analogue techniques or numerical models (usually applicable at level III/final detail engineering). This kind of methodology should be followed both, in a case of functional design as well as in case of structural design. Examples of level I tools (rules of thumb) and level III tools (models) are presented below. Level II tools can be found on the websites: http://ihe.nl/we/dicea/cress.htm or http://www.cress.nl (CRESS-program).

Level I Tools (Rules of Thumb)

Some examples of rules of thumb (tools for first estimate) are given below.

a) *Stability of revetments under wave attack*

$$\frac{H_s}{\Delta\ D} \leq F \frac{\cos\alpha}{\xi_p^{0.5}}; (\mathrm{ctg}\,\alpha \geq 2) \tag{1}$$

with ξ_p= breaker similarity index on a slope; $\xi_p = \tan\alpha\,(H_s/L_{op})^{-0.5} = 1.25\,T_p.H_s^{-0.5}.\tan\alpha$, H_s = significant wave height, Δ = relative mass density of units, D = thickness of cover layer (= D_{n50} for stone), α = angle of slope, F = 2 to 2.5 for rock, = 3 for pitched stone, = 4 to 5 for place blocks, = 5 to 6 for interlocked blocks and cabled block mats.

b) *Maximum (depth-limited) wave height*

$$H_{s,max} = (0.5 \text{ to } 0.6)\ h \tag{2}$$

where *h* is the local depth.
More precisely: use Goda's graphs (Goda, 1985) or ENDEC: http://www.cress.nl

c) *Current attack:*

$$\Delta D = \frac{U^2}{2g} \tag{3}$$

where U is the depth-average velocity and g is the gravity.
Multiply (3/4)D for a uniform flow and (3/2)D for a non-stationary, turbulent flow.

d) *Scour depth (h_{scour}) and length of toe protection ($L_{toe\ prot.}$)*

$$h_{scour} \cong H_s \text{ and } L_{toe\ prot} \geq 2\ h_{scour} \tag{4}$$

where H_s is the local wave height

e) *Minimum length of protection of crest/splash area*

$$L_s > 3D_{n50} \quad (>0.5\text{m}) \tag{5}$$

f) *Granular filter*

$$D_{50up}/D_{50down} \leq 6 \text{ to } 10 \tag{6}$$

for uniformly graded materials (for breakwaters ≤ 3 to5), or more general:

$D_{15up}/D_{85down} \leq 5$; material/soil tightness (for breakwaters ≤ 3 to 4)
$D_{15up}/D_{15down} \geq 5$; permeability criterion
$D_{60}/D_{10} < 10$; internal stability (= uniform grading)

(up = upper layer, down = lower layer)

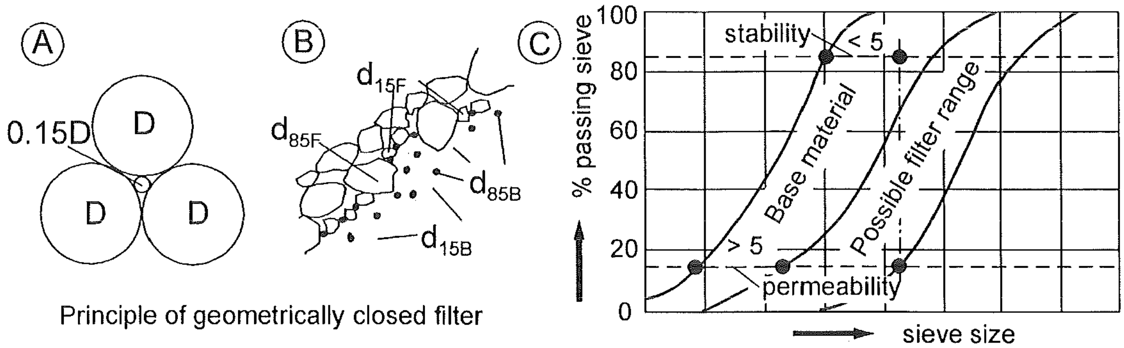


Fig. 9. Geometrically closed granular filters

g) *Geotextile filter (for uniform gradation of subsoil)*

$$O_{90geot} < 2D_{90base} \quad \text{for low turbulence and } H_s < 0.5\text{m} \tag{7a}$$

$$O_{90geot} \leq D_{90base} \quad \text{for high turbulence and/or dynamic loading, } H_s \geq 0.5\text{m} \tag{7b}$$

h) *Wave run-up, $R_{u2\%}$ (= 2% run-up exceeded by 2% of waves only)*

$$R_{u2\%}/H_s = 8\tan\alpha \qquad (\text{for } ctg\alpha \geq 3 \text{ and } 0.03 < H_s/L_p < 0.05) \tag{8}$$

or more general:

$R_{u2\%}/H_s = 1.6\ \xi_p$ for $\xi_p < 2$ to 2.5

and

$R_{u2\%}/H_s = 3.2$ for $\xi_p > 2.5$

where H_s= significant wave height, α = angle of slope and ξ_p = breaker index; for riprap, use 0.6 $R_{u2\%}$.

Level III tools (models)

Knowledge of the relevant wave climate is crucial to the design and construction of coastal structures (do remember: rubbish in rubbish out). Good reliable data measured over a long period is rarely available; and in many cases limitations of time and/or cost do not permit such data to be obtained. The alternative is to derivate long-term estimates of wave climate by hindcasting on the basis of wind data. Special attention still deserves the prediction of wave climate in areas exposed to hurricanes, typhoons, and tsunamis, especially for countries, which do not have own (proper equipped) forecasting services.

An essential parameter in the design of hydraulic or coastal structures is the probability of occurrence of severe events (high water levels, high waves). The design procedure based on the probabilities of water levels alone plus appropriate wave height is presently widely applied. However, this procedure does not reflect the full picture, as it does not allow for the possible correlation between the various parameter causing extreme events (tide, surge magnitude, wind direction, wave height and period). For an optimum design the joint probability of all these parameters should be taken into account. A number of recent manuals and guidelines have included this as a recommended approach; however, its application needs more detailed statistical data and correlations, and probabilistic calculation methods. (CUR/TAW, 1990, CUR/CIRIA, 1992, PIANC, 1992, CEM, 2002). Further developments in this direction should be stimulated, especially concerning the more user-friendly programs for re-working of statistical data into the required full joint design probability.

The next important design activity is to transfer the 'offshore' wave conditions into shallow water. Actually, the most frequently used methods are 1-D models as developed by Goda and by Battjes&Janssen (ENDEC-model). This item is currently the subject of the further improvement of a number of computer programs of various levels of sophistication. The most advanced method is at this moment probably the SWAN-model (**S**imulating **Wa**ves **N**earshore), which was developed by the Technical University of Delft in the Netherlands and is a public domain model (Booij et al., 1996). However, even this model still needs further validation under various conditions. Shallow foreshores considerably affect wave propagation and hence wave impact and run-up on coastal structures. This concerns for instance the evolution of wave height distributions and wave energy spectra between deep water and the toe of coastal structures

As it was already mentioned, there are a number of models available. As an example, two numerical models have been applied in recent studies in the Netherlands to model the wave propagation over the foreshore and one numerical model has been applied to model wave motion on the structure (Van Gent and Doorn, 2001). The models applied for wave propagation over the shallow foreshore are a spectral wave model (SWAN; Ris, 1997 and Ris et al., 1998) and a time-domain Boussinesq-type model (TRITON; Borsboom et al., 2000, 2001). The model applied for modelling wave motion on the structure is a time-domain model based on the non-linear shallow-water wave equations (ODIFLOCS; Van Gent, 1994, 1995). SWAN model simulates propagation of short waves. It does not model processes where bound low-frequency energy becomes free due to wave breaking, which is a relevant process for situations with shallow foreshores. For the modelling of wave breaking (depth-induced wave breaking and white capping), wave set-up, bottom friction and triad wave-wave interaction the default settings were used. The applied Boussinesq-type model is the two-dimensional wave model for wave propagation in coastal regions and harbours TRITON (WL | Delft Hydraulics), which is described in Borsboom et al. (2000, 2001). This efficient model simulates wave propagation and wave breaking in the time-domain, which also allows for simulation of processes where bound low-frequency energy becomes free due to wave breaking. This is a relevant process for situations with shallow foreshores. Wave breaking was implemented based on a new method where wave breaking is modelled as an eddy-viscosity model in combination with a surface roller, similar to the method applied by Kennedy et al. (2000). For the determination of the eddy viscosity use is made of the concept of surface rollers, as also applied by Schäffer et al. (1992). In contrast to many other existing models for wave breaking, this breaker model in TRITON allows for modelling of more *severe* wave breaking, as is the case in the discussed applications.

The general impression from examining the wave energy spectra is that the time-domain model simulates both the spectral shapes and the energy levels rather accurately for the conditions with significant energy dissipation due to severe wave breaking. Also the energy shift to the lower frequencies is modelled surprisingly well. The accuracy of the predictions for the wave periods is higher than obtained with the spectral wave model, though at the cost of higher computational efforts. The model is considered as suitable to provide estimates of the relevant parameters for wave run-up and wave overtopping on dikes with shallow foreshores.

For wave interaction with a dike the model applied here is the time-domain model ODIFLOCS (Delft University of Technology) which simulates wave motion on coastal structures (Van Gent, 1994, 1995). Perpendicular wave attack on structures with frictionless impermeable slopes is simulated by solving the non-linear shallow-water wave equations. Steep wave fronts are represented by bores. Use is made of an explicit dissipative finite-difference scheme (Lax-Wendroff), (Hibberd and Peregrine, 1979). Similar models have been shown to predict well wave reflection and wave run-up on impermeable rough slopes (Kobayashi *et al,* 1987).

Based on the investigations described in this paper the following conclusions can be drawn:

The spectral wave model, applied for wave propagation of short waves over the foreshore (SWAN), yields valuable insight in the evolution of wave energy spectra over the foreshore. It also shows that the computed energy levels in the short waves are rather accurately predicted, considering the rather extreme energy dissipation in the tests. The wave parameters H_{m0} and $T_{m-1,0}$ at the toe of the structure are both under predicted (13% and 21% respectively), using the default settings of this numerical model. Modifications of the numerical model settings for this kind of applications might improve the results. Further improvements of this model could be dedicated to decrease wave energy transfer to higher frequencies and to increase wave energy transfer to lower frequencies.

The time-domain wave model applied for wave propagation over the foreshore (TRITON) shows accurate results for the wave parameters H_{m0} and $T_{m-1,0}$ at each position. The deviations at the toe of the structure remain below 10% and 5% respectively (based on the energy in the short waves). The evolution of the wave energy spectra is rather accurately simulated despite the extreme energy dissipation. Also the energy transfer to lower frequencies is clearly present. The model to include wave breaking in this Boussinesq-type model appears to be effective in reducing the wave energy without significant loss of accuracy in the simulation of wave energy spectra. Further validations of this model include 2DH-situations with angular wave attack, directional spreading and non-uniform depth-contours.

The time-domain wave model applied for the simulation of wave interaction with the dike (ODIFLOCS) shows that accurate results on wave run-up levels can be obtained if use is made of measured surface elevations of the incident waves (on average 10% under predictions of the wave run-up levels). The use of incident waves based on numerical results from the spectral wave model (SWAN) doubles the mean differences (18%) because both numerical models lead to too much wave energy dissipation. The use of incident waves calculated by the time-domain wave model (TRITON) reduces the differences significantly (on average less than 3%) because the numerical models lead to counteracting errors. Applying the two models together led to relatively accurate predictions of the wave run-up levels for the present data set.

Stability of cover layers; some examples

The sudden intensification of research on rubble mound breakwaters in the 80's was triggered by several damage to a series of relatively new breakwaters (Sines in Portugal, Arzew in Algeria and Tripoli in Libya). Also a substantial number of damage cases in Japan led to review of existing design formulae and the development of the famous Goda's formula for vertical breakwaters (Goda, 1985). It was evident that there were fundamental problems with our methods of designing rubble mound breakwaters.
The failure of Sines, Arzew and Tripoli breakwaters are described in Burcharth (1987). The main causes of the failures were:

1) The relative decrease in armour unit strength with increasing size was not considered and/or taken into account. This was crucial for slender, complex types of armour units (Burcharth, 1980, 1987).
2) The second reason for the major failures of rubble mound breakwaters was underestimation of the wave climate.
3) The third reason was bad model testing with incorrect modelling of the structure and the seabed topography.

A lot of research on the strength of slender armour units followed these failures resulting in strength-design formulae for Dolosse and Tetrapods by which one can estimate the tensile stresses as function of incident waves, size of the units and for (Dolosse) the waist ratio. The tensile strength can then be compared to the concrete tensile strength in order to estimate if breakage takes place or not. The formulae also provide the relative number of broken units given the wave climate, the size and the tensile strength of the concrete (Burcharth et al., 2000). The failures involving broken slender concrete units resulted in two trends:

a. return to bulky units like cubes, and
b. development in stronger complex (multileg) units, still with hydraulic stability higher than, for example, cubes. The Accropode is a result of this development.

These failures also stimulated research into concrete material strength problems related to armour units. For example, Burcharth (1984) studied fatigue for slender units. A thermal stress caused by temperature differences in the concrete during curing is a problem related to large bulky units. Burcharth (1983, 1991, 1992) studied this problem theoretically and by full-scale experiments for Sines, and formulated some guidelines to avoid the problem. The introduction of a hole in Antifer cubes was a result of this work. The solution was necessary for casting large cubes in the very hot climate of Somalia (Burcharth, 1991).

All the older design were mostly based on (simplified) formula of Hudson dated from 50's, which gained its popularity due to its simplicity and the status of US Army Corps. However, the problems with using this formula started in 70's with introduction of random waves and the necessity of transformation of regular waves into irregular waves. In 80's, the number of testing facilities and test results with random waves became so large that the necessity of new design formulas became evident.
Note: the Hudson formula is still preferred in some countries, e.g. USA, for shallow water conditions.

The new research in 80's provided more understanding of failure mechanisms and new more sophisticated formulae on stability and rocking of rubble mound structures and artificial armour units.

Formulae developed by Van der Meer (1988) by fitting to model test data, with some later modifications, became standard design formulations. However, the reason of this development was quite different than above mentioned. To explain this we have turn back to 70'th when the author was involved in Delta project, the largest project of damming tidal gaps in the Netherlands, following the necessary actions after flood disaster in 1953. The author has discovered at that time that there was little known on stability of cover layers under wave attack, and that the existing formulations (Hudson, Irribaren, Hedar) were not perfect. He was strengthened in his suspicion by research of John Ahrens, a brilliant researcher from US Corps of Engineers, who was probably too far ahead in time for a general acceptance. Even in his home organisation he never gained the recognition, as he deserved; his work was never mention in US Shore Protection Manual. Ahrens (1975) performed extensive tests on riprap stability and the influence of the wave period; the test were conducted in the CERC large wave tank (with regular waves). Pilarczyk (1983, 1984) continued to replot Ahrens's data and obtained surprising similarity with the later design graph by Van der Meer (1988). This work by Ahrens and his later research on dynamic stability of reefs and revetments, together with work by van Hijum on gravel beaches (Van Hijum and Pilarczyk, 1982), were the reason for the author to prepare a proposals for a systematic research on static and dynamic stability of granular materials (rock and gravel) under wave attack. This program, commissioned early 80's to the Delft Hydraulics, was successfully realized under direct guidance by Van der Meer in 1988.

The basic structure of the Van der Meer formula is such that the stability number H/ΔD is expressed in terms of natural or structural boundary conditions, for example (the sample formula is valid for rock under plunging waves):

$$\frac{Hs}{\Delta D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \frac{1}{\sqrt{\xi_m}} \tag{9}$$

in which: H_s = significant wave height, Δ = relative mass density, D_{n50} = nominal stone diameter, P = permeability coefficient representing composition of the structure, S = damage level, and ξ_m = surf similarity parameter (Iribarren number).

The work of Van der Meer is now generally applied by designers and it has considerably reduced (but not eliminated) the need to perform model experiments during design process. We have always to remember that each formula represents only a certain schematisation of reality. Moreover, as far as these formulas are based on experiments and not based on fully physical understanding and mathematical formulations of processes involved, each geometrical change in the design may lead to deviation in the design results, and to the need of performance of model investigation. Another advantage of the Van der Meer formulae over the formula of Hudson is the fact that the statistical reliability of the expression is given, which enables the designer to make a probabilistic analysis of the behaviour of the design (d'Angremond, 2001). *Note*: the statistical uncertainty analysis for almost all formulae (including Hudson) is given by Burcharth in CEM (2002).

Following the same philosophy, Van der Meer and others have modified and extended the formulae for the stability of rock to many other aspects of breakwater design such as stability of some artificial units, toe stability, overtopping and wave transmission. However, these latest formulae (overtopping and transmission) are still in very rudimentary stage and need further improvement and extension. It concerns especially such structures as submerged reefs with a wide crest where all design aspects (stability, transmission, and functional layout) are not understood properly yet. Some experience with these structures is obtained in Japan, however, the generally valid design criteria are still absent.

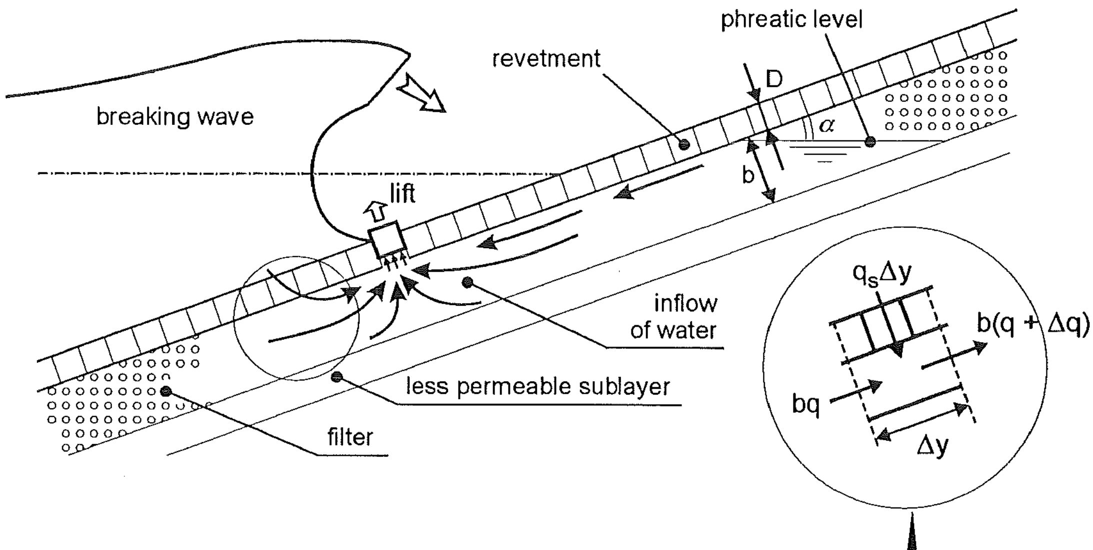
What has been said for slopes under wave attack is largely valid for slopes and horizontal bottom protection under currents. The designer has a number of black box design tools available, but the understanding of the contents of the black box is far from complete. Specifically when these black box design formulae are used in expert systems, one may in the end be confronted with serious mistakes. If an experienced designer still realises the shortcomings and limitations of the black box formula, the inexperienced user of the expert system can easily overlook the implication of it.

Although a reliable set of design formulae is available, the main challenge in the field of rubble mound structures is to establish a conceptual model that clarifies the physical background of it. This will require careful experimental work, measuring the hydrodynamic conditions in the vicinity of the slope and inside the breakwater. Burcharth et al. (1999c) studied the internal flow process in physical models at different scales and in prototype and developed a method for scaling of core material, thus minimizing scale effects on stability. Possibly, an intermediate step has to be taken by developing a 2-D mathematical model (often called "numerical flume") that describes the pressures and flow field with sufficient accuracy, examples of such development can be found in (Van Gent, 1995, Troch, 2001, Itoh et al., 2001). Experimental work will also remain necessary to assess the influence of turbulence. A second challenge is a further exploring of the opportunity to use single instead of double armour layers and further modification of filter rules. This will lead to considerable savings.

The first results of these new developments are promising, but it will take quite some time before this research leads to engineering tools. Another development (and also challenge for future) is the multifunctional design of breakwaters (and may be other structures), for example, a combination of protective function with energy production (wave or tide power), aquaculture, public space for (restaurants, underwater aquarium/parks), etc. Many activities in this direction are already undertaken in Japan. More information on new developments in design techniques and alternative design of maritime structures, and the future needs for research, can be found in ASCE (1995), PHRI (2001), and CEM (2002).

It should be stressed that the proposed developments for breakwaters were partly stimulated/initiated by early developments in understanding and quantification of physical processes in block revetments (De Groot et al., 1988, Burger et al., 1990, Pilarczyk et al., 1995, Kohler and Bezuijen, 1995, CUR 1995, Bezuijen and Klein Breteler, 1996, Klein Breteler et al, 1998, Pilarczyk, 1998, 2000).

Wave attack on revetments will lead to a complex flow over and through the revetment structure (filter and cover layer), which are quantified in analytical and numerical models (CUR, 1995, ASCE, 1995). The stability of revetments with a granular and/or geotextile filter (pitched stones/blocks, block mats and concrete mattresses) is highly influenced by the permeability of the entire revetment system. The high uplift pressures, induced by wave action, can only be relieved through the joints or filter points in the revetment (Figure 10). The permeability of the revetment system is a decisive factor determining its stability, especially under wave attack, and also it has an important influence on the stability of the subsoil. The permeability of a layer of closely placed concrete blocks on a filter layer with and without a geotextile has been investigated in recent years in the Netherlands in the scope of the research programme on stability of revetments



$$\frac{\delta^2\phi}{\delta y^2} = \frac{1}{\Lambda^2}(\phi - \phi_s)$$

Fig. 10. Physical processes in revetment structure

The usual requirement that the permeability of the cover layer should be larger than that of the under layers cannot usually be met in the case of a closed block revetment and other systems with low permeable cover layer (i.e. concrete geomattresses). The low permeable cover layer introduces uplift pressures during wave attack. In this case the permeability ratio of the cover layer and the filter, represented in the leakage length, is found to be the most important structural parameter, determining the uplift pressure. The schematised situation can be quantified on the basis of the Laplace equation for linear flow (Figure 10). In the analytical model nearly all-physical parameters that are relevant to the stability have been incorporated in the "leakage length" factor. For systems on a filter layer, the leakage length Λ is given as:

$$\Lambda = \sqrt{\frac{bDk}{k'}} \qquad (10)$$

where: Λ = leakage length (m), D = thickness of the revetment cover layer (m), b = thickness of the filter layer (m), k = permeability of the filter layer or subsoil (m/s), and k' = permeability of the top (cover) layer (m/s).

The pressure head difference, which develops on the cover layer, is larger with a large leakage length than with a small leakage length. This is mainly due to the relationship k/k' in the leakage length formula. The effect of the leakage length on the dimensions of the critical wave for semi-permeable revetments is apparent from the following equation:

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} \qquad (11)$$

where: H_{scr} = significant wave height at which blocks will be lifted out (m); ξ_{op} = $\tan\alpha/\sqrt{(H_s/(1.56T_p^2))}$ = breaker parameter (-); T_p = wave period (s); Δ = relative mass density of cover layer = $(\rho_s - \rho)/\rho$, and f = stability coefficient mainly dependent on structure type and with minor influence of Δ, $\tan\alpha$ and friction.

This research has proved that the stability of a revetment is dependent on the composition and permeability of the whole system. of the cover layer. Formulas have been derived to determine the permeability of a cover layer and filters, including a geotextile. Also, stability criteria for granular and geotextile filters were developed based on the load – strength principle, allowing application of geometrically open filters, and thus allowing optimisation of composition and permeability of revetments. It is obvious that only a certain force exceeding a critical value can initiate the movement of a certain grain in a structure. That also means that applying geometrically closed rules for filters often may lead to unnecessary conservatism in the design and/or limitation in optimisation freedom (see Figure 11). Also, it often results in execution problems especially when strict closed filter (with many layers) has to be executed under water under unstable weather conditions (Pilarczyk, 1994, Schiereck, 2001).

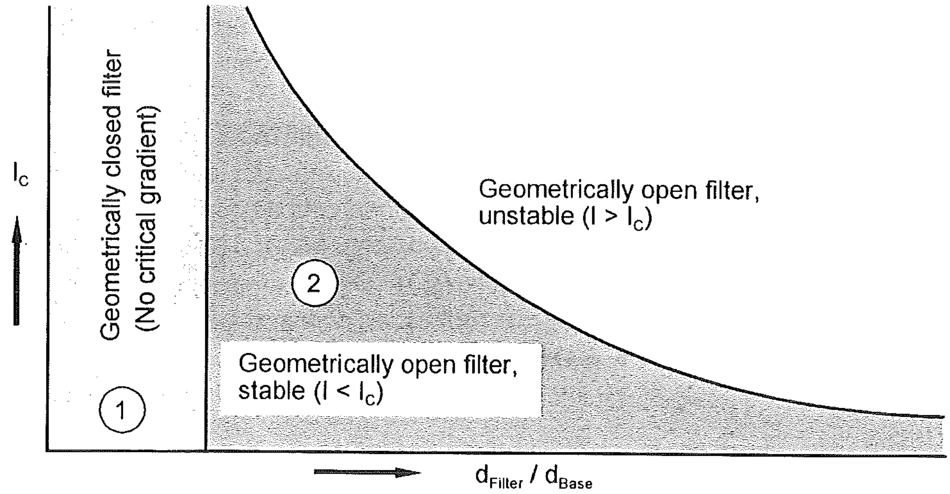


Fig. 11 Application of granular filters (Schiereck, 2001)

In the scope of these studies, also the internal strength of subsoil has been studied in terms of critical hydraulic gradients. It was recognised that to reduce the acting gradients below the critical ones a certain thickness of the total revetment is needed. This has resulted in additional design criteria on the required total thickness of revetment to avoid the instability of the subsoil. That also means that granular filter cannot always be replaced by geotextile only. For high wave attack (usually, wave height larger than 0.5m or high turbulence of flow) the geotextile functioning as a filter must be often accompanied by a certain thickness of the granular cushion layer for damping hydraulic gradients. All these design criteria can be found in Pilarczyk (2000).

The main problem in extension of these achievements to other applications (other revetments, filter structures, bottom protection, breakwaters, etc.) is the lack of calculation methods on internal loads (i.e., hydraulic gradients) for different structural geometry and composition. Also the geometrically open filter criteria need further development. Research in these fields is still needed and will result in more reliable and cost effective designs.

Verification of design

Not all hydraulic or coastal structures or their components are understood completely; moreover, the existing design techniques represent only a certain schematisation of reality. Therefore for a number of structures and/or applications the verification of design by more sophisticated techniques can still be needed.

Whilst certain aspects, particularly in the hydraulic field, can be relatively accurately predicted, the effect of the subsequent forces on the structure (including transfer functions into sublayers and subsoil) cannot be represented with confidence in a mathematical form for all possible configurations and systems. Essentially this means that the designer must make provisions for perceived failure mechanisms either by empirical rules or past experience. However, using this approach it is likely that the design will be conservative. In general, coastal structures (i.e. revetments, sea walls, etc.) are extended linear structures representing a high level of investment. The financial constraints on a project can be so severe that they may restrict the factors of safety arising from an empirical design. It is therefore essential from both the aspects of economy and structural integrity that the overall design of a structure should be subject to verification. Verification can take several forms: physical modelling, full-scale prototype testing, lessons from past failures, etc.

Engineers are continually required to demonstrate value for money. Verification of a design is often expensive. However, taken as a percentage of the total costs, the cost is in fact very small and can lead to considerable long-term savings in view of the uncertainties that exist in the design of hydraulic and coastal structures. The client should therefore always be informed about the limitations of the design process and the need for verification in order to achieve the optimum design.

MATERIALS AND SYSTEMS; SOME EXAMPLES

The cost of production and transportation of materials required for hydraulic and coastal structures is an important consideration when selecting a particular design solution. Thus it is important to establish the availability and quality of materials for a particular site at an early stage when considering design options. Using the available tools and models, the structure can be designed to perform the functional requirements. An additional problem is that these functions will change with time in service because of material degradation processes. Therefore the designer's skill must also encompass consideration of durability and degradation processes. A degradation models for materials and structures should be developed so that the whole-life consequences may be considered at the design stage (NB. a provisional model for armour stone, which considered rock and environmental parameters, is presented in the CUR/CIRIA Manual (1991).

Wastes and industrial by-products as alternative materials

Domestic and industrial wastes and industrial by-products form a still growing problem especially in high-industrialized countries or highly populated regions. A careful policy on application of these materials in civil engineering may (partly) help to reduce this problem. Current European policies aim to increase the use of waste materials of all kinds and to find economic, satisfactory and safe means of their disposal. The use of waste materials in hydraulic and coastal structures is limited by their particle size distribution, mechanical and chemical stabilities and the need to avoid materials which present an actual or potential toxic hazard (CUR/CIRIA, 1991, CUR/RWS 1995, Henneveld and Van der Zwan, 1997).

In the Netherlands, due to the lack of natural rock resources, the application of waste materials in civil engineering has already a long tradition. The extensive research on properties of waste materials allows making a proper selection depending on environmental requirements. Waste materials such as silex, quarry wastes, dredging sludge (depending on the source/location), and many minestone wastes have little or no hazardous contamination. These materials can be used as possible core, embankment fill or filter material. The engineering properties of many waste materials are often comparable or better than traditional materials. Slags have good friction properties due to their angularity and roughness and typically have high density. Mine wastes sometimes have poor weathering characteristics, but are usually inert and have satisfactory grading for deep fills. The fine materials such as fly ashes and ground slags are already in general use as cement replacement and fillers. Good quality control, not only for limiting the potential for toxic hazard, but also of the mechanical properties of waste materials can considerably increase the use of such low-cost materials in appropriately designed coastal and bank protection structures.

Geosynthetics and durability

Geosynthetics are relatively a new type of construction material and gained a large popularity especially in geotechnical engineering and as component for filter structures. There is a large number of types and properties of geosynthetics, which can be tailored to the project requirements (Pilarczyk, 2000). Geosynthetics have already transformed geotechnical engineering to the point that it is no longer possible to do geotechnical engineering without geosynthetics; they are used for drainage, reinforcement of embankments, reduction of settlement, temporary erosion control, and hazardous waste containment facilities (Giroud, 1987). These latest are very often planned as land reclamation along the shores.

When geosynthetic materials or products are applied in civil engineering, they are intended to perform particular functions for a minimum expected time, called the design life. Therefore, the most common (and reasonable) question when applying geosynthetics is 'what is the expected/guaranteed lifespan of these materials and products'. There is no a straight answers to this question. Actually, it is still a matter of 'to believe or not to believe'. Both the experimental theory and practice cannot answer this question yet. However, the Dutch evaluation of the long-term performance of the older applications of geotextiles (back to 1968) has proved that the hydraulic functioning was still satisfactory. A similar conclusion has been drawn from the recent evaluation of the long-term performance of nonwoven geotextiles from five coastal and bank-protection projects in USA (Mannsbart and Christopher, 1997).

The technology of geosynthetics has improved considerably in the years. Therefore, one may expect that with all the modern additives and UV-stabilizers, the quality of geosynthetics is (or can be, on request) much higher than in the 60s. Therefore, for the 'unbelievers' among us, the answer about the guaranteed design life of geosynthetics can be at least 50 years. For 'believers', one may assume about 100 years or more for buried or underwater applications. These intriguing questions on the lifespan of geosynthetics are the subject of various studies and the development of various test methods over the world. Also, the international agencies related to normalization and standardization are very active in this field. The recent guide (European Standard) of the European Normalization Committee presents the actual 'normalized knowledge' on this subject (CEN/CR ISO, 1998). The object of this durability assessment is to provide the designing engineer with the necessary information (generally defined in terms of material reduction or partial safety factors) so that the expected design life can be achieved with confidence.

Geosystems

In recent years traditional forms of river and coastal works/structures have become very expensive to build and maintain. Various structures/systems can be of use in hydraulic and coastal engineering, from traditional rubble and/or concrete systems to more novel materials and systems such as geotextiles/geosynthetics, natural (geo)textiles, gabions, waste materials, etc. Moreover, there is a growing interest both in developed and developing countries in low-cost or novel engineering methods, particularly as the capital cost of defence works and their maintenance continue to rise. The shortage of natural rock in certain geographical regions can also be a reason for looking to other materials and systems. This all has prompted a demand for cheaper, less massive and more environmentally acceptable engineering. However, besides the standard application in filter constructions, the application of geosynthetics and geosystems in hydraulic and coastal engineering still has a very incidental character, and it is usually not treated as a serious alternative to the conventional solutions. That was for the author the main reason to write the state-of-the-art on application of geosynthetics and geosystems in hydraulic and coastal engineering (Pilarczyk, 2000). Some of the recent applications are shown in Figure 12.

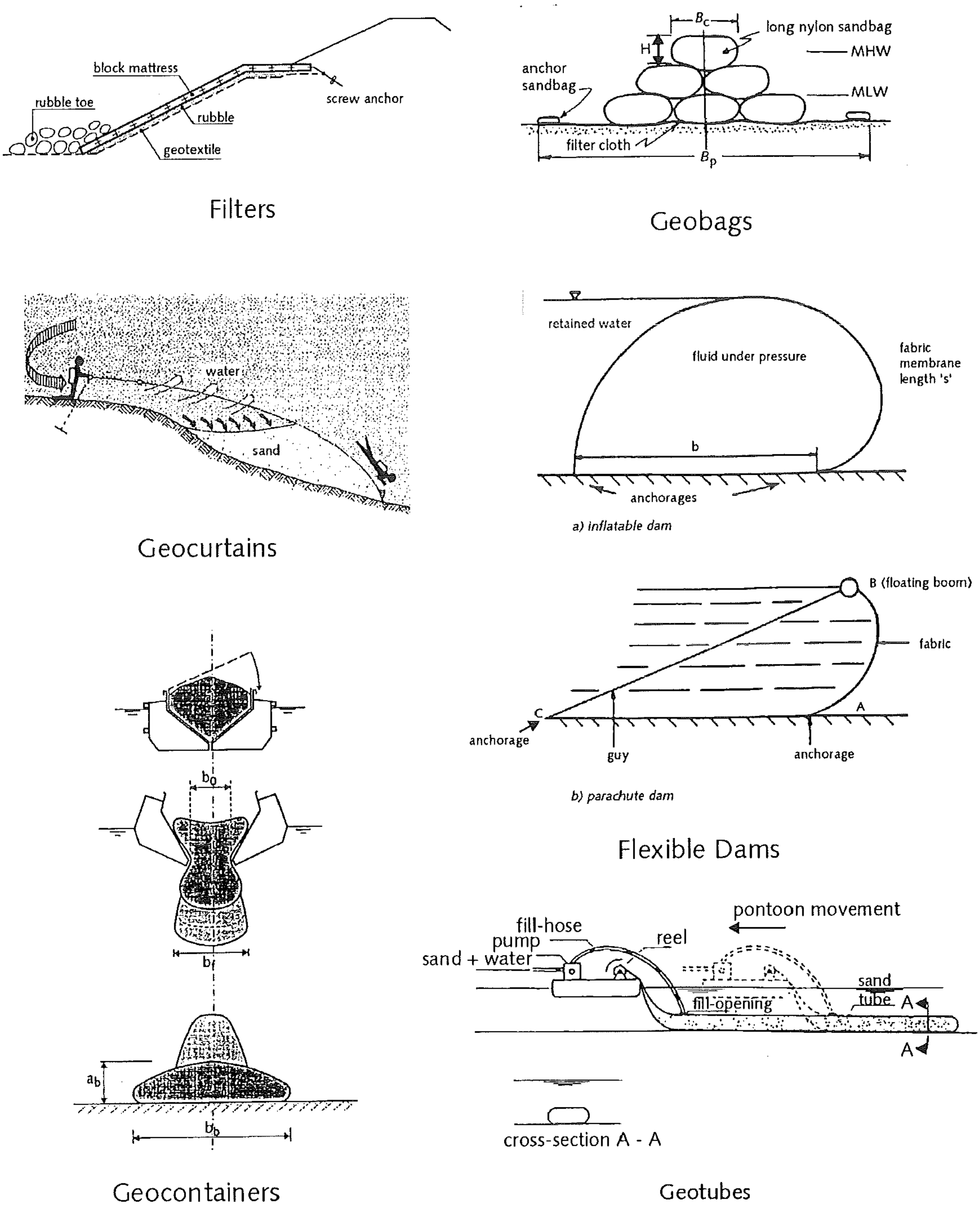


Fig. 12. Some concepts on the application of geotextile systems

These new (geo)systems (geomattresses, geobags, geotubes, seaweed, geocurtains and screens) were applied successfully in number of countries and they deserve to be applied on a larger scale. Recently, geocontainers filled with dredged material have been used in dikes and breakwaters in a number of projects around the world, and their use in this field is growing very fast. Also, a number of new applications for geosynthetic curtains and screens have been developed and tested in practice.

Because of the lower price and easier execution these systems can be a good alternative for protective structures in hydraulic and coastal engineering both in developed and developing countries. The main obstacle in their application, however, is the lack of proper design criteria (in comparison with rock, concrete units, etc.). In the past, the design of these systems was mostly based on rather vague experience than on the general valid calculation methods. More research, especially concerning the large-scale tests and the evaluation of the performance of projects already realised, is still needed. In Pilarczyk (2000) an overview is given of the existing geotextile systems, their design methods (if available), and their applications. Where possible, some comparison with traditional materials and/or systems is presented. The recent research on some of these systems has provided better insight into the design and applications.

TECHNOLOGY TRANSFER AND CAPACITY BUILDING

Know-how/technology transfer is an important expedient in the sustainable development of nations. Technology transfer is sustainable when it is able to deliver an appropriate level of benefits for an extended period of time, after major financial, managerial and technical assistance from an external partner is terminated. Apart from clearly identified objectives for Technology Transfer projects, proper project design and well-managed project execution, essential factors conditioning the survival of projects include: policy environment in recipient institution/country, appropriateness of technology and management organisational capacity (Overbeek et al, 1991).

Technology Transfer means the transfer of knowledge and skills, possibly in combination with available tools, to institutions and individuals, with the ultimate aim to contribute to the sustainable development of the receiving institution (national) or country (international). Professional educational institutes are likely to restrict their work to specialised education and training of individuals. Institutions, however, generally aim at a broader approach. The objective of Technology Transfer should be to reinforce the capability of institutions and individuals to solve their problems independently. The required support can be indicated in a diagram differentiating between the analysis and solution phases of engineering problems. Obviously, knowledge and experience is required on three levels to obtain optimum results:

- knowledge of the processes
- knowledge and experience in the use of process simulation techniques
- experience in practical applications

Knowledge and experience can best be transferred in phases during a project that runs over several years. In many cases, however, budget restrictions call for another approach. The advisor may be called upon as a consultant, and the project includes only some of the phases mentioned above.
Not only knowledge and skills must be transferred to the client's staff, but it must also be integrated in the client's organization for future, independent use. In order to guarantee an efficient interaction between the transfer and the integration activities, distinction in phases is required. One may distinguish the following realization phases:

1. professional education (general scientific/technological level)
2. professional training (new/specific technology/skills/tools)
3. development phase (physical/logistic adjustments at recipient party)
4. institute support (advisory services/exchange visits during project)

The necessary number of phases may vary depending of situation (country, type of project).

Capacity building is important pre-condition for the realization of future challenges and transfer of know-how, especially for developing countries.

Hydraulic and Coastal engineering is a complex art. At this moment a limited number of phenomena can be understood with the help of the laws of physics and fluid mechanics. For the remainder, formulas have been developed with a limited accuracy. In addition, input data are limited availability and form another source of uncertainty. Consequently, a sound engineering approach is required, based on practical experience and supported by physical and numerical models, to increase the understanding of many phenomena and to come up with sustainable solutions. Especially, standard' solutions do not exist in coastal engineering; solutions very much depend on the local circumstances as well as the social and political approach towards coastal engineering. Consequently, the transfer of coastal engineering knowledge is a complex art as well.

Sustainable transfer of hydraulic and coastal engineering technology at post-graduate level should therefore aim at increasing the capacities and skills of the engineers such that they are able to analyse a problem correctly and identify possible directions of solutions. Simply learning formulas and learning standard solutions for standard problems are not fruitful and even dangerous: such training does not increase the engineer's understanding of the underlying processes and serious failures may be the result. Transfer of hydraulic and coastal engineering technology should therefore be problem oriented, practical in nature and geared towards the specific needs of the engineers following the training programme (Verhagen, 1999).

A general trend can be observed from applying rules to more conceptual thinking. Rules will change fast, so it is more important for an engineer to know the design philosophy. Also engineers have to learn where to find the most up-to-date knowledge regarding the design, which he is making at a certain moment. Because the increased growth in science, rules are outdated faster than in the past; this means that in most cases engineers should not apply the rules they have learned in university. So they should be trained always first to verify if the design method they learned in university is still valid.
Engineers should be trained in a flexible application of the design methodologies they have learned. For design purposes the use of complicated computer software is increasing. The packages become more and more user friendly, but the insight in the computational process becomes less. This means that the direct link between output and input is less obvious. Engineers have to become more and more aware of the need of checking the output of these programs in inconsistencies (rather than on numerical accuracy). Because of the high quality of presentation methods of modern software, input inconsistencies are often not recognized in time. Engineers have to be trained to become more and more keen on this problem (Verhagen, 1996).

Transfer of technology to developing countries can also be a complex matter and needs properly educated engineers. Copying solutions from the western, industrial countries for application in developing countries is in general not the best solution for solving the problems of developing countries. The main reason for that is that the available resources in the developing world are different from the resources in western countries. In the industrialized countries there is a strong tendency to solve problems in such a way that the amount of required labor decreases, so a capital-intensive solution is searched for. The reason for this is the very costly social system and the high standard of living. This causes a big difference between the hourly income and the hourly costs of labor. In developing countries this difference is much less, but on the contrary it is difficult and expensive to import industrial products from elsewhere. Also it is difficult to have sufficient capital available. For those countries it is more economic to search for solutions, which require hardly any investments, but are relatively labor intensive. These solutions generally require often more maintenance. However, increased maintenance costs are sometimes very pleasant, provided the initial investment is very low. The cost of the solution can be spread over a longer period without borrowing money for a capital-intensive solution. Maintenance is very important for sustainability of the investment/projects. Maintenance has to be done on the right moment. Also, there has to be a maintaining agency and there has to be a maintenance plan.

In solving these problems, one should always analyse the cause of the problem. Sometimes it is easier to change something in the estuary or river, than to combat the erosion. When it is not possible to take away the cause of the problem, then a number of technical tools are available as discussed in this book. In the design of these methods in most cases a low-investment approach can be followed. 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 materials, 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 realise these points during the design phase.

We may conclude from the above that modern engineer must be educated in various technical and non-technical fields. The task of engineers involved in problems of developing countries should be the adaptation (translation) the actual knowledge to appropriate technologies suitable for their problems and their possibilities.

CONCLUSIONS

Problem identification and understanding is very important for a proper choice of solution to water (flood) management and coastal problems. Generally, it may be concluded that there are both physical as well as social aspects to every problem. As a consequence mere technical solutions often turn out to be mistake. Furthermore, the future use of coast in general should be tailored to fit within the system, whether it is a recreational coast, a wetland or an ecosystem. For sustainable management, land use planning is required which does take floodplains and coastal waters into account. Proper quality of environmental boundary conditions defines the quality of design. In many cases, especially in less developed countries this can be a main obstacle in design of hydraulic and/or coastal structures. Further developments in forecasting and transformation techniques should be stimulated.

There are a large number of hydraulic and coastal structures. For some of them, workable design criteria have been developed in recent years (rubble-mound breakwaters, riprap, block revetments, filter structures, etc.). However, many of these criteria/formulae are still not quite satisfactory, mainly because they are lacking physical background, what makes extrapolation beyond the present range of experience rather risky. To solve this problem, it will be necessary to continue physical model experiments (on scale and in prototype) to develop, validate and calibrate new theories. Moreover, there is still a large number of systems with not adequate design techniques, for example, groins, submerged/reef breakwaters, a number of revetment types (gabions, geomattresses), geosystems, open filter, prediction and measures against scouring, etc. However, opposite to the functional design, the structural design can always be solved by the existing means (design criteria if available or model investigation), assuming availability of funds. Functional design (especially for coastal problems) is one of the most important and most difficult stages in the design process. It defines the effectiveness of the measure (project) in solving specific problem. Unfortunately, there are still many coastal problems where the present functional design methods are rather doubtful, especially concerning shoreline erosion control measures (i.e. groins, sea walls). Also, adequate measures against lee-side erosion (flanking) deserve more attention.

Alternative (waste) materials and geosynthetics and geosystems constitute potential alternatives for more conventional materials and systems. They deserve to be applied on a larger scale. The geosynthetic durability and the long-term behaviour of geosystems belong to the category of overall uncertainties and create a serious obstacle in the wider application of geosynthetics and geosystems and, therefore, are still matters of concern.

The understanding of the coastal responses in respect to the sedimentary coast and its behaviour is at least qualitatively available. However, reliable quantification is still lacking which make functional design of shore erosion control structures very risky. Much mathematical and experimental work is still to be done. Because of scale effects, the experiments will have to be carried out in large facilities or may be verification is even only possible on the basis of prototype observations over a long period. This work is so complicated that international co-operation is almost a prerequisite to achieve success within a reasonable time and cost frame.

Research on hydraulic and coastal structures should benefit from more co-operations among researchers and the associated institutions. Publishing basic information and standardised data would be very useful and helpful in establishing a more general worldwide data bank available, for example, on a website. Systematic (international) monitoring of realised projects (including failure cases) and evaluation of the prototype and laboratory data may provide useful information for verification purposes and further improvement of design methods. It is also the role of the national and international organisations to identify this lack of information and to launch a multiclient studies for extended monitoring and testing programmes, to provide users with an independent assessment of the long-term performance of hydraulic and coastal structures, including alternative materials and systems (geosynthetics, geosystems, alternative/waste materials, etc.).

Inventory, evaluation and dissemination of existing knowledge and future needs, and creating a worldwide accessible data bank are urgent future needs and some actions in this direction should be undertaken by international organisations involved (PIANC, IAHR, UNESCO/WMO, US Coastal Council). It should be recommended to organise periodically (within time span of 5 to 10 years) state-of-the-art reports on various sub-items of hydraulic and coastal engineering, which should be prepared by international experts in a certain field. It can be organised by creating semi-permanent working groups on specific subject and their activities should be paid from a common international fund, which should be established by one of the international organization.

Adjustment of the present education system as a part of capacity building for solving future problems should be recognised as one of the new challenges in hydraulic and coastal engineering. It is also important for the proper technology transfer to developing countries and development and maintaining of appropriate technologies for local use. Moreover, solidarity must be found in sharing knowledge, costs and benefits with less developed countries which are not able to facilitate the future requirements of integrated coastal management by themselves (on its own) including possible effects of climate change. More attention should be paid to integration of technological innovation with institutional reforms, to rise of awareness to change human behaviour, to developing of appropriate technologies that are affordable by poorer countries, to promoting technologies that would fit into small land holdings (local communities), and to capacity building (education), which is needed to continue this process in sustainable way.

Finally, there is a continuous development in the field of hydraulic and coastal engineering, and there is always a certain time gap between new developments (products and design criteria) and publishing them in manuals or professional books. Therefore, it is recommended to follow the professional literature on this subject for updating the present knowledge and/or exchanging new ideas.

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Date	Topic	Notes
01/06/04	Introduction & Overview	Introduction, Types of Structures & the Coastal Environment (updated 1/9/04)
01/08/04	Coastal Processes, Winds and Currents	Coastal Processes, Winds and Currents (updated 1/13/04)
01/13/04	Hurricane Winds, Waves and Design Conditions	Waves, Design Condition and Breakers (updated 1/13/04)
01/15/04	Design Conditions, Wave Predictions & Breakers	Design Conditions Supplement (update 1/15/04)
01/20/04	Wind, Waves and Soil Properties	Wind, Fetch and Waves (updated 1/20/04) Soil Properties (updated 1/21/04)
01/22/04	Scour & Erosion	Structure-Induced Sediment Scour (updated 1/26/04)
01/27/04	Scour & Erosion (continued)	
01/29/04	Soils, Stresses and Consolidation	Consolidation and Settlement in Clay (updated 2/13/04)
02/03/04	Breakwaters	General Description

		Rubble Mound Structure Design
02/05/04	Example of Rubble Mound Breakwater Design	Example of Rubble Mound BW Design
02/10/04	Settlement in Sand & Bearing Capacity	Settlement in Sand & Bearing Capacity (updated 2/16/04)
02/12/04	Example of Rubble Mound Breakwater Design (continued)	Example of Rubble Mound BW Design
02/17/04	Rubble Mound Structure Stability & Composite Structures	Composite Breakwaters (overview) Composite Breakwater Design
02/19/04	Blanket stability in current fields	Blanket stability in current fields
02/24/04	Composite Structures (continued)	
02/26/04	**Review**	
03/02/04	**EXAM 1**	
03/04/04	Revetments, Seawalls, Bulkheads - Bank Protection	Revetments, Seawalls, Bulkheads - Bank Protection USACE EM 1110-2-1614, Design of Revetments, Seawalls and Bulkheads
03/09/04 03/11/04	**SPRING BREAK**	
03/16/04	Revetments, Seawalls, Bulkheads - Forces & Press	Revetments, Seawalls, Bulkheads - Forces & Press
03/18/04	Sheet Pile Walls & Bulkheads	Sheet Pile Walls & Bulkheads USACE EM 1110-2-2504, Design of Sheet Pile Walls Lecture Slides
03/23/04	Revetments, Seawalls, Bulkheads - Sloped Revetments	Revetments, Seawalls, Bulkheads - Sloped Revetments Lecture Slides
03/25/04	Floating/Offshore Structures	Offshore Structures Summary
03/30/04	Guest Lecture: Construction Management	
04/01/04	Offshore structures videos	

04/06/04 Vertical Wall Structures — Vertical Wall Structure Calculations

04/08/04

04/13/04 **DESIGN PROJECT**
04/15/04 **PRESENTATIONS**

04/20/04 **Review**

EOC 6430 Coastal Structures, Handouts

Ocean Waves

Beach Profile & Features

Supplement on Radiation Stress and Wave Set-up

Rubble Mound Stability Calculation Tables (CEM VI-5)

Bedding Layer Design

Quarry Stone Size

Rubble Mound Breakwater Equation Summary

Example of Rubble Mound BW Design

EOC 6430 Coastal Structures, Homework

HW 1, Wind & Waves (solution)

HW 2, Scour (solution)

HW 3, Rubble Mound Breakwater

HW 4, Caisson Breakwater (solution)

Project Specifications (presentations due week of 4/12/04)

wave data (tab delimited)

profile data (tab delimited)

http://www.oas.org/cdcm_train/

Coastal Infrastructure Design, Construction and Maintenance Training

Participating Countries: Antigua & Barbuda, Dominica, Grenada and St. Lucia

This project aimed to reduce the long-term vulnerability of coastal infrastructure in the project countries and in the broader Caribbean region by expanding the capacity for appropriate design, construction and maintenance of coastal infrastructure works, through series of training courses that were designed and implemented under the project. The target audience for the training included public- and private-sector engineers, planners, architects and environmental officials.

[CDCM Project Background] | [CDCM Issues and Recommendations]

Training Courses: Descriptions, Schedules and Materials

The centerpiece of the project was a series of training courses developed and delivered under the project:

- *Course 1: Coastal Zone Management*
- *Course 2: Coastal Defense Systems*
- *Course 3: Monitoring and Maintenance of Coastal Infrastructure*
- *Course 4: Design of Marine Structures*

[CDCM Training Course Page]

Hurricane Hazard Information for Caribbean Coastal Construction

The *Hurricane Hazard Information for Caribbean Coastal Construction* system is designed to provide storm hazard data useful for practicing engineers, architects, and planners in the Caribbean. Technical documents on storm hazards and storm hazard modeling are also available.

[UWI Hazard Data Server Page] | [Mirror Site]

Documents

Needs Assessment: To provide a baseline for planning the training program, a assessment of existing coastal infrastructure design, construction and maintenance resources, expertise and issues in the Eastern Caribbean was conducted November 2000-January 2001.

Deficiencies of the Caribbean Region in Terms of Coastal Design, Construction and Maintenance: A separate review of CDCM deficiencies in the Caribbean and the challenges facing the development and institutionalization of CDCM training in the region was also undertaken. March 2000.

Solutions to Coastal Disasters Conference Paper: A paper on the CDCM Training project was presented by Jan Vermeiren and Raymond Charles at the ASCE's *Solutions to Coastal Disasters 2002* conference.

[CDCM Training Documents Page]

This project is funded by the USAID-Jamaica/Caribbean Regional Program under a special objective for *Hurricane Lenny Recovery in the Eastern Caribbean.* [USAID/J-CAR Special Objective Document]

http://www.oas.org/cdcm_train/

Course 1: Coastal Zone Management
Antigua and Barbuda, 16–22 June 2001, and St. Lucia, 2–6 July 2001. This course introduced the concepts and configuration of coastal zone management and exposed the participants to broad approaches to addressing problems in the coastal zone. The Barbados coastal zone management initiatives were used as a case study to emphasize the organizational, functional and legislative imperatives of coastal zone management. The climatic, hazards, vulnerability and environmental/biological aspects of the coastal zone were introduced. Overviews of coastal processes, beach management and the impact of inland activities on coastal waters were provided. Case studies were used throughout this course.
A total of 38 representatives of the CDCM training project countries (31), the broader OECS (6) and CARICOM (1) participated in these two offerings of the coastal management course and were from the fields of engineering (18), planning (10), fisheries (7) and surveying (3).

Materials

Note: all materials are in Adobe Acrobat .pdf format

- Cover, Table of Contents, Preamble: text (277k)
- Chapter 1 - Elements of Coastal Management: text (325k)
- Chapter 2 - Land Use Planning: text (300k)
- Chapter 3 - Coastal Processes: text (525k) | additional slides (900k)
- Chapter 4 - Coastal Hazards and Vulnerability: text (325k) | slides (1,450k) | Figure 1 (30k) | Figure 2 (25k)
- Chapter 5 - Mitigation Alternatives: text (6,600k)
- Chapter 6 - Beach Management: text (1,050k)
- Chapter 7 - Marine Biology Issues: text (335k)
- Chapter 8 - Climate Change: text (310k) | slides (2,475k) | figures (2,060k)
- Chapter 9 - Integrated Watershed Management: text (555k)
- Chapter 10 - Environmental Impact Assessment: text (315k)
- Chapter 11 - Monitoring and Measurement: text (980k)
- Chapter 12 - Legislative Environment: text (315k)

Course 2.1: Coastal Defense Systems I

St. Lucia, 19–22 July 2001. This course was designed as an introduction to coastal defenses and introduced the participants to coastal elements, coastal processes, hazards and vulnerability and sea defenses. Best practices for coastal monitoring and data collection were presented. The fundamental concepts of failure and design were introduced and the types of available sea defense designs were discussed in detail. A few of the participants had difficulties with some of the deep engineering concepts, such as design wave estimation and design reliability, but generally the course was quite well received. Midway through the course, participants worked in small groups on open-ended design assignments on actual situations in the Caribbean. Participants reported that these exercises were useful and effective in clarifying the course material. This interactive approach was supplemented by a field trip to Soufriere on the last day of the course, which afforded an opportunity to view the coastal elements and processes that had been presented and discussed in the first three days of the training course.

Materials

Note: all materials are in Adobe Acrobat .pdf format

- Cover, table of contents: text (290k)
- Chapter 1 - Coastal hazards and vulnerability: text (320k)
- Chapter 2 - Cross shore sediment process: slides (2,850k)
- Chapter 3 - Introduction to coastal engineering: text and slides (317k)
- Chapter 4 - Coastal processes—tides and tidal flows: text (315k)
- Chapter 5 - Coastal processes—waves: text (390k)
- Chapter 6 - Longshore sediment transport processes: slides (1,760k)
- Chapter 7 - Coastal mapping: slides
    - Introduction (2,065k)
    - Coastal project assessment (6,185k)
    - Nearshore surveying (5,050k)
    - Beach surveying (5,570k)
    - Monitoring and measurements (1,175k)
- Chapter 8 - Sediment budgets and modeling: text (335k)
- Chapter 9 - Alternatives for coastal storm damage mitigation and functional design of coastal structures: slides
    - Alternatives for coastal storm damage mitigation (5,820k)
    - Functional design of coastal structures (2,470k)
    - Structure hydraulic response (1,420k)
- Chapter 10 - Coastal and offshore structures: text (320k)
- Chapter 11 - Structural design: slides (1,870k)

Course 3: Monitoring and Maintenance of Coastal Infrastructure
Grenada, 10–13 September 2001. This course began with an overview of the Coastal Engineering Manual, which is under development by the US Army Corps of Engineers. This was followed by modules on materials in the marine environment, infrastructure monitoring activities and vulnerability assessment. Availability and use of appropriate hazard risk information is critical to successful design and maintenance programs. Under the heading of data acquisition and remote sensing, a range of appropriate models, tools and collection methods were introduced. These included the TAOS storm hazard model, a storm hazard assessment model developed by Charles Watson, the SHOALS data collection system, developed by the US Army Corps of Engineers, and on-site beach monitoring methodologies. The course concepts were reinforced during field visits to three sea defense construction sites along the Western Main Road. The workshop concluded with a presentation of replacement analysis and an interactive discussion period. There were 23 participants in the course.

Materials

Note: all materials are in Adobe Acrobat .pdf format (except for the presentations in Chapters 2 and 4)

- Course title page, table of contents: text (355k)
- Chapter 1 - Materials in a marine environment: title page (260k), slides part 1 (1,250k) and part 2 (550k)
- Chapter 2 - Monitoring of coastal erosion: title page (265k), *Coastal Engineering Manual* presentation (4,800k)
- Chapter 3 - Vulnerability of coastal infrastructure [files too large to post]
- Chapter 4 - TAOS (The Arbiter of Storms) model: text (300k)
    - Presentation on the *Hurricane Hazard Information for Coastal Engineers:* presentation (1,400k)
- Chapter 5 - Data acquisition and remote sensing I [files too large to post]
- Chapter 6 - Data acquisition and remote sensing II: text (450k)
- Chapter 7 - Maintenance management: title page (265k)
    - Structure failure modes: slides (1,030k)
    - Overview of maintenance and monitoring: slides (1,550k)
- Chapter 8 - Replacement analysis: title page (265k)
    - Evaluating coastal structures: condition index system: slides (885k)
    - Repair and rehabilitation of coastal structures: slides (900k)

Course 4: Design of Marine Structures

Trinidad, 24–28 September 2001. The course began with a detailed, design-oriented hazard assessment, with an introduction to approaches and procedures for geotechnical assessments in the coastal zone, highlighted with appropriate case studies from the region. A presentation on beach nourishment design built upon related discussions in previous courses in this series. General marine structure design principles were introduced, followed by practical exercises on the design and protection of rubble structures. Finally, the topic of hazard mapping was introduced, using a coastal hazard mapping project in Belize as a case study. The course also included a full-day field trip along the north, east and west coasts of Trinidad and included visits to pocket and strip beaches, sea walls and causeways. 31 individuals participated in this course.

Materials

Note: all materials are in Adobe Acrobat .pdf format

- Course title and table of contents: text (270k)
- Chapter 1 - Hazard Assessments: slides (340k)
- Chapter 2 - Geotechnical Investigations: text (340k) | slides and figures [too large to post'
- Chapter 3 - Beach Nourishment Design: slides [too large to post]
- Chapter 4 - Rubble Mound Stability: slides (685k)
- Chapter 5 - Foundation Design: slides (655k)
    - Orinco River Drainage: slides (475k)
- Chapter 6 - Design Principles
- Chapter 7 - Coastal Structure Foundations: slides (375k)
- Chapter 8 - Scour and Scour Protection: slides (955k)
- Chapter 9 - Case Studies: slides (1,075k)

http://www.oas.org/cdcm_train/

See also:

http://www.unesco.org/csi/csiinf.htm

Stichting Postacademisch Onderwijs

Cursus "Innovatief denken en doen in kustverdediging"

Dinsdag 19 april 2005 –IK 2– 10:45-11:45 uur

Kustgedrag op grotere tijd- en ruimteschaal

Marcel J.F. Stive, Technische Universiteit Delft

SYLLABUS

Het doorgronden van kustvormen (kustmorfologie of meer algemeen geomorfologie) en dynamisch kustgedrag (kustmorfodynamica) is de basis voor het denken over innovatieve kustverdediging. Ter introductie van de begrippen morfologie en morfodynamica zegt mijn homepage (http://www.waterbouw.tudelft.nl/public/stive/):

Geomorphology and morphodynamics

The systematic study of landforms and their origin is known as *geomorphology*. Within this study of landforms, the Section Hydraulic Engineering is specialized in the study of features of alluvial sediment, which are shaped, by the action of water along coasts, in estuaries and in rivers. This is the field of *coastal*, *estuarine* and *fluvial* morphology, which is covered by the chairs of Prof. Marcel J.F. Stive (focus on coasts and estuaries) and Prof. Huib. J. de Vriend (focus on rivers and estuaries) respectively. The morphological features cover a wide range of scales, from small ripples and larger bars to entire coasts, estuaries and rivers.

Morphodynamics concerns the ongoing evolution of morphological features due to the, generally nonlinear, interaction between these features and the forcing hydrodynamic conditions, together with climatic, geological, biological and anthropogenic conditions.

Sediment transport is the central process in causing morphological change. The effective transports may be directly linked to the magnitudes of the forcing conditions, but they may also be net transports which result from subtle, often small residuals connected to large to-and-fro movements due to, for instance, tidal flow fluctuations, low-frequency flow fluctuations (flow instabilities, wave-group related long-waves) and high-frequency (short-wave) flow fluctuations.

Due to the *nonlinear interactions* and associated feedback, morphodynamic response is often not only directly related to the energetic forcing frequencies (*forced behaviour*), but exhibits also *free behaviour* (apparently or effectively unrelated to the energetic forcing frequencies). The morphodynamic response scales of sedimentary features display such variability that seldom all features can attain an equilibrium state (even in a dynamic sense) in response to the forcing conditions. As a result, the overall morphological system may remain in a continuous transient state.

Reductionistische en holistische benadering

Op kleine tijd- en ruimteschalen is de kennisontwikkeling gericht op de kwantificering van stromingsgeïnduceerde sediment transport. Hierbij worden de processen beschreven op de schaal van de individuele golfcyclus, waarbij de instantane waterbeweging en het geassocieerde sediment transport wordt beschouwd (in deze cursus niet behandeld). Recente ontwikkelingen die wellicht tot een paradigma hebben geleid worden besproken in Stive en Reniers (2003, zie Annex 1 syllabus). Het gaat hierbij om de rol van stroom versnelling in het beschrijven van sediment transport door sterk niet-lineaire golven over zandbanken (wellicht verklaart dit het onvermogen van huidige dwarstransport modellen om kustwaartse beweging van zandbanken te beschrijven).

Op de volgende, hogere of grotere tijd- en ruimteschaal wordt sediment transport gemiddeld over meerdere tot vele individuele golfcycli. Op deze schaal vinden toepassingen plaats ten behoeve van voorspelling als gevolg van interventies als constructies, baggeren en suppleties (cf. bijdrage IK 3 door Edwin Elias en IK 10 door Jelmer Cleveringa). In deze klasse vallen ook recente inspanningen om de drie-dimensionale bodemmorfologie van de brandingszone als functie van de golfcondities op "event-schaal" te hindcasten en hopelijk te voorspellen (cf. Stive en Reniers, 2003).

Bewegen we nog een schaal hoger dan komen we op het modelleren van de bodemevolutie op de schaal van seizoenen, jaren, decaden etc. en de daarmee geassocieerde grootschalige kustvormen die zich slechts langzaam in de tijd ontwikkelen (cf. Delft 3D – MOR). Ondanks het opmerkelijke recente succes van deze reductionistische benadering bestaan inherente beperkingen voor deze klasse van problemen. Praktisch gezien is de rekentijd een beperking, maar dat is wellicht een kwestie van tijd. Er zijn twee fundamentele beperkingen die de voorspellingshorizon kunnen beperken, t.w. accumulatie van modelfouten door opschaling en niet-onderkende feedbacks tussen grootschalige respons en kleinschalige processen.

De holistische benadering is gericht op het direct modelleren op de hogere schaal van interesse. Details van kleinschaliger processen worden genegeerd en fenomenologische relaties worden direct op de schaal van interesse geformuleerd. Een klassiek voorbeeld is de één-lijn theorie, die bijvoorbeeld het voorkomen van grootschalige kustlijn kenmerken als spits en kapen kan verklaren (zie ref Ashton et al., 2001, in Stive en Reniers, 2003). Een tweede voorbeeld is het ASMITA model dat de interactie tussen kust en getijdenbekkens beschrijft (zie Annex 2, Stive, 2004).

Een systematische beschouwing over holistische modellering is beschreven in Annex 3 (Stive, 2003). De fundamentele gedachte is dat hoe groter de beschouwde schaal hoe groter de ruimtelijke interactie tussen de verschillende kustelementen moet worden beschouwd. Immers, dan neemt de schaal waarop sediment in het kustsysteem gedeeld wordt toe.

De impact van klimaatverandering op kustontwikkeling

Voor een introductie moet altijd een keuze gemaakt worden uit het brede spectrum van invalshoeken. Gekozen is voor het beschouwen van kustontwikkeling en de interactie tussen deze ontwikkeling en die van zeegaten in het licht van klimaatverandering. Hierbij is gepoogd een tamelijk generiek beeld te geven dat een brede toepassing kent.

Annexen

1. Stive en Reniers, 2003. Sandbars in motion, Science.
2. Stive, 2004. How important is global warming for coastal erosion? Climatic Change, **64**: 27-39.
3. Stive, 2003. Advances in morphodynamics of coasts and lagoons. Keynote International Conference on Coasts and Estuaries, Hangzou, November 2003.

Websites

http://www.waterbouw.tudelft.nl/
http://www.water.tudelft.nl

Hydrodynamics and Morphodynamics of Texel Inlet

Edwin Elias, Delft University of Technology

Chapter 1
INTRODUCTION

Although, morphodynamic process-based models have been around over the last decades, their application in tidal inlet systems remains fairly limited. One of the main reasons might be that tidal inlets are extremely complex systems with a variety of processes and mechanisms that are not fully understood. Morphodynamic studies tended to focus on longer-term simulations werein complexity was reduced by forcing the model with representative steady boundary conditions rather than randomly varying input conditions (De Vriend and Ribberink, 1996; Cayocca, 2001). These studies used models that typically consisted of a number of coupled modules that compute waves, currents, sediment transport and bed-level changes sequentially (see Fig. 1.1, left). By applying larger time-steps for the computation of bed-level update then for the computation of flow and sediment transports morphodynamic simulations over larger time spans could be made.

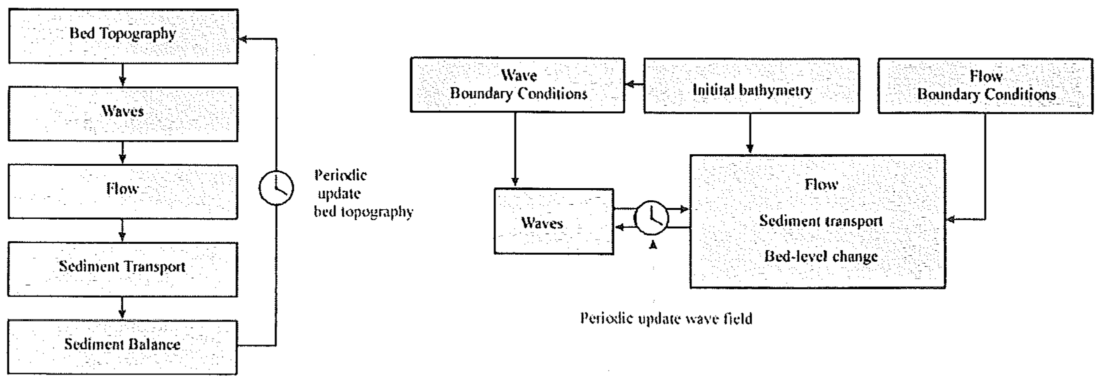

Figure 1.1: Schematic representation of the Delft3D-MOR model (left) and Delft3D Online Morphology model (right)

Recent advances in process-based modelling include the computation of sediment transport and bed level change fully integrated in the flow module; see right panel of Fig. 1.1 (Lesser *et al.*, 2004). Morphologic changes are now calculated simultaneously with the flow calculations. One of the major assets of this type of model is the capability to increase the spatial and temporal resolution of point-oriented field observations. Point-oriented observations are used to force the model as-realistically-as-possible (quasi real-time by measured time-series of wind, waves and tides) and the model results provides synoptic, more-or-less realistic data of high spatial and temporal resolution over the inlet domain. Analysis of this data can provide valuable information on governing flow and sediment transport patterns in the instrumented and the un-instrumented areas of the domain. Such short-term process-based model simulations provide a valuable tool in identification of the dominant flow- and sand transport patterns, and model data can be used to obtain fundamental understanding of the mechanisms responsible.

This syllabus presents an application of Delft3D Online Morphology for Texel Inlet. Quasi real-time model results obtained from Delft3D Online Morphology are used to investigate several aspects of the evolution of the Texel Inlet, and the underlying processes and mechanisms in detail. The research is presented in two chapters based on papers submitted to peer-reviewed international journals; Chapter 2, *"Mechanisms of sand exchange between ebb-tidal delta and inlet due to human intervention"* (Elias *et al.* 2005c), and Chapter 3, *"Field and model data analysis of sand transport patterns in Texel Tidal Inlet"* (Elias *et al.* 2005a). Each of the chapters can be read independently.

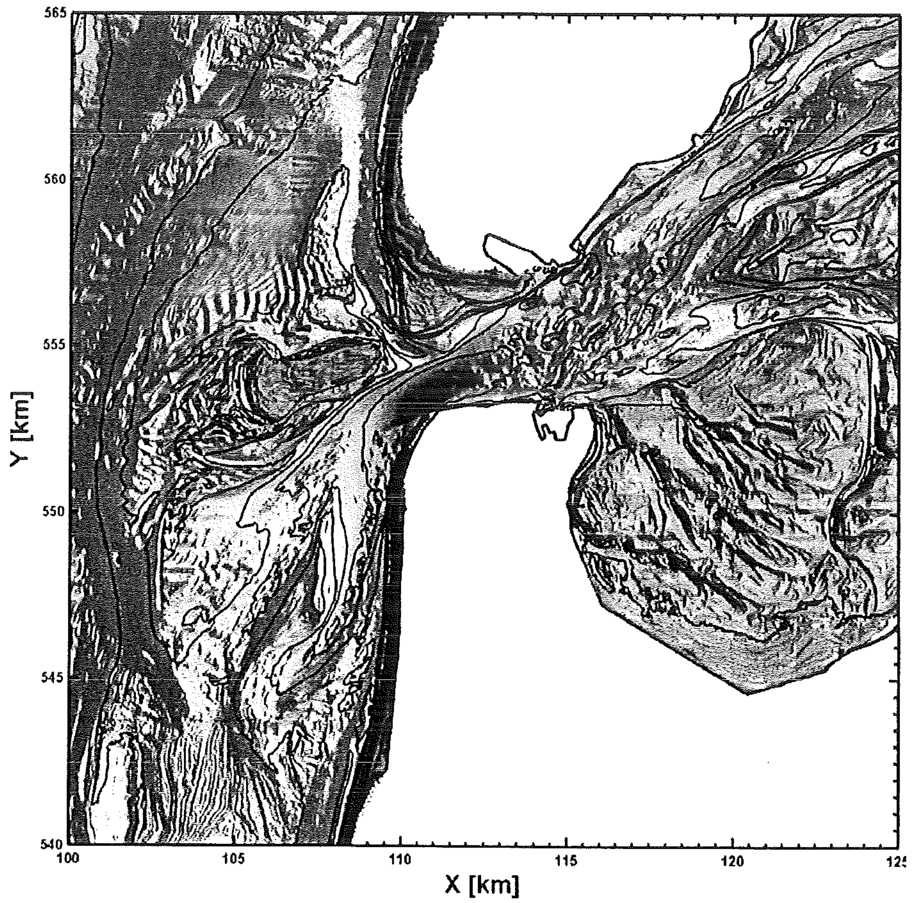

Figure 1.2: Impression of the channels and shoals of Texel inlet (1997 bathymetry).

Chapter 2 addresses the impact of Closure of the Zuiderzee on the ebb-tidal delta development of Texel Inlet. One of the eye-catching characteristics of the present-day ebb-delta bathymetry is the distinct asymmetrical shape of the ebb-tidal delta (see Fig. 1.2). The main channels are distinctively updrift orientated causing erosion problems along the North-Holland coast. It is clear that the Closure of the Zuider Sea in 1932 has had a major impact in the development of Texel Inlet, the ebb-tidal delta and the adjacent coastlines. In this part the effects of the closure on the inlet dynamics are summarized, based on the re-analysis of field observations as presented in Elias *et al.* (2003; 2005b). These authors postulated a conceptual model that describes the morphologic adjustment of tidal inlets due to large-scale human intervention. The kernel of this model is that the morphological adjustment towards a new overall equilibrium is split into two stages related to the existence of more then one temporal response scale. The Delft3D Online Morphology model has been used to generate synoptic data of high spatial and temporal resolution over the inlet domain during various stages of the morphological adjustment process. Model data were used to validate the observation-based conceptual ideas on inlet evolution.

Part 2 describes the flow and sand transport patterns in Texel Inlet based on an integrated approach of field- and model-data analysis. Field data on water levels, flow, sedimentation and erosion patterns, bathymetric features and bed-forms are analyzed and conceptual descriptions of the dominant sand transport patterns on the ebb-tidal delta and the interaction with the adjacent coastlines are given. Additionally, the field data is used to validate and calibrate a Delft3D Online Morphology model. It is shown that the Delft3D Online Morphology model is capable of quasi real-time simulation of the dominant flow and transport patterns over a 3-month period on the scale of the inlet. The high-resolution numerical model results prove to be a valuable tool in identifying the main transport patterns and mechanisms in the inlet domain. Qualitative transport patterns in Texel Inlet and its associated ebb-tidal delta are derived by integration of the high-resolution observations and model results.

Chapter 2
IMPACT OF BACK-BARRIER CHANGES ON EBB-TIDAL EVOLUTION

E.P.L. Elias, M.J.F. Stive, J.A. Roelvink

Abstract:

Texel inlet, the largest inlet in the Dutch Wadden Sea, has undergone drastic changes in the morphology of basin, ebb-tidal delta and adjacent coastlines after closure of a major part of its back-barrier basin. Despite intensive monitoring and analysis, present observation-based conceptual models lack the subtle physics necessary to explain the observed transient developments, such as channel relocation on the ebb-tidal delta and large sediment import into the basin. Fundamental understanding of the inlet dynamics and evolution is obtained by integrating field- and model-data analysis. The state-of-the-art process-based model Delft3D Online Morphology has been used to generate synoptic data of high spatial and temporal resolution over the inlet domain during various stages of the morphological adaptation process. Model data were used to validate the observation-based conceptual ideas on inlet evolution. Model results indicate that the back-barrier changes have triggered the observed ebb-tidal delta developments. Sediment import is explained to result from spatially uneven sediment availability due to (1) an imbalance between flow and bed-topography on the ebb-tidal delta and a balance in the upper part of the basin after closure, and (2) due to higher contributions of wind and waves on the ebb-tidal delta than in the basin. With the ongoing morphological adaptation of the ebb-tidal delta the imbalance between flow and bathymetry and the sediment import into the basin diminishes.

2.1 INTRODUCTION

Tidal inlets occur along a major part of the world's coastlines. Due to the complexity of physics involved, the interaction between water motion (tide, wind and wave-driven), sediment transports and the (highly variable) channel and shoal structures on different scales of aggregation (De Vriend, 1991), inlet dynamics are notoriously complex.

Traditionally, inlet research has focused on observation-data analysis due to the large spatial- and temporal scales involved (tens to hundreds of kilometres in space and time scales of centuries). Analysis of field data has resulted in a range of conceptual models and empirical relations to explain the variety in size, volume, and the gross-scale distribution of channels and shoals in the inlet system (O'brien, 1931; Escoffier, 1940; O'brien, 1969; Hayes, 1975; Hine, 1975; Walton and Adams, 1976; Fitzgerald, 1988; Hicks and T.M., 1996; Kana *et al.*, 1999). These studies have significantly contributed to an improved understanding of inlet behaviour and evolution, but lack the subtle physics necessary to explain observed transient developments, e.g. when inlets are impacted by large-scale human intervention (Elias *et al.*, 2003; Elias *et al.*, 2005a).

Texel inlet, the largest inlet in the Dutch Wadden Sea, has undergone drastic changes in the morphology of basin, ebb-tidal delta and adjacent coastlines after closure of a major part of its back-barrier basin (closure of the Zuiderzee in 1932). Intensive monitoring of the inlet by Rijkswaterstaat (Ministry of Transport, Public Works and Water Management) has resulted in the availability of long-term datasets of water levels, wind, waves, currents and discharges, bathymetry, bedforms and sediments. These observations provide a unique opportunity to study intervention induced inlet evolution. Observation-based conceptual models and hypotheses on the governing processes and mechanisms for the ebb-tidal delta alterations and the sediment exchange between basin and ebb-tidal delta were proposed (Sha, 1986a, b; Elias *et al.*, 2005a). Elias *et al.* (2003) recognized that the closure-induced effects on the ebb-tidal delta (channel relocation) and in the basin (sedimentation) have appeared in a time span of approximately 40 years after closure. Nevertheless, basic questions such as how the sand exchanges between inlet, ebb-tidal delta and adjacent coast and which processes determined this exchange are still not satisfactory understood.

In this paper we aim to expand insight into the main mechanisms responsible for the large changes in ebb-tidal delta and basin morphology after closure. We use process-based model results to validate the existing observation-based conceptual models and hypotheses. The paper starts with a short description of the physical setting of Texel Inlet, and the morphodynamic changes in basin and ebb-tidal delta since closure. Recent insights and conceptual model descriptions of the inlet behaviour are summarized. These observation-based ideas on post-closure inlet evolution lack comprehensive descriptions of the underlying physics. Elias *et al.* (2004; 2005a) have proven that short-term process-based model simulations provide a valuable tool in identification of the dominant flow- and sand transport patterns, and model data can be used to obtain fundamental understanding of the mechanisms responsible. In the following sections we present the numerical model, the model method, simulations and results. In the discussion we present a hypothesis on the main mechanisms for sediment exchange between basin and ebb-tidal delta (including the adjacent coastlines). We conclude by generalizing our findings which contributes to the understanding of tidal inlet behaviour under human intervention.

2.2 TEXEL INLET

2.2.1 Physical Setting

Texel Inlet is the largest tidal inlet of the Dutch Wadden Sea and is located in the north-western part of the Netherlands between Den Helder and the barrier island of Texel (Fig. 2.1A). With bathymetric data of the inlet and ebb-tidal delta being available since the 16^{th} century, the inlet is probably the longest regularly monitored inlet worldwide.

Fig. 2.1B shows the present-day geometry and bathymetry of the inlet including its back-barrier drainage area. The inlet gorge is formed by Marsdiep that is about 2.5-km wide with a maximum depth of 53 m. Marsdiep forms the connection between the main channel in the basin, Texelstroom, and the main-ebb channels, Schulpengat and Nieuwe Schulpengat, on the ebb-tidal delta. The ebb-tidal delta protrudes approximately 10 km seaward and 25 km alongshore, determining the nearshore bathymetry of the adjacent North-Holland coast in the south and the Texel Island coast in the north. The ebb-tidal delta contains a large supra-tidal shoal area (Noorderhaaks) facing Marsdiep. On the southern part of the ebb-tidal delta, along the North-Holland coastline, the two main channels (Schulpengat and Nieuwe Schulpengat) are updrift orientated, with respect to the direction of tidal propagation and littoral drift. Along Texel island a smaller channel (Molengat) separates the coastline and the Noorderhaaks shoal.

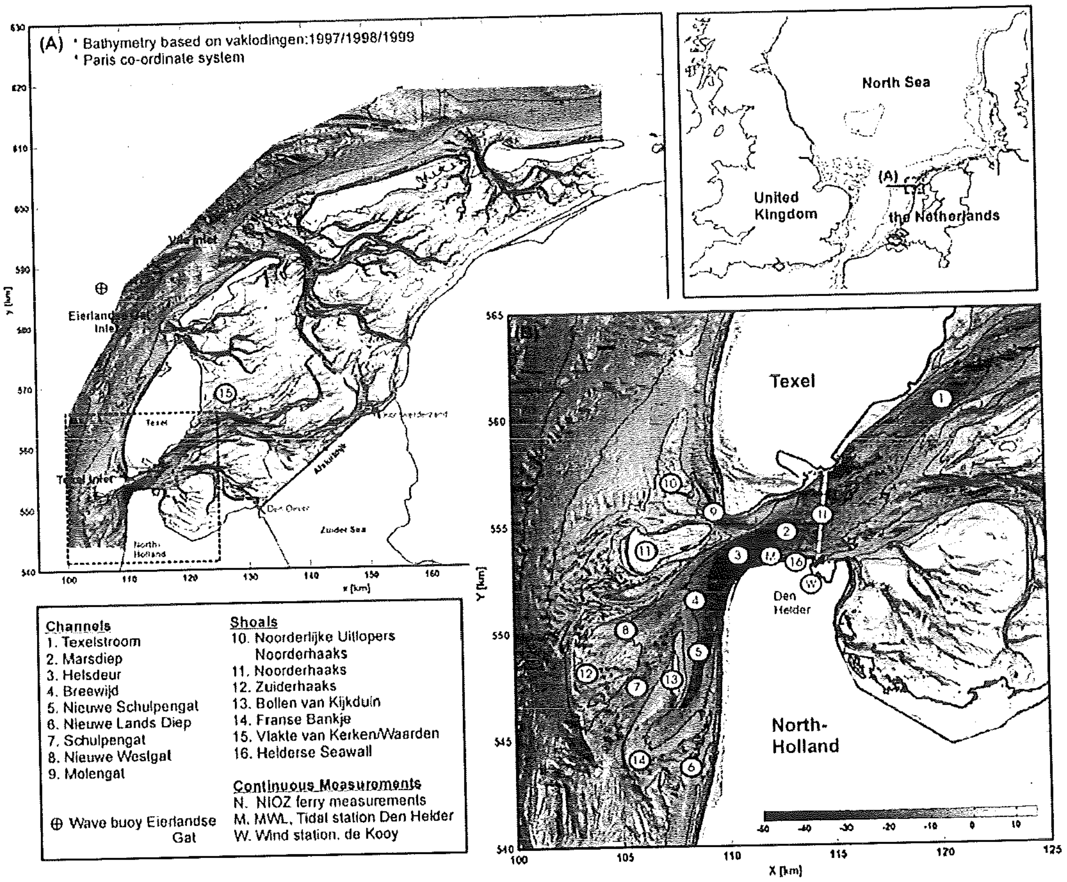

Figure 2.1: Location map of (A) the Western part of the Dutch Wadden sea and (B) Texel Inlet in detail (co-ordinates are based on the Paris co-ordinate system).

At Texel Inlet tides are the main driving force behind the horizontal water flow, with the semi-diurnal M_2 constituent being the dominant component. The tide has a mean tidal range of 1.38 m in Den Helder, increasing to 2.0 m during spring tide, while during neap tide it drops to about 1.0 m. Recent measurements (Ridderinkhof *et al.*, 2002) show an average tidal prism through the inlet of 1×10^9 m^3, with maximum ebb and flood tidal velocities ranging between 1.0 and 2.0 m/s. The wave height distribution at the ebb-tidal delta is dominated by wind-generated waves in the North Sea basin with only a minor contribution of swell. The mean significant wave height is 1.3 m from the west-southwest, with a corresponding mean wave period of 5 seconds.

Following the classification of Hayes (1979) the inlet qualifies as mixed-energy wave-dominated, even under spring tide conditions. However, the morphology of the inlet shows tide-dominated characteristics such as a large ebb-tidal delta. This is caused by the large tidal prism and the relatively low wave energy (Davis and Hayes, 1984).

2.2.2 Closure of the Zuiderzee

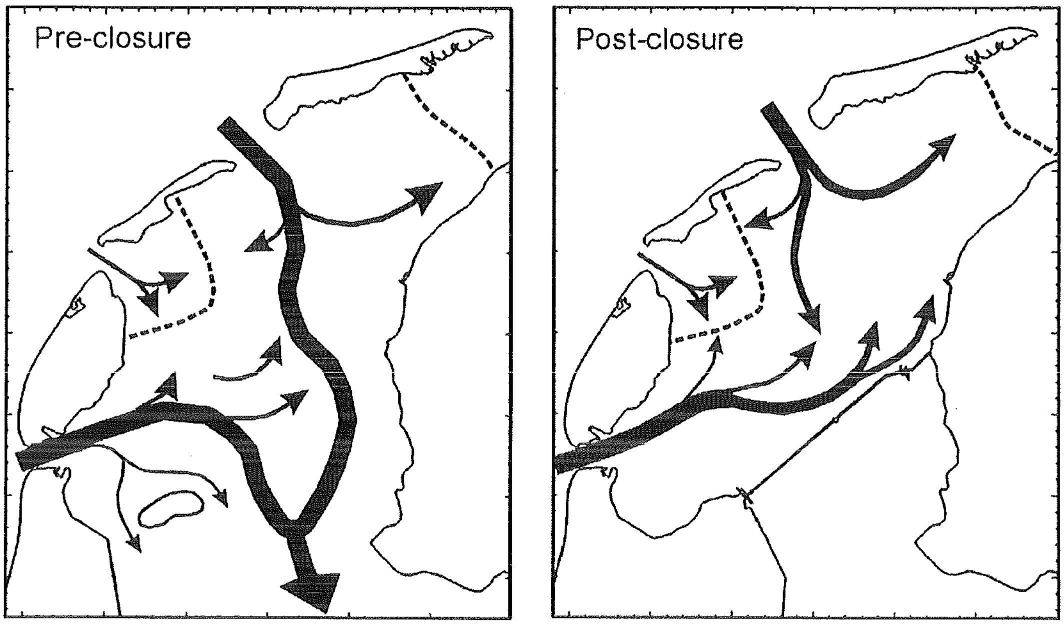

Figure 2.2: Impression of the tidal wave propagation prior and after closure (redrawn after Thijsse, 1972).

The closure of the Zuiderzee (1925-1932) is the largest single intervention ever constructed in the Wadden Sea. Preceding the closure the Texel and Vlie tidal basins covered the south-western part of the Wadden Sea and the former Zuiderzee (Fig. 2.2, left panel). In total, the basin covered a surface area over 4000 km^2 with a basin length of 130 km. The major part of the former Zuiderzee coast was protected by dykes and during storm surges breaching occurred frequently, flooding large areas. The closure dam ensured safety by reducing the Zuiderzee coastline from approximately 250 km of dykes to 30 km of dam. Additionally, land reclamation in the enclosed basin would be simplified and the basin formed a large fresh-water reservoir that reduces salt intrusion into the hinterland.

After the closure, the basin area reduced substantially to an area of roughly 712 km^2 (Fig. 2.2, right panel) and a length of about 30 km. The closure considerably altered the hydrodynamics and morphodynamics in the remaining active part of the basin (Rietveld, 1962; Elias *et al.*, 2003). The tidal characteristics changed from a propagating to a standing tidal wave resulting in an increase of the tidal range and tidal prism through Marsdiep by approximately 26%. With these large changes in hydrodynamics and particularly in basin geometry, pronounced changes in morphology of Texel inlet have taken place. The closure separated the back part of the basin (Zuiderzee) that contained a relative large portion of shoals. In the remaining active part of the basin the remaining shoals area was too small compared to the channel area, therefore, a morphologic adjustment of the basin was to be expected (Eysink, 1990). Large sedimentation was observed, ~200 million (M) m^3 of sediment, during a period of approximately 40 years (Fig. 2.3, left panel).

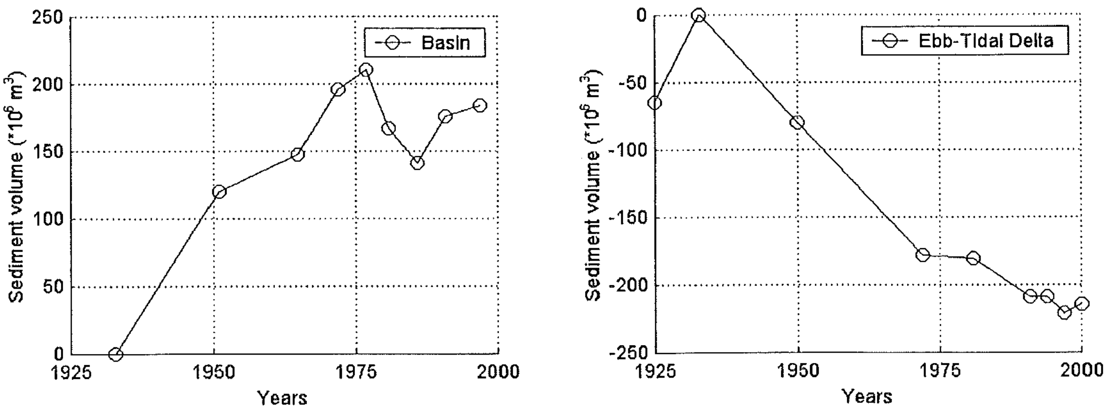

Figure 2.3: Sediment-volume change in the basin (left) and on the ebb-tidal delta including the adjacent coasts (right) with respect to 1933 volume (Elias *et al.*, 2003).

The two basins, Texel and Vlie, are now more or less separated by a tidal divide. A troughflow from Vlie to Texel inlet remains, although residual transports between the two basins are minor (Ridderinkhof, 1988). The changes in Vlie inlet were small compared to the large changes in Texel inlet.

2.2.3 Post-closure ebb-tidal delta evolution (1933 - 2003)

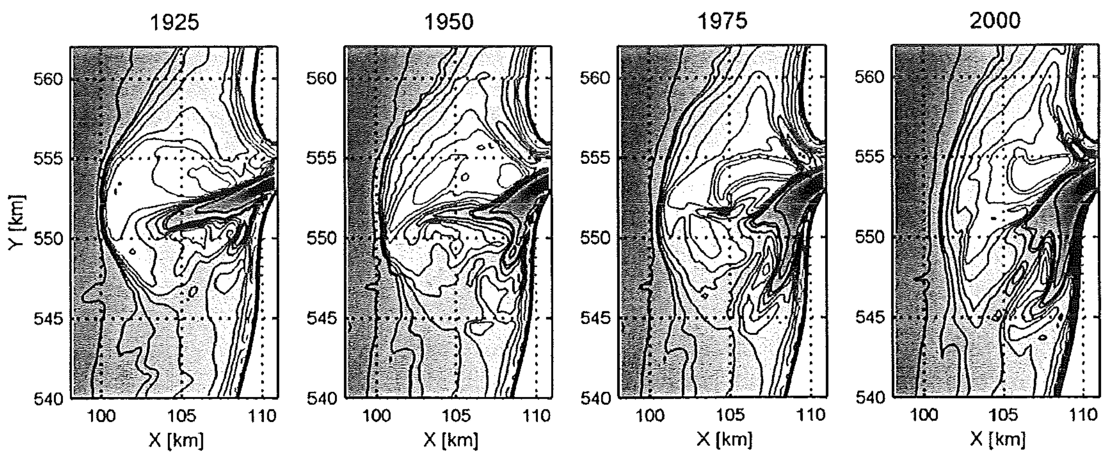

Figure 2.4: Ebb-tidal delta bathymetry of the Texel Inlet for the years 1925, 1950, 1975 and 2000 (complete maps are compiled by filling in missing data with the nearest measurements available).

ELIAS *et al.* (2005b) describe the pre-closure arrangement of channels and shoals as characteristic for a stable inlet system (Fitzgerald, 1988). The main-ebb-channel (Westgat) protruded seaward, with marginal flood channels along the adjacent shorelines on both sides. In front of, but mainly north of Westgat, the main shoal area (Noorderhaaks) is formed as an area where transports due to ebb-tidal currents and incident wave-generated currents balance (see Fig. 2.4, 1925). This relative stable ebb-tidal delta state was distorted by the closure. The morphological response time of the inlet system to the effects of the closure, the present state of the system (equilibrium or non-equilibrium, sedimentation or erosion) and the main mechanism for sediment transport through the inlet have long been a matter of debate among Dutch coastal scientists and practioneers. A re-analysis of observations, complemented by state-of-the-art tidal inlet theory, resulted in a major improvement of the insights into the dynamics of Texel Inlet (Elias *et al.*, 2003). These authors recognized that the ebb-tidal delta behaviour over the period 1932-2002 was a forced response to the effects of human intervention. The changes in hydrodynamics and the sediment demand of the basin contributed to a changed morphologic forcing by the basin on the ebb-tidal delta (Elias *et al.*, 2003; Elias *et al.*, 2005b). The temporal development of the distorted inlet system was described by a conceptual model comprising four stages (Fig. 2.5).

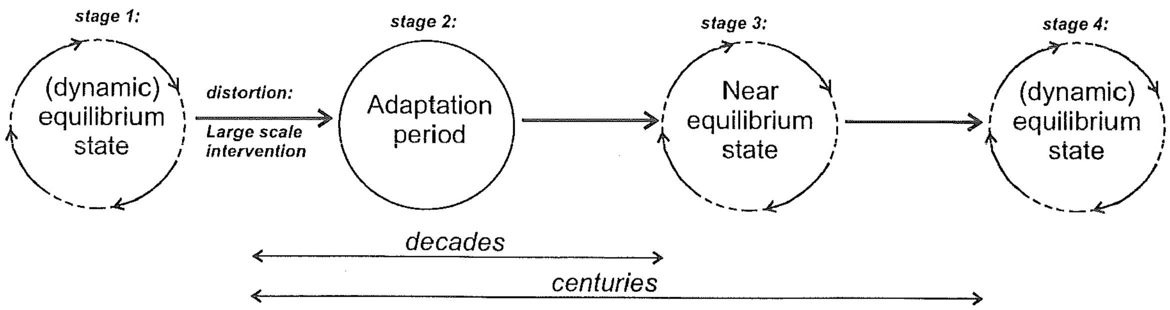

Figure 2.5: Conceptual model for Wadden Sea inlets impacted by large-scale intervention (Elias *et al.*, 2003).

The essence of the conceptual model is the distinction of four stages of development, acknowledging the likely existence of different morphological time-scales associated with the closure (Kragtwijk *et al.*, 2004). The first and the last stage concern the equilibrium behaviour of the natural inlet system preceding the human intervention, and its possible equilibrium behaviour in the future. In-between, two stages of adjustment can be distinguished that, respectively, describe the individual adjustment of the morphological units ebb-tidal delta, basin and adjacent coastlines over a period of approximately 40 years caused by the intervention, and a subsequent period of large-scale adaptation of the entire inlet system through sediment exchanges between the morphological units.

Elias *et al.* (2003; 2005b) conclude that during the adaptation stage following the engineering works the ebb-tidal delta dynamics cannot be described in terms of the natural hydrodynamic processes, viz. the ratio of wave versus tidal energy alone. These author's hypothesized that the ebb-tidal delta evolution after closure was triggered by the changed hydrodynamics in the basin. The changed tidal characteristics (Van Veen, 1950; Sha, 1989), the northward displacement of the basin centre, the closing of southward directed channels and the amplified tidal prism increased flow through Texelstroom (Elias *et al.*, 2003). Texelstroom determines the ebb outflow into the inlet gorge that re-orientated resulting in a more southward-directed outflow onto the ebb-tidal delta. The expression back-barrier steering was introduced to describe this 'forcing' induced by the basin on the ebb-tidal delta (Elias *et al.*, 2005b). The altered ebb outflow triggered the development of the former flood-dominated Schulpengat as main ebb channel, while the original westward-oriented main ebb channel Westgat filled in and disappeared (Fig. 2.4, 1925-1950). Initially, Schulpengat developed as a single channel but since 1956 two separate channels (Schulpengat and Nieuwe Schulpengat) formed in an updrift direction aligned along the North-Holland coastline (Fig. 2.1d, 1950-1975).

The recent developments of the ebb-tidal delta (1975-2000) have been governed by a redistribution of sediment (Elias *et al.*, 2005a). The shallow sub-tidal part of Noorderhaaks migrated landward and extended northward, forming a large cuspate spit extending along the Texel coastline (Noorderlijke Uitlopers of Noorderhaaks). The northern part of the ebb-tidal delta is dominated by this spit development. The area south of the Noorderhaaks remains dominated by the main tidal channels along the North-Holland coastline (e.g. Nieuwe Schulpengat, Schulpengat and Nieuwe Westgat). The Nieuwe Schulpengat continued to extent in updrift direction along the North-Holland coastline, reaching a maximum channel length in 1990. In front of the channel, sediment settled due to segregation of flow and the ebb-shield was formed. The shoal Bollen van Kijkduin separates Nieuwe Schulpengat from the seaward located Schulpengat. Along the southern margin of Noorderhaaks, there are indications for the formation of a new westward-oriented channel (Nieuwe Westgat). Nieuwe Westgat and Schulpengat contribute sediments to the southward outbuilding of Zuiderhaaks.

The re-orientation of the main channels on has had large consequences for the coastal maintenance of the adjacent coasts. Over 200 Mm^3 of sand was eroded from the ebb-tidal delta and coastlines (see Fig. 2.3b). Both, the Texel as the North-Holland coastline are still subject to structural erosion (Cleveringa, 2001; Elias and Cleveringa, 2003; Elias *et al.*, 2003) and maintenance of these stretches of coast belongs to the most intensive of the entire Dutch coastal system (Mulder, 2000; Roelse, 2002).

2.2.4 Main mechanisms for sediment import

The corresponding rates of sedimentation in the basin (Fig. 2.3a) and erosion of the adjacent ebb-tidal delta and coasts (Fig. 2.3b) point to sand exchange between these elements (Van Marion, 1999). Hence, sediment import into the basin through Marsdiep must have dominated after closure. The governing mechanisms for these transports have long been a matter of debate. Studies, e.g. Sha (1989), Dronkers (1998)(1998), Louters and Gerritsen (1994), Ligtenberg (1998) and Elias *et al.* (2003), conclude to a net import of sediment in the order of 1 to 3 Mm^3/year. Ligtenberg (1998) suggested that after closure tidal asymmetry induced large sediment import into the basin. As the morphology of the basin adapted to the effects of closure, tidal asymmetry and the related sediment import into the basin reduced. In the present state only a small sediment import remains. Based on analysis of recent flow observations supplemented with results of a process-based model Bonekamp *et al.* (2002) conclude to sediment export for the present situation as the ebb-dominant tidal residual transport exceeds the flood-dominated tidal asymmetry driven transport.

Sediment transports due to the main tidal asymmetries have long been assumed to be most dominant, but the recent study of Elias *et al.* (2005a) shows that the non-tidal mechanisms (e.g. wind and waves) can also contribute significantly to the sand transports in the inlet gorge; it is a likely possibility that in the present situation sediment import into the basin results primary from the interaction between tides, wind and waves.

Preliminary results from ongoing research into the sediment transport due to estuarine circulation show that density-driven flow might be a third factor for augmented sediment import during periods of major fresh-water discharge into the basin through the discharge sluices at Den Oever and Kornwerderzand.

2.3 PROCESS-BASED MODEL SIMULATIONS

2.3.1 Introduction

Fundamental understanding of inlet dynamics can nowadays be obtained by process-based models. These models, describing the water motion, sediment transport and bottom change by a series of mathematical formulations, have been applied in the modelling of coastal processes since the early days of computer technology. Recent studies (e.g. Wang *et al.*, 1995; De Vriend and Ribberink, 1996; Cayocca, 2001; Hibma *et al.*, 2003; Elias *et al.*, 2004; Elias *et al.*, 2005a) have shown that such models can be used successfully to study morphological evolution in the complex environments of tidal inlet systems.

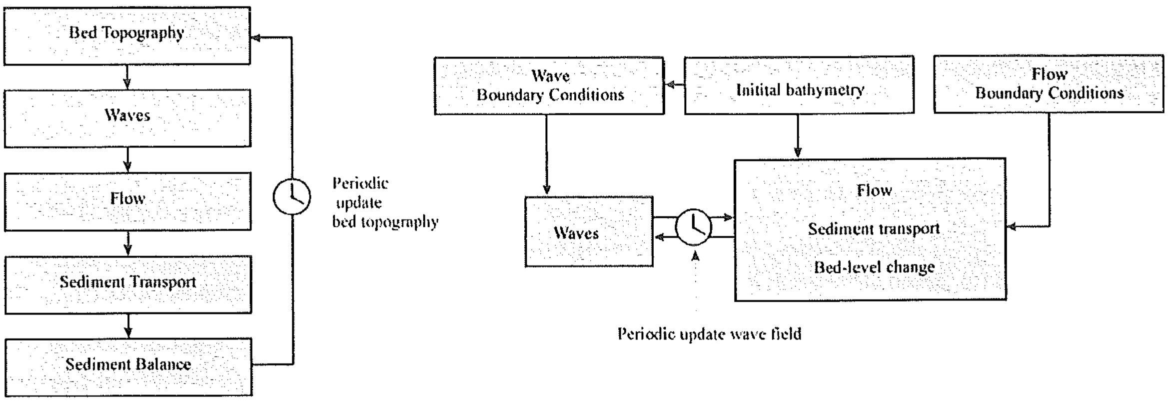

Figure 2.6: Schematic representation of the Delft3D-MOR model (left) and Delft3D Online Morphology model (right)

The studies of Wang *et al.* (1995), Cayocca (2001), De Vriend and Ribberink (1996) and Hibma et al. (2003) used a more-or-less 'traditional' approach. The model consists of a number of coupled modules that compute waves, currents, sediment transport and bed-level changes sequentially (see Fig. 2.6a). By applying larger time-steps for the computation of bed-level update then for the computation of flow and sediment transports morphodynamic simulations over larger time spans could be made. As the frequency of bed-level update is typically in the order of (several) tides to days these models were forced by representative steady boundary conditions rather than randomly varying input conditions (Latteux, 1995; De Vriend and Ribberink, 1996; Cayocca, 2001). The construction of representative conditions, but also the correct coupling of the various modules made such model simulations complicated and results sometimes unpredictable. This might be one of the reasons why still only a limited number of process-based model applications have been conducted for complex areas such as tidal inlet systems.

Recently, the computation of sediment transport and bed level change has been fully integrated in the flow module; Delft3D Online Morphology model (Lesser *et al.*, 2004). Fig. 2.6b illustrates the model principle. As the morphologic changes are calculated simultaneously with the flow calculations one of the major assets of this type of models is the capability to increase the spatial and temporal resolution of point-oriented field observations. Point-oriented observations are used to force the model as-realistically-as-possible (quasi real-time by measured time-series of wind, waves and tides) and the model results provides synoptic 'field'-data of high spatial and temporal resolution over the inlet domain (Elias *et al.*, 2005a). Analysis of this data can provide valuable information on governing flow and sediment transport patterns in the instrumented and the uninstrumented areas of the domain. Additionally, such models allow to perform sensitivity analyses for identification of the dominant processes and mechanisms by varying the forcing conditions (Elias *et al.*, 2004). In this study the effect of tides is investigated in detail.

Our ultimate goal of quasi real-time simulations is to obtain estimates of the long term (years to decades) transport patterns and magnitudes. As yet, feasible computational effort limits us to the simulation of shorter periods (Elias *et al.*, 2004; Elias *et al.*, 2005a) wherein morphologic change is negligible compared to the morphodynamic scale of the inlet. However, by making simulations on the 1933, 1950, 1975 and 2000 bathymetries synoptic data over the inlet domain is obtained during various stages of the inlet development. By studying the residual and instantaneous flow and transport patterns for each of the simulations, insight is obtained in the (alterations in) dominant processes and mechanisms for sand exchange between inlet, ebb-tidal delta and coast.

2.3.2 Delft3D Online Morphology Model

The Delft3D Online Morphology model is used for depth-averaged simulations of the water motion and sediment transports in the inlet domain. A short description of the model basics is presented; testing and validation of the model is discussed in detail by Lesser *et al.* (2004). The Delft3D-FLOW module forms the centre of the model system simulating water motion due to tidal and meteorological forcing by solving the well-known unsteady shallow-water equations (Stelling, 1984). The Alternating Direction Implicit method is used to solve the vertical and horizontal momentum and continuity equations (Stelling, 1984; Leendertse, 1987) on a staggered grid.

Wave effects, such as enhanced bed shear stresses and wave forcing due to breaking, are integrated in the flow simulation by running the 3rd generation SWAN wave processor (Version 40.11-ABCDEF). The SWAN-model is based on discrete spectral action balance equations, computing the evolution of random, short-crested waves (Holthuijsen *et al.*, 1993; Booij *et al.*, 1999; Ris, 1999). Physical processes included are: generation of waves by wind, the dissipation due to whitecapping, bottom friction and depth induced breaking, and, non-linear quadruplet and triad wave-wave interactions. Wave propagation, growth and decay are solved on a subset of the flow grid. The results of the wave simulation, such as wave height, peak spectral period, and mass fluxes are stored on the computational flow grid and included in the flow calculations.

The Online Morphology version supplements the flow model results with sediment transport computations (Fig. 2.6b). Each computational time-step sediment transports of non-cohesive sediment are computed within the flow simulation based on the transport formulations of Van Rijn (1993; 2000; 2002). Herein, the total sediment transport is obtained from the sum of the bed load and suspended load transports. Bed load transports represent the particle movements in the wave boundary layer due to currents and waves. Suspended load transports are the transports above the bed layer due to e.g. transport of sediment by high-frequency oscillating flow (cross-shore orbital motion and advective transports by the time-averaged current velocities including wave effects such as wave motion induced reduction of current velocities near the bed, and, enhancement of near-bed concentrations due to wave stirring). Before bed-level updating sediment transport rates are corrected for bed-slope effects.

2.3.3 The Texel Outer Delta Model Application

Model domain

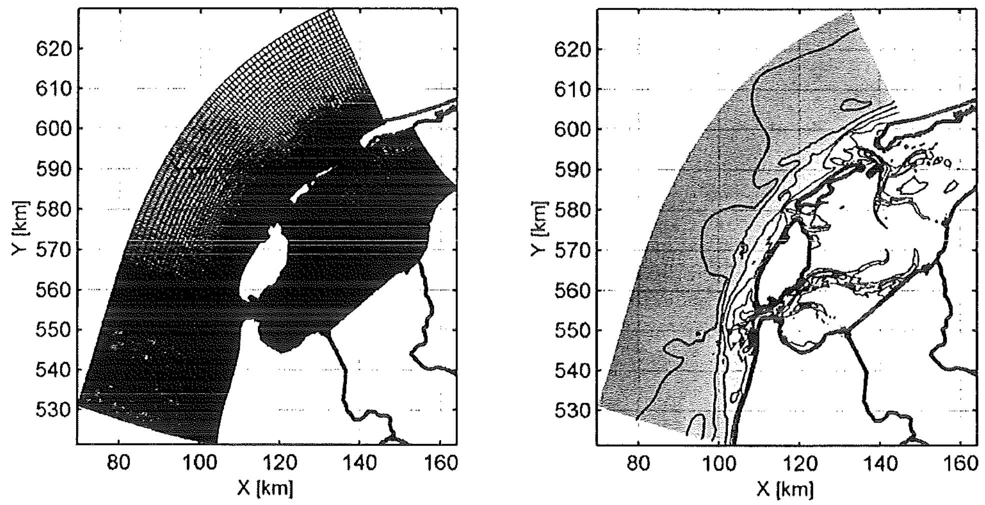

Figure 2.7: Texel Outer Delta model grid (left) and 1933 bathymetry (right)

The Texel Outer Delta (TOD) model application contains Texel inlet and adjacent coastlines (Elias *et al.*, 2005a). The Eierlandse Gat and Vlie inlet are included in the model domain to enable the simulation of the important internal residual volume transport between Vlie and Texel inlet (Ridderinkhof, 1988). The well-structured, orthogonal curvilinear grid has 38311 points, with a maximum resolution of 80x120 m at the location of Texel inlet (Fig. 2.7a). Depending on the simulation the bed topography of the nearshore, the inlets and the basin is based on bathymetric data of 1933 (Fig. 2.7b), 1950, 1975 and 2000 (De Kruif, 2001). Depth measurements were triangularly interpolated to the curvilinear grid.

The North-Holland coastline, the landward coastline in the back-barrier basin, and the island coastlines form closed boundaries (free- slip conditions). The northern basin periphery is chosen on the Terschelling tidal divide (as present in 2000) and set as a zero-velocity boundary. The location of this tidal divide might not be entirely representative for the 1933 situation. Comparison of the 1933 and 2000 bathymetry shows that the tidal divide has migrated slightly westward estimated at 0.5 – 1 km. Sensitivity studies by varying the boundary location and conditions (velocity versus water level boundary, zero-velocity, estimated velocity and estimated water levels) shows that the choice of boundary condition and the exact location of this boundary does not affect the transport model results for Texel Inlet. The open-sea boundaries are located "far away", outside the sphere of Texel inlet's influence and prescribed as water-level elevations.

Bottom roughness is prescribed by a global Chézy coefficient of 61 $m^{1/2}/s$. Enlargement of the bed shear stress due to waves is included using the formulation of FREDSOE (1984). The time step for the flow computations is 60 seconds to fulfill the maximum courant number criterion of 15. Default settings of 1.0 m^2/s and 10.0 m^2/s for the uniform horizontal eddy viscosity and eddy diffusivity coefficients have been applied. The secondary flow option has been used to take into account spiral flow intensity. Computations start from an uniform water level. A two day spin-up and 60 minutes smoothing time prior to the actual computations is sufficient to dissipate the errors induced by the discrepancy between boundary conditions and initial state.

Forcing conditions

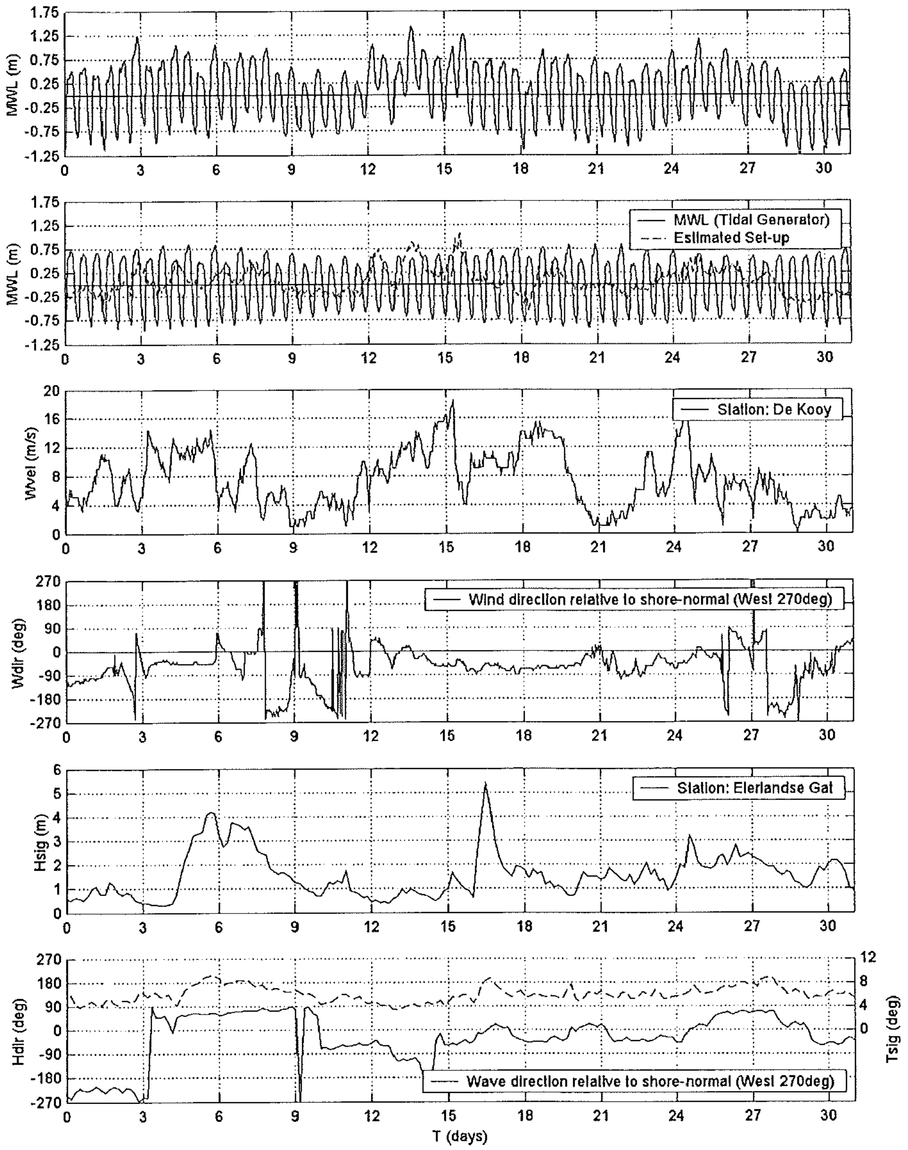

Figure 2.8: Model boundary conditions (time in days relative to 01-01-1999). From top to bottom: (a) Tidal water levels, (b) observed water levels station Den Helder, (c) wind velocities and (d) directions (station De Kooy Den Helder), (e) significant wave heights and (e) directions and peak periods (station Eierlandse Gat).

One-month simulations are made as-realistically-as-possible using tides, wind and waves as model forcing. Boundary conditions are based on the month of January 1999 (Fig. 2.8) due to the availability of observational data (Elias *et al.*, 2004). The open-sea boundaries are driven by measured time-series of water level elevations including tidal and non-tidal contributions (Fig. 2.8b). The water levels are calibrated on the observations of tidal gauges along the Dutch coast via Kalman filtering. Wind stresses on the free surface are based on hourly data of a general wind climate, derived from the measurements of station De Kooy near Den Helder (Fig. 2.8c and d). Waves are forced by spatially constant, time-varying series of hourly wave heights, periods and directions on the open-sea boundaries (Fig. 2.8e,f), based on the Eierlandse Gat wave buoy observations.

Previous studies point to the importance of tidal asymmetry for the sediment exchange between basin and ebb-tidal delta after closure (Dronkers, 1998; Ligtenberg, 1998). To investigate the importance of the tidally driven transports, 1-month simulations are made wherein the open-sea boundaries are prescribed by representative harmonic constituents (frequencies, amplitudes and phases) of the water level elevations. A 1-month period includes two spring-neap tidal cycles (see Fig. 2.8b), therefore, due to the periodicity of tides, provides a fairly representative depiction of the long-term tidal residual transport patterns.

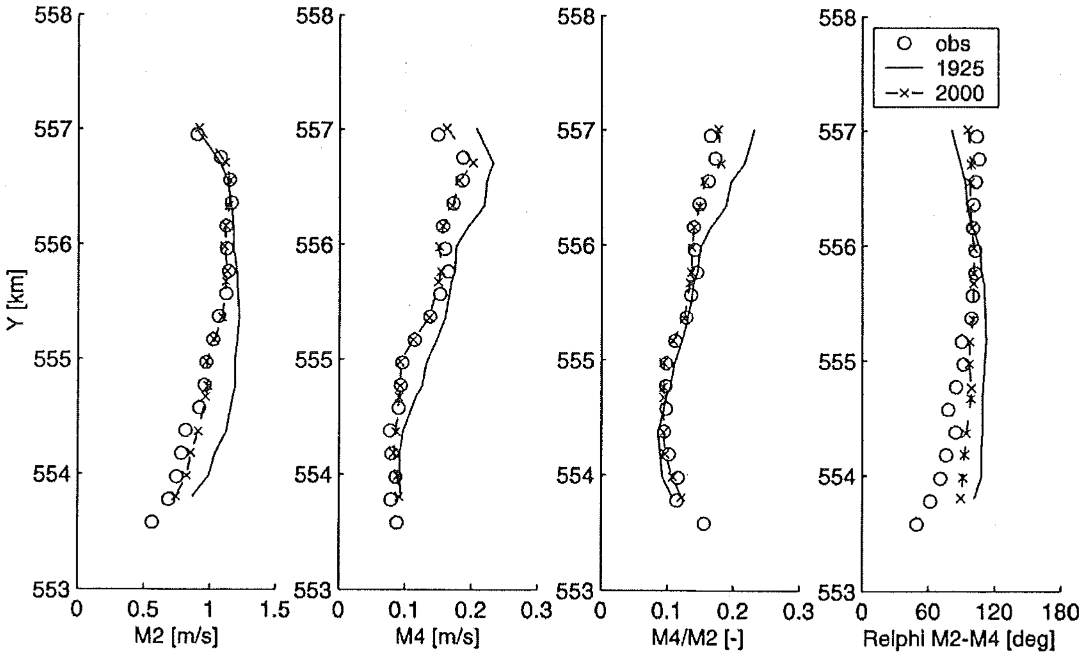

Figure 2.9: From left to right, comparison of (a) M_2 velocity amplitude, (b) M_4 velocity amplitude, (c) M_4/M_2 amplitude ratio and (d) M_4 phase relative to the M_2 for the modelled and observed tidal flow in the NIOZ-ferry transect (see Fig. 2.1 for location).

The tidal model is calibrated on the correct representation of the main tidal constituents (for flow) at the location of Den Helder. Since 1998 an ADCP attached below the hull of the ferry from Den Helder to Texel (location see Fig. 2.1B) is used to measure e.g salinity, surface temperature and flow through the inlet gorge (Ridderinkhof *et al.*, 2002). Preliminary results of the ongoing measurements have been made available by NIOZ for analysis (Bonekamp *et al.*, 2002). Flow data have been used to calculate time-series of depth-averaged flow at seventeen equidistantly distributed aggregation points between Den Helder and Texel. Harmonic analysis has been applied to these time series, and to the simulated flow output in the nearest grid cells, to derive tidal mean flow conditions and amplitudes and phases of the main tidal flow constituents.

The correspondence in the velocity amplitudes of the main tidal constituent (M_2) and its first overtide (M_4) of model and NIOZ-ferry observations is presented in Fig. 2.9 (compare obs and 2000). The focus is on the M_2 and M_4 as in literature (Van De Kreeke and Robaczewska, 1993) it is shown that the long-term residual sediment transport essentially depends on the interaction of the fundamental constituent and the Eulerian mean current, and the interaction of the fundamental constituent and its even over-tides. The correspondence in amplitude ratio and the relative phase difference between model and NIOZ ferry observations (Fig. 2.9c and 9d) indicate that the model is capable to reproduce the tidal asymmetry for the present situation reasonably well.

To our knowledge there is no evidence in the North Sea tidal characteristics that the closure of the Zuiderzee has distorted the tidal wave propagation outside the sphere of inlet influence. The model boundaries are located well outside the ebb-tidal delta domain, and we therefore assume that the closure of the Zuiderzee has had no effect on the external forcing conditions. Changes in the tidal characteristics in the inlet domain result from the local interaction between bathymetry and tides only. Under this assumption we can apply similar tidal and quasi real-time boundary conditions on the different model bathymetries (1933, 1950, 1975 and 2000). Note that we have used an existing model and the model domain did not include the former Zuiderzee. Therefore it was not possible to simulate the pre-closure situation.

Note that no validation could be performed on the accuracy of the simulated transport magnitudes due to the lack of suitable field data. The ongoing analysis of NIOZ-ferry measurements might provide an impression of the transport of suspended matter through Marsdiep in the near-future ((Merckelbach and Ridderinkhof, 2004). Such datasets would provide an indispensable tool for model validation. For the moment, the simulated transport patterns must be considered in the qualitative sense rather than on the exact representation of magnitudes. Sensitivity analysis of model forcing and model parameters by variations in e.g. boundary conditions, bathymetry, bottom friction and wave parameters showed that the qualitative flow and transport patterns are relative insensitive to small-scale fluctuations. This provides confidence in the modelled patterns.

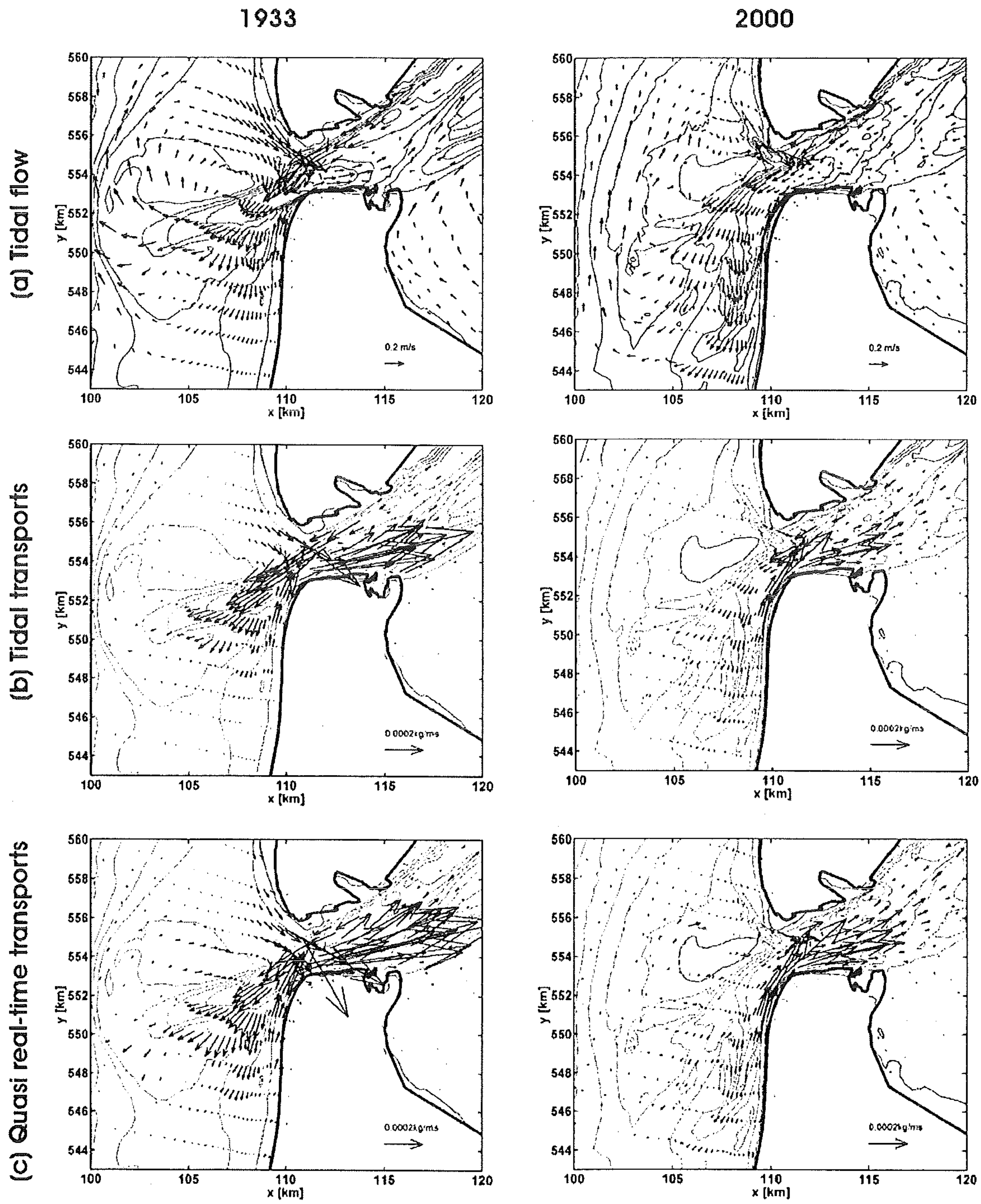

Figure 2.10: Modeled residual vector fields for the 1933 (left) and 2000 (right) simulation. From top to bottom, (a) tidal flow, (b) tidal transport and (c) quasi real-time transports. For clarity only a limited number of vectors have been plotted in x- and y-direction.

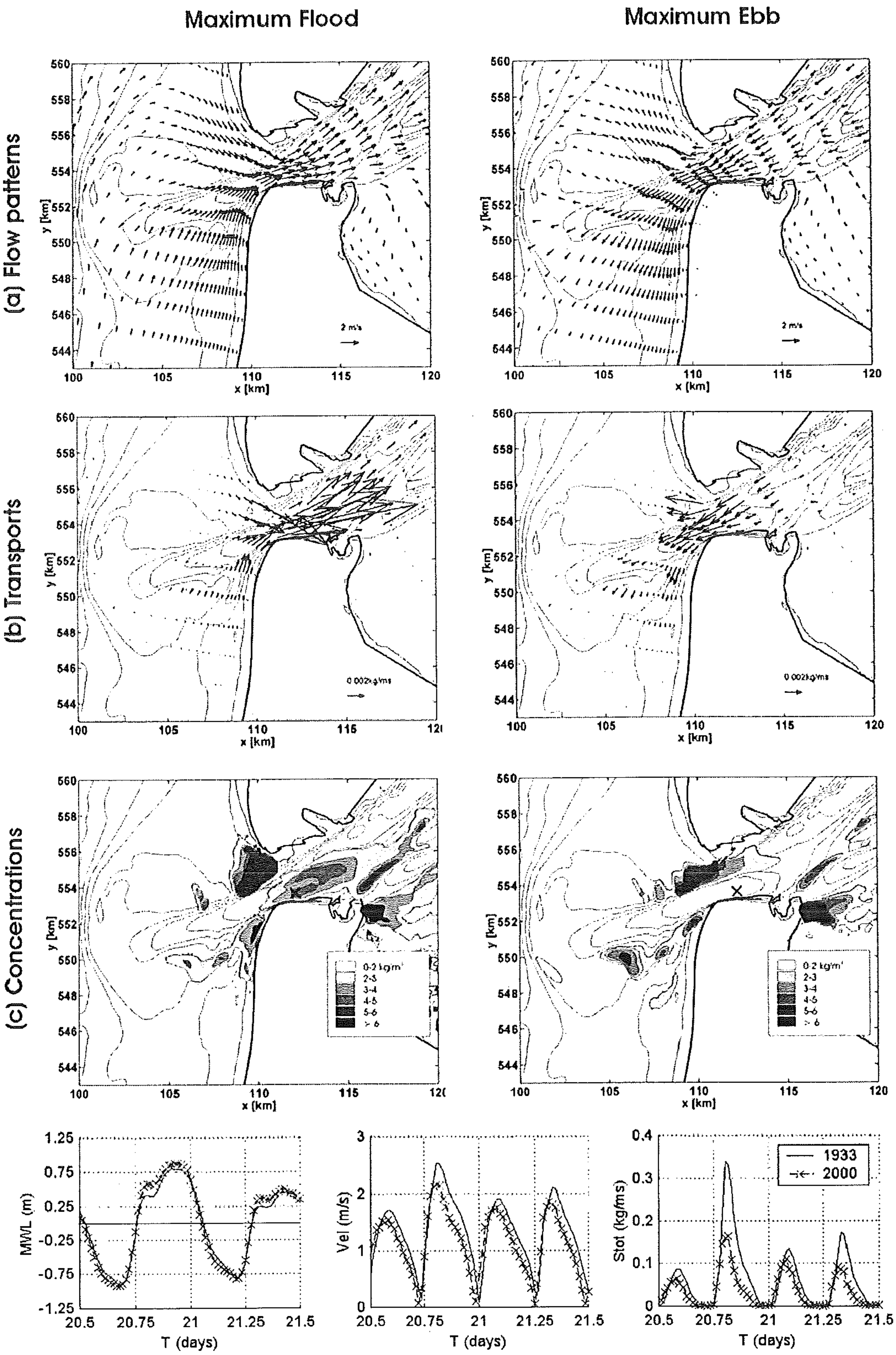

Figure 2.11: Modelled vector fields for maximum flood (left) and maximum ebb (right) for the 1933 simulation. From top to bottom, (a) flow, (b) transports (for clarity only a limited number of vectors have been plotted in x- and y-direction), and (c) sediment concentrations the cross indicates the location of the point-observations, see bottom panels from left to right: mean water levels (MWL), velocity (*Vel*) and total sediment transports (*Stot*).

2.4 Model Results

The analysis of the process-based model results focuses on two main aspects of the post-closure inlet dynamics. Firstly, the back-barrier steering mechanism is investigated; ELIAS et al. (2003; 2005b) suggested that the ebb-tidal delta bed topography alterations formed in response to the changed tidal characteristics of the basin. Analysis of the (residual) flow and transport patterns can possibly provide more insights. Secondly, we aim to identify the dominant mechanisms of sediment exchange between basin and ebb-tidal delta by analysis of the transport rates through Marsdiep.

2.4.1 Flow and transport patterns

Tidal and quasi real-time 1-month model simulations have been made on the 1933, 1950, 1975 and 2000 bathymetry. Fig. 2.10 illustrates the differences in residual patterns for (a) tidal flow, (b) tidal transports and (c) quasi real-time transports. Only results for the 1933 and the 2000 simulation are presented in detail.

Comparison of the residual transport patterns of the tidal- and the quasi real-time simulation shows similar characteristics in the distribution of ebb- and flood-dominated zones (compare Fig. 2.10b and c). The major difference between the two simulations are the dissimilar magnitudes of the sediment transport rates in the channels (and on the shoals). Addition of wind and waves contributes significantly to the flood-dominant transports along the North-Holland and Texel coastline, thereby sediment import into the basin is augmented. As the gross-scale distribution seems governed by tides, and wind and waves contribute mainly to the magnitudes the analysis of the flow- and transport patterns focuses on the tidal simulations.

The tidal residual flow field (Fig. 2.10a) illustrates the distortion of the alongshore propagating tidal wave due to interaction with the complex bed topography of the ebb-tidal delta and the cross-shore inlet currents. In both simulations the southern part of the ebb-tidal delta is dominated by the ebb-residual, whereas the northern part is dominated by the flood-residual. Ebb-tidal currents through Texelstroom accelerate in the narrow constriction of the inlet gorge (Fig. 2.11b) and the outflow dominates the updrift part of the ebb-tidal delta. The strongest residual currents are observed in the inlet gorge where the cross-sectional area is minimal and segregation of flow results in an ebb- and a flood-dominated part. The ebb outflow along Texel coastline prevails over the flood inflow along the North-Holland coastline. This pattern corresponds well to recent flow observations (Ridderinkhof *et al.*, 2002).

Comparing the 1933 and 2000 residual flow and transport fields shows similar patterns in the upper part of the basin and in the inlet gorge (although transport magnitudes differ). The largest instantaneous (Fig. 2.11a) and residual flow magnitudes are observed in the inlet gorge directly after closure. Along the Texel coastline ebb-residual velocities exceed 0.5 m/s and maximum tidal velocities of 2.5 m/s are observed around the tip of Helderse Zeewering (Fig. 2.1d, see Helsdeur). With ongoing adaptation of the bathymetry and enlargement (increasing depth) of the inlet gorge the maximum instantaneous- and residual velocity magnitudes decrease. In 1975 ebb residuals do not exceed 0.4 m/s and in 2000 the maximum residual flow magnitudes range between 0.1 to 0.2 m/s, whereas maximum instantaneous flow velocities are in the order of 2.0 m/s (Fig. 2.11d, 2000).

On the ebb-tidal delta the differences in patterns between the 1933 and 2000 simulations are larger. One of the main features is the imbalance between the location of largest residual flow and transports and the location of the main channels in 1933, and the balance in 2000. After closure the majority of the ebb flow is directed from Marsdiep onto the southern part of the ebb-tidal delta. The main channel is westward directed, whereas largest flow velocities and transport rates are observed at the location of Schulpengat. Main gradients in residual transport rates are observed at the locations where Molengat and (Nieuwe-) Schulpengat developed (erosion) and in Westgat (sedimentation). This inequality between flow and bed topography, directly after closure before morphological updating, might testify to the back-barrier steering effect.

The contradiction between ebb-flow dominance and flood-transport dominance in Marsdiep (compare Fig. 2.10a and 2.10b) can be explained by analysis of the flow-, transport- and sediment concentration fields in more detail (Fig. 2.11a,b and c). Vector fields for the 1933 tidal simulation during maximum flood (Fig. 2.8, T = 20.8125 days) and maximum ebb (Fig. 2.8, T = 21.0625 days) are plotted.

Ebb-flow enters the inlet mainly from the Texelstroom; sediment transport rates and sediment concentrations in Texelstroom are small. This suggests that during ebb only a limited amount of sediment is transported from the basin seaward. It is only in the inlet gorge and on the ebb-tidal delta that sediment is picked-up and transported seaward. The adaptation of the inlet gorge is expected as the pre-closure cross-sectional area of the inlet gorge is predicted to be to small to accommodate the closure-induced tidal-prism increase of approximately 26% (e.g. O'brien, 1931).

Flood flow enters the inlet from the south, along the North-Holland coastline, and from the north, along the Texel coastline. Towards the inlet flow accelerates and sediment is picked-up; sediment concentrations and transport rates increase drastically at the locations of Molengat and Schulpengat (Fig. 2.11b and c). A vast quantity of sediment is transported into the basin. Largest transport rates are observed along the North-Holland coastline.

Averaged over the tidal cycle sediment import along the North-Holland coastline exceeds sediment export along the Texel coastline (Fig. 2.10b). Including wind and waves considerably increases the sediment import (Fig. 2.10c) due to augmented flood-transports rates along the North-Holland coastline.

With ongoing morphological adaptation of the bathymetry, filling in of Westgat and the scouring of Schulpengat and Molengat, the imbalance between transports and bed-topography, and the residual transport rates decrease. In 2000 the residual flow and transport patterns are smaller and correspond reasonably well with the location of the main channels (Fig. 2.10a and b, 2000).

2.4.2 Sediment exchange between basin and ebb-tidal delta; transport rates in Marsdiep

An impression of the sediment exchange between basin and ebb-tidal delta is obtained by analysis of the transport rates through Marsdiep. In Fig. 2.12 the cross-sectional averaged cumulative sediment transports through Marsdiep are plotted for each of the simulations (positive values indicate sediment import).

Despite the fact that the short-term simulations might not present quantitative estimates of the long-term (e.g. year-averaged) sediment-transport rates the qualitative patterns show several interesting features. Firstly, the dominance of the wind- and wave driven contributions to the sediment transports. Note that it are not the wind and waves itself that are important, but the interaction between wind, waves and tides (Elias *et al.*, 2004; Elias *et al.*, 2005a). The wind-driven flow generated by the prevailing south(western) wind climate during the simulated period

(Fig. 2.8c and d) augments flood flow and retards ebb flow. Consequently wind-driven transports are directed towards the inlet increasing sediment import.

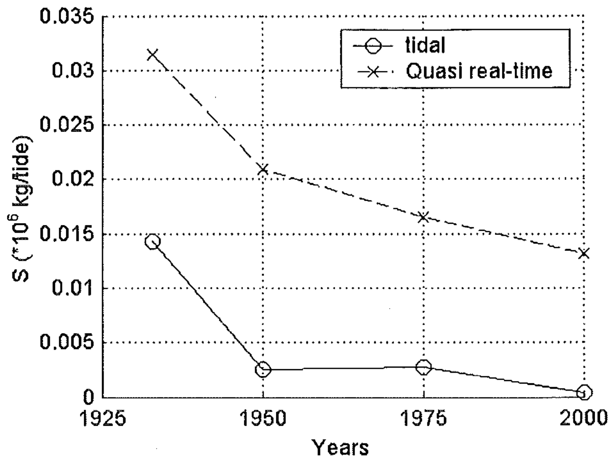

Figure 2.12: Modelled tide-averaged total-load transports averaged over Marsdiep.

Waves contribute directly to the sediment transports due to wave-breaking induced radiation stresses and tidal asymmetry (on the shoals and along the coastline). Indirectly, waves contribute through the mechanisms of enhancement of bed shear stresses and stirring-up of sediment allowing more sediment into suspension to be transported by the tidal flow. The significantly augmented flood transports along the North-Holland and Texel coastlines (Fig. 2.10c) illustrate that especially these indirect mechanisms contribute to the residual sediment transports. As waves are larger, more important, on the ebb-tidal delta than in the sheltered back-barrier area sediment import into the basin is promoted; see Elias *et al.* (2004; 2005a) for details.

A second interesting feature illustrated by the tidally driven transports (Fig. 2.12, 1933) is the large sediment import into the basin after closure and the diminishing rates towards the present situation (similar to the basin and ebb-tidal delta sediment-volume change, Fig. 2.3a). Presently, sediment import due to tides is minor (Elias *et al.*, 2004), possibly tides even induce sediment export (Bonekamp *et al.*, 2002). The similar decrease in sediment transport rates in both the tidal as quasi real-time simulations indicates that the decreasing sedimentation rates of the basin (Fig. 2.3a) are related to the tidal mechanisms. Dronkers (1984) and Ligtenberg (1998) suggested that the decrease is related to alterations in tidal asymmetry; directly after closure a large tidal asymmetry and associated sediment import into the basin existed, and with ongoing morphological adaptation of the basin tidal asymmetry and associated transports reduced.

To obtain insight in the possible correlation between tidal asymmetry change and decreasing sediment transport rates in Marsdiep the tidal flow fields of the 1933 and 2000 simulation are studied in detail. Van de Kreeke and Robaczewska (1993) suggest that the net long-term (monthly-mean) tidal transport through inlets (for coarse sediment) essentially depends on the interaction of the fundamental constituent and the Eulerian mean current, and the interaction of the fundamental constituent and its even over-tides. Since the M_2 is dominant in Texel inlet, in first order, tidal sediment transport essentially depends on the M_0M_2 tidal residual transport and the M_2M_4 tidal asymmetry. The comprehensive Van Rijn sediment formulations within the Delft3D Online Morphology model make it difficult to separate the tidal-residual and tidal-asymmetry driven transports. Therefore, as in Bonekamp *et al.* (2002), the simple model of Groen (1967) is used. This formulation reads,

(1)

herein is b the settling lag time factor, C and $C_{equilibrium}$ represent the instantaneous and equilibrium depth-averaged sediment concentration respectively, w is the angular frequency of the main tidal constituent (M_2), A is a proportionally constant depending on e.g. sediment and bed characteristics and (u, v) are the depth-averaged velocities. For $\beta = 0$ the concentration adjusts instantaneously to the flow and the sediment transports are proportional to the depth-averaged flow velocity by a power 3. This power-relation is often seen as representative for bed load transport. Groen (1967) and Bonekamp *et al.* (2002) use $\beta = 1$, to account for settling lag, as a fair approximation of the suspended load transports in the Dutch Wadden Sea. As we are interested in the relative importance of the interactions rather then obtaining accurate estimates of magnitudes, we avoid the specification of sediment characteristics and transport results are expressed as a squared velocity (m^2/s^2).

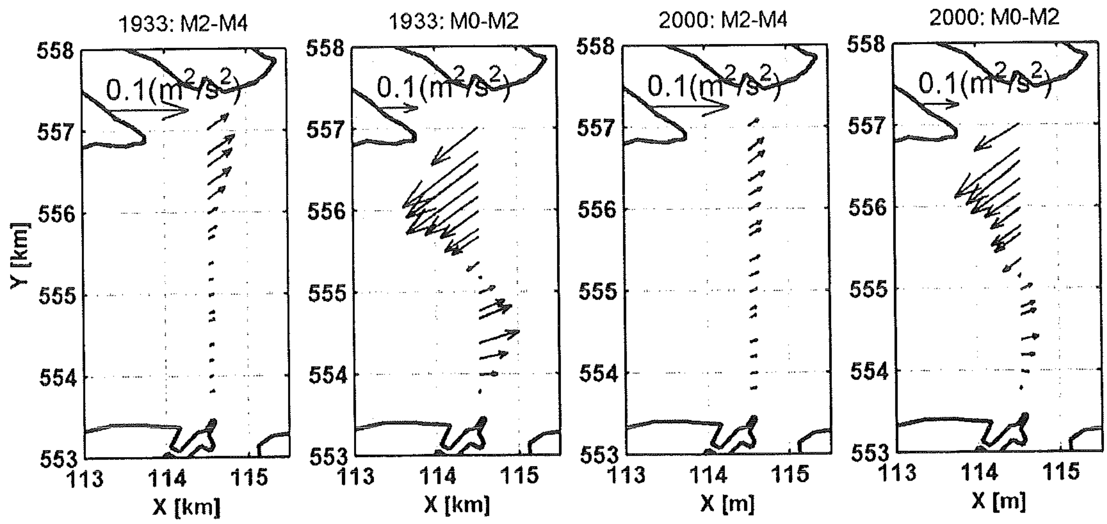

Figure 2.13: Tidally-averaged suspended-load sediment transport vectors based on the GROEN (1967) transport formulation for harmonically analyzed flow model results. From left to right: M2M4 and M0M2 transport vectors for the 1933 and 2000 simulation respectively.

Fig. 2.13 shows comparable suspended load transport vectors for the tidally averaged M_2-M_4 and M_0-M_2 interactions in both the 1933 and 2000 simulation. The ebb-dominant tidal residual sand transport dominates over the flood-dominant tidal asymmetry transport inducing a net export of sediment; in correspondence to Bonekamp *et al.* (2002). Tidal asymmetry driven transports are small due to their proportionality to the cosine of the relative phase between M_2 and M_4 $(2\varphi_{M2} - \varphi_{M4})$. Sediment import occurs for $-90 < 2\varphi_{M2} - \varphi_{M4} < 90$ and sediment export for $90 < 2\varphi_{M2} - \varphi_{M4} < 270$. In 2000 the relative phase difference in both the simulation as the observations varies around 90 degrees (Fig. 2.9d). In 1933 the relative phase difference increases to an averaged value of 110 degrees, that for $\beta = 0$ would result in sediment export (not shown). For $\beta = 1$ (Fig. 2.13) the settling lag acts as an additional shift in phase and a small sediment import is observed (for details see Bonekamp *et al.*, 2002). Based on these results it must be concluded that the change in tidal asymmetry cannot explain the observed decrease in modelled sediment transports rates through Marsdiep.

2.4.3 Summary of model results

The main findings of the model studies include;

1– In 1933 just after closure an imbalance between flow and ebb-tidal delta bed-topography existed.

2– In 2000 flow and ebb-tidal delta bed-topography are in (quasi-)equilibrium.

3– The 1933 residual flow and transport rates exceed the 2000 transport rates.

4– Marsdiep is ebb-dominant for tidal flow, but flood-dominant for tidal transports.

5– Tidal asymmetry cannot explain the diminishing sediment import (transport rates) in Marsdiep.

6– Wind and waves augment sediment transport into the basin considerably due to the increased flood transports.

2.5 DISCUSSION

This discussion section focuses on two aspects. Firstly, we present the obtained insights into the behaviour and development of Texel inlet after closure of the Zuiderzee. The back-barrier steering mechanism and the main mechanisms behind the large morphological changes in the inlet system are discussed. The discussion is presented in the form of a hypothesis that combines existing studies and field data, and the model results presented in this paper. Finally, we indicate the relevance of our study for inlet research in general

2.5.1 Texel Inlet

Back-barrier steering mechanism

Prior to closure of the Zuiderzee the ebb-tidal delta and main ebb-channel Westgat are assumed to be in as stable quasi-equilibrium state (Elias *et al.*, 2005b). The main-ebb-channel (Westgat) protruded seaward, with marginal flood channels along the adjacent shorelines on both sides. In front of, but mainly north of the Westgat, the main shoal area (Noorderhaaks) is formed where ebb-tidal currents and incident wave-generated currents balanced.

After closure drastic changes in ebb-delta morphology were observed. Most noticeable, the southward development of a new main-ebb channel and disappearance of the original westward oriented main-ebb channel (Fig. 2.4). Based on the analysis of observations this rotation was related to the changed hydrodynamics in the basin (Elias *et al.*, 2003; Elias *et al.*, 2005b). These author's used the expression back-barrier steering to describe the forcing exerted by the basin on the ebb-tidal delta.

Model results illustrate that after closure the majority of the ebb-flow was directed from Marsdiep onto the southern part of the ebb-tidal delta and, in 1933, an imbalance between the location of the largest (residual) flow velocities and the presence of tidal channels occurred (Fig. 2.10 and 11). We presume that the closure impacted the hydrodynamics instantaneously, thus the 1933 flow patterns are representative for the post-closure situation, while changes in morphodynamics take time to complete, hence the 1933 bathymetry is representative for the pre-closure bathymetry. The modelled imbalance between flow and bed topography, directly after closure before morphological updating, testifies to the back-barrier steering effect. Thus, morphological changes on the ebb-tidal delta are initiated by the back-barrier induced changes in hydrodynamics.

Sediment Demand, transport capacity and Availability

One of the major unresolved aspects for Texel Inlet are the main mechanisms responsible for the large sediment import into the basin after closure. Sediment import could not be explained by applying a simple transport relation (Groen, 1967) to the modelled tidal flow fields; the ebb-residual sediment export dominates over the flood-dominant tidal-asymmetry driven import (see section Model Results). Using the basic principles of sediment demand, sediment transport capacity and sediment availability we can hypothesize on the mechanisms of sediment import.

Sediment demand; As a result of the closure both the basin and the ebb-tidal delta require sediment. The basin needs sediment to restore the distorted balance between channel and shoal areas; with the closure the relative shallow Zuiderzee was separated from the deep Wadden Sea. Hence, the remaining shoals area is too small relative to the volume of channels (e.g. Eysink, 1990). But also the ebb-tidal delta needs sand. Based on empirical relations (Walton and Adams, 1976) the ebb-tidal delta sand volume was expected to enlarge as the tidal prism increased after closure.

Sediment transport capacity; The large tidal prisms and associated tidal currents are capable of transporting vast quantities of sediment during flood from the ebb-tidal delta to the basin and vice versa during ebb. Residual ebb-flow exceeds the residual flood-flow.

Sediment availability; In principle when a system is forced out of equilibrium the adaptation rate is dependent on the size of the distortion. After closure, on the ebb-tidal delta an imbalance between flow and bed-topography is observed; e.g. large flow velocities are directed onto a relative shallow area. This denotes a large scouring capacity of the tidal currents on the ebb-tidal delta. In the upper part of the ebb-tidal delta new channels are formed (Molengat and Schulpengat) and during ebb a vast amount of sediment is redistributed seaward and deposited (see Fig. 2.11b and c, maximum ebb). Consequently a large quantity of sediment is available for transport into the basin with the flood tidal currents (see Fig. 2.11c showing the large sediment concentrations during maximum flood in Molengat, Schulpengat and in Marsdiep). In the sheltered environment of the basin sediment partly settles.

The majority of the ebb outflow from the basin onto the ebb-tidal delta is concentrated in Texelstroom. Alterations in the size and position of Texelstroom have been minor compared to the large changes on the ebb-tidal delta (Elias *et al.*, 2003). Hence, the upper-part of the basin remained in near-equilibrium state after closure and only a limited amount of sediment is available to be transported seaward during ebb.

Fig. 2.11 (maximum ebb) shows that the majority of the ebb-outflow originates from Texelstroom that contains only limited sediment concentrations and sediment transport rates. Hence, during ebb lower sediment concentrations are transported from the basin onto the ebb-tidal delta (see Fig. 2.11 location of Texelstroom).

In conclusion, both basin and ebb-tidal delta require sediment. More sediment is available during flood to be transported from the ebb-tidal delta into the basin than during ebb. The spatially uneven sediment availability plausible explains why sediment is imported into the basin despite the residual export of flow. With the ongoing morphological adaptation of the ebb-tidal delta the imbalance between flow and bathymetry, and the sediment import into the basin diminish.

Wind, Waves and Estuarine Circulation

In the present situation the tidal residual transport rates through Marsdiep are only minor; both basin and ebb-tidal delta are in a near-equilibrium state (Elias *et al.*, 2005a). Model results point to wind- and wave-driven contributions to the sediment transports as principal mechanism for sediment import. The underlying principle is similar to the tidal sediment import after closure: spatially uneven sediment availability on the ebb-tidal delta and in the basin.

On the ebb-tidal delta wind and waves are more important than in the basin. Wave breaking, enhancement of bed shear stresses and stirring-up of sediment allows more sediment into suspension to be transported by the tidal flow. As waves are larger on the ebb-tidal delta than in the sheltered back-barrier environment sediment import during flood exceeds sediment export during ebb.

A mechanism that we have not investigated in this study, but which might have played a role are residual transports due to estuarine circulation. Prior to closure the majority of the fresh-water was discharged into the back part of the basin, roughly 130 km away from the inlet gorge. Due to mixing processes we can assume that water was already well-mixed as it reached the inlet gorge. Since closure, fresh-water is discharged through sluices in the closure dam, only 30 km from the inlet. Stratification in the inlet gorge has been observed during periods of high fresh-water discharge (Blok and Mol, 2001). The importance of stratification, the resulting estuarine circulations and residual sediment transports are under investigation, but preliminary results indicate that during periods of major fresh-water discharge sediment import into the basin is augmented.

2.5.2 Using Complex Models in Tidal Inlet Research

This study illustrates the necessity of process-based model simulations in cases of large changes in complex systems. The use of simplifications of flow and sediment transport mechanisms can lead to spurious results. Even an intermediate approach, using complex flow model results and a simple transport formulation (assuming that the transports can be expressed as a cubic expression of the flow velocity) might present an over-simplification of reality.

Sediment transports are the sum of flow and sediment concentrations, and sediment availability forms a crucial element. Different forcing conditions (viz. on the ebb-tidal delta wind-and waves are generally more important for bringing sediment in suspension than in the sheltered basin) in basin and ebb-tidal delta can lead to spatially uneven sediment availability. Such imbalances in availability can result in sediment transports opposite to what would be expected from the residual flow results and simplified models. Despite ebb-dominant flow, the higher-sediment concentrations stimulate flood-dominant sediment import.

Due to the large spatial- and temporal scales inlet systems (tens to hundreds of kilometres in space and time scales of centuries) it is practically impossible to obtain field data of sufficient spatial and temporal resolution to resolve the larger-scale patterns. Quasi real-time (high-resolution) model simulations are indispensable assets to obtain such coverage. Inlet research can greatly benefit from integrating field and high-resolution model data .

2.6 CONCLUSIONS

Inlet research has long relied on expert-judgment based interpretations of observations. However, due to the spatial extent of inlet systems and the temporal and spatial variety in dominant processes it is practically impossible to obtain field data with sufficient resolution and accuracy over the entire inlet domain. This makes identification of the dominant processes and mechanisms difficult, especially if the inlet is subject to large morphological change due to e.g. human intervention, and the observed developments cannot be explained by existing conceptual models and empirical relations. The large morphological changes on the ebb-tidal delta of Texel Inlet after closure of a major part of the basin clearly illustrate this statement.

An integrated approach of field- and model data analysis can considerably help to understand inlet dynamics and evolution. The state-of-the-art process-based model (Delft3D Online morphology, Lesser *et al.*, 2004) fully integrates sediment transport and bed level change in the flow module; morphologic changes are calculated simultaneously with the flow calculations. One of the main assets of this type of model is the ability to compute quasi real-time. Point-oriented observations can be used to force the model as-realistically-as-possible and the model generates synoptic 'field'-data of high spatial and temporal resolution in the instrumented and the un-instrumented areas of the inlet domain.

Initial simulations during various stages of the morphological adaptation process were made. The changing flow and sediment transport patterns of the various simulations allows to identify the dominant processes and mechanisms for the large morphological changes and the sand exchange between inlet, ebb-tidal delta and coast.

Analysis of the model results shows that shape and volume of the ebb-tidal delta are not only determined by the relative importance of wave versus tidal energy (e.g. Hayes, 1979). Moreover, the back-barrier steering mechanism is shown to trigger the observed morphological developments on Texel Inlet's ebb-tidal delta. After closure, the imbalance between the location of the largest residual flow and the presence of tidal channels on the ebb-tidal delta, and the balance in the upper part of the basin, provides a larger sediment availability during flood than during ebb. This spatially uneven sediment availability plausibly explains why sediment is imported into the basin despite the residual export of flow. With the ongoing morphological adaptation of the ebb-tidal delta the imbalance between flow and bathymetry and the sediment import into the basin diminishes. Also, in the present situation inequality in sediment concentrations during ebb and flood due to higher contributions of wind and waves on the ebb-tidal delta than in the basin might partly explain the sediment import into the basin.

Model simulations could be significantly improved by field observations on sediment concentrations and transport rates. The absence of such field measurements limits our model to a qualitative analysis rather than providing quantitative estimates. In addition, further research is needed to identify the importance of estuarine circulation for the sediment import into the basin.

Chapter 3
FIELD- AND MODEL DATA ANALYSIS OF SAND TRANSPORT PATTERNS IN TEXEL TIDAL INLET, THE NETHERLANDS

E.P.L. Elias, Cleveringa, J, M.C. Buijsman, J.A. Roelvink, M.J.F. Stive

Abstract:

Texel inlet, the largest inlet in the Dutch Wadden Sea, has undergone drastic changes in the morphology of basin, ebb-tidal delta and adjacent coastlines after closure of a major part of its back-barrier basin. Despite intensive monitoring and analysis, present observation-based conceptual models lack the subtle physics necessary to explain the sand exchange between inlet, ebb-tidal delta and adjacent coastlines.

Fundamental understanding of the inlet dynamics and evolution is obtained by integrating field- and model-data analysis. The state-of-the-art process-based model Delft3D Online Morphology has been used to generate synoptic data of high spatial and temporal resolution over the inlet domain. It is shown that the Delft3D Online Morphology model is capable of quasi real-time simulation of the dominant flow and transport patterns over a 3-month period on the scale of the inlet. The high-resolution numerical model results prove to be a valuable tool in identifying the main transport patterns and mechanisms in the inlet domain. Qualitative transport patterns in Texel Inlet and its associated ebb-tidal delta are derived by integration of the high-resolution observations and model results.

The present ebb-tidal delta developments are best described as a second-stage self-organizing phase of redistribution and recirculation of sediments to obtain a natural dynamic equilibrium state, adapted to the changed configuration of the main-ebb channels. Sand is transported from the abandoned ebb-delta front (western margin of Noorderhaaks) and along the adjacent coastlines into the basin were it partly settles. Ebb-tidal currents redistribute sand back from the basin mainly onto the southern ebb-tidal delta shoals. Large gross transport rates, but small morphological changes points to sediment recirculation. Sediment import into the basin results from net flood dominated transport due to tidal asymmetry, landward directed wind- and wave driven flow, and larger flood transport capacities due to wave effects (e.g. enhanced bed shear stresses and stirring of sediment), that exceed the net ebb-dominated tidal residual transports.

Keywords: Sediment Processes; inlets; process-based models;

3.1 INTRODUCTION

A large part of the world's inhabited coastlines is formed by sequences of tidal inlets and barrier islands. Basically, tidal inlets are constricted openings in the coastline through which tides exchange water and sediment between the back-barrier and the open-sea. Tidal inlet systems are known to participate dynamically in the coastal tract (Delft3D Online Morphology model, Cowell *et al.*, 2003) due to the capacity to store (or release) large quantities of sand in the deltas. This importance in the coastal sand balance has resulted in an early interest into inlet behaviour by coastal engineers. Due to the large spatial- and temporal scales involved, tens to hundreds of kilometres in space and time scales of centuries, these studies have long relied on observation-data analysis (De Vriend, 1991), and a range of conceptual models and empirical relations to explain the variety in size, volume, and the distribution of channels and shoals in the inlet system was produced (e.g. Escoffier, 1940; Battjes, 1962; O'brien, 1969; Hayes, 1975; Hine, 1975; Oertel, 1975; Walton and Adams, 1976; Hayes, 1979; Hubbard *et al.*, 1979; Fitzgerald, 1988; Sha, 1990; Hicks and T.M., 1996; Kana *et al.*, 1999; Elias and Cleveringa, 2003). These studies have significantly contributed to an improved understanding of the inlet behaviour and evolution, but lack comprehensive descriptions of the underlying physics. With the sediment transport in the vicinity of inlets being notoriously complex due to the non-linear interaction between water motion (tide, wind and wave-driven), and the (highly-variable) channel and shoal structures many aspects of inlet dynamics are yet unknown. Awareness of this gap in inlet knowledge is often emphasized when inlets are impacted by changes in forcing such as large-scale human intervention, and existing empirical relations and conceptual models fail to explain many of the observed developments. A clear example of such case is the response of the Texel inlet to the closure of a major part of its back-barrier basin (Closure of the Zuider Sea in 1932). Extensive changes in inlet morphology have been observed since the closure (e.g. Battjes, 1962; Sha, 1990; Elias *et al.*, 2003), nevertheless still no univocal conclusions about the effects of the closure on the hydrodynamics and morphodynamics of the inlet system, and the consequences for the coastal sand balance of the Holland coast haven been obtained.

Nowadays, fundamental understanding of inlet dynamics can be obtained by mathematical modelling. Process-based models based on physical laws describing the water motion, sediment transport and bottom change by a series of mathematical formulations have long been applied in the modelling of short-term coastal processes. Recent studies by e.g. Wang et al. (1995), Cayocca (2001), Van Leeuwen (2002), Hibma (2004) and Van Maren (2004) have shown that the nowadays such models can be used successfully even in the complex environments of tidal inlet systems. Recent advances in process-based modelling (Delft3D Online Morphology model) include the computation of sediment transport and bed level change fully integrated in the flow module (Lesser et al., 2004)(Lesser *et al.*, 2004). Morphologic changes are now calculated simultaneously with the flow calculations. One of the major assets of this type of model is the capability to increase the spatial and temporal resolution of point-oriented field observations. Point-oriented observations are used to force the model as-realistically-as-possible (quasi real-time by measured time-series of wind, waves and tides) and the model results provides synoptic, more-or-less realistic data of high spatial and temporal resolution over the inlet domain. Analysis of this data can provide valuable information on governing flow and sediment transport patterns in the instrumented and the un-instrumented areas of the domain.

Our research goal is to gain fundamental understanding of inlet– coast interaction by analysis of the Texel Inlet and the neighbouring coastlines of North-Holland and Texel (the Netherlands). Large changes in the morphology of the basin, the ebb-tidal delta and the adjacent coastlines were observed after closure of a major part of its back-barrier basin (closure of the Zuider Sea in 1932). Although, the major closure-induced effects on the ebb-tidal delta and in the basin have appeared in a time span of approximately 40 years after closure (Elias *et al.*, 2003), the coastlines still belong to the most erosive sections of the Dutch coast and maintenance requires large efforts; since 1991 over 25 Mm3 of sand has been nourished (Roelse, 2002). It is assumed that the majority of the sand losses results from the interaction of the coastline with Texel Inlet. Especially, the ebb-tidal delta channels Molengat and Nieuwe Schulpengat locally induce severe erosion (Cleveringa, 2001; Elias and Cleveringa, 2003). How the sand exchange between inlet, ebb-tidal delta and adjacent coast occurs, and which processes determine this exchange is still not fully understood even though many studies have been conducted in the past.

We present an up-to-date conceptual sand transport model for the inlet based on an integrated approach of field- and process-based model data analysis. We start this paper with a general description of Texel Inlet and outline the research method. In the next sections we subsequently present the analysis of recent high-quality observational data, the process-based model and model results. Model results are obtained from the quasi real-time simulation of flow and transports through Texel Inlet over a 3-month period. Quasi real-time denotes that the model simulations are driven as-realistic-as-possible by measured time-series of water levels, wind and waves. The state-of-the-art process-based model Delft3D-Online Morphology has been used. To our knowledge quasi real-time simulations of flow and transports on the large-scale of the inlet, *O(km)*, over periods significantly longer than the tidal period have not yet been made. The high-resolution model results provide synoptic data of flow and sediment transports over the inlet domain, enable the validation of the observational data-based conceptual ideas on flow and transports and make identification of the dominant processes and mechanisms for flow and transports in the instrumented and un-instrumented areas of the system possible. We conclude by assimilation of the obtained insights into a schematic representation of the dominant transports patterns for the present state of Texel Inlet.

3.2 STUDY AREA

Texel Inlet is the largest tidal inlet of the Dutch Wadden Sea and is located in the northwestern part of The Netherlands between Den Helder and the barrier island Texel (Fig. 3.1). The inlet is probably the longest regularly monitored inlet worldwide with bathymetric datasets of the ebb-tidal delta being available since the 16th century. Texel Inlet is a mixed-energy tide-dominated (METD) inlet (Davis and Hayes, 1984). Following the classification of Hayes (1979) the inlet qualifies as mixed-energy wave-dominated, even under spring tide conditions. However, the morphology of the inlet shows tide-dominated characteristics such as a large ebb-tidal delta. This is caused by the large tidal prism and the relatively low wave energy. Fig. 3.1 shows the present-day geometry and bathymetry of the inlet including its back-barrier drainage area.

The Marsdiep inlet gorge has a minimum width of 2.5 km and a maximum depth of 53 m. On average the tidal prism through the inlet is $1x10^9$ m^3, with both ebb- and flood- tidal velocities between 1 and 2 m/s. Marsdiep forms the connection between the main channel in the basin, Texelstroom, and the main-ebb channels, Schulpengat and Nieuwe Schulpengat, on the ebb-tidal delta. The ebb-tidal delta protrudes approximately 10 km seaward and 25 km alongshore, determining the nearfield bathymetry of the adjacent North-Holland coast in the south, and the Texel Island coast in the north. The ebb-tidal delta is asymmetrically shaped. The centre is formed by the large supra-tidal shoal Noorderhaaks, which faces the inlet gorge Marsdiep. North of Noorderhaaks, a large spit-shaped sub-tidal shoal (Noorderlijke Uitlopers van de Noorderhaaks) extends along the Texel coastline. The Molengat channel separates this shoal from the coastline. Interaction of Molengat with the coast is regarded as a main factor in the structural sand-losses of the adjacent beaches (Cleveringa, 2001). On the southern part of the ebb-tidal delta the two main channels, Schulpengat and Nieuwe Schulpengat, are distinctly updrift orientated with respect to the northern directed tidal wave propagation and net littoral drift. Nieuwe Schulpengat locally induced severe erosion of the North-Holland coastline (Elias and Cleveringa, 2003).

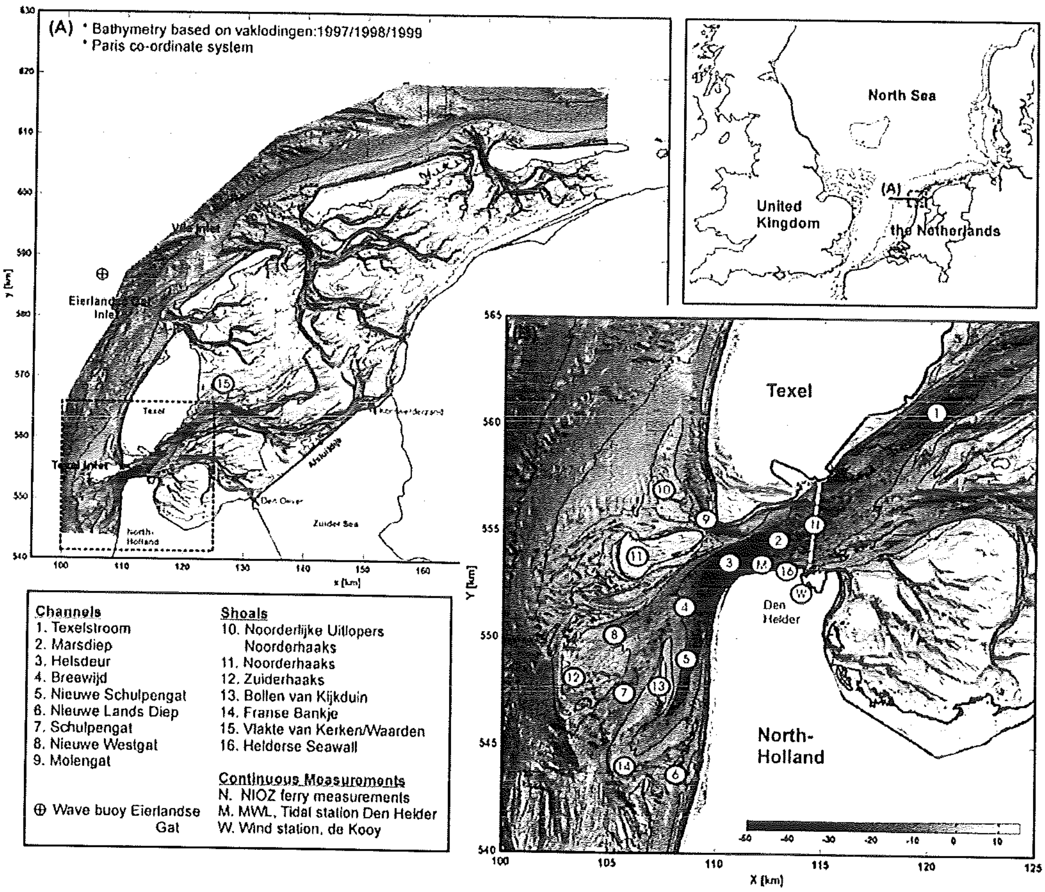

Figure 3.1: Location map of (A) the Western part of the Dutch Wadden sea and (B) Texel Inlet in detail (Co-ordinates are based on the Paris co-ordinate system).

Based on the analysis of bedforms, sedimentary structures, geomorphology and current data Sha (1986a) proposed a conceptual sand transport model to explain the asymmetrical ebb-tidal delta shape. In brief, sand is transported to the seaward margin of the ebb-tidal delta by the ebb flow through the channels and is then carried further north by converging tidal currents along the delta front. Part of the sediments are deposited on the sub-tidal Noorderhaaks shoals due to the weak circular character of the currents. The predominant waves from the north redeposit the sediments shoreward, forming swash bars and channel-margin bars that produced Noorderhaaks. The recent studies of Elias *et al.* (2005a; 2005c) show that the present asymmetrical ebb-tidal delta shape formed as a result of adaptation to the effects of the closure of the Zuider Sea in 1932. This closure, being the largest and most far reaching intervention ever constructed in the Wadden Sea, reduced the drainage area of Texel basin from approximately 4000 km^2 to 720 km^2. The altered hydrodynamics in the basin (e.g. change in tidal characteristics, amplified tidal prism and the northward displacement of the basin centre) increased flow through Texelstroom. As a result the outflow onto the ebb-tidal delta was more southward directed triggering the updrift migration of the main tidal channels and alterations in shoal areas (Elias *et al.*, 2005c). The major closure-induced effects on the ebb-tidal delta appeared in a time span of approximately 40 years after closure (Elias *et al.*, 2003). Ever since relative stability exists in the position and orientation of the main channels and shoals on the ebb-tidal delta.

3.3 METHOD

To obtain an impression of the present morphodynamic inlet behaviour we have determined the dominant sediment transport patterns based on the analysis of field observations and model data. Texel Inlet has been intensively monitored by Rijkswaterstaat (Ministry of Transport, Public Works and Water Management). Available long-term observational datasets of water levels, wind, waves, currents and discharges, bathymetry, bedforms and sediments provide an unique opportunity to study inlet evolution in detail. However, these datasets have two major drawbacks: (1) the accuracy of the older measurements is unknown and (2) due to the large effects of the closure of the Zuider Sea on the behaviour of ebb-tidal delta and basin, the local measurements taken during the adaptation period might not be representative for the present state of near-equilibrium (Elias *et al.*, 2003). This latter deficiency might explain why still no conclusive description of the dominant flow- and transport patterns on the ebb-tidal delta and through the inlet exists even though many studies have been conducted (e.g. Beckering Vickers, 1951; Battjes, 1962; Sha, 1986a, b, 1989; Steijn, 1997; Van Marion, 1999; Steijn and Jeuken, 2000).

Following a short description of the wind and wave climate, we concentrate on the analysis of measurements that were obtained recently and accurately. Flow through the inlet gorge is analyzed using high-frequent ADCP flow observations in the inlet gorge and main channels on the ebb-tidal delta (Blok and Mol, 2001; Ridderinkhof *et al.*, 2002). The analysis of bathymetric changes, using three-yearly echo-sounding based surveys over the period 1986-2000, and bed-form analysis of multi-beam survey data of seafloor bathymetry (Rab, 2003) provides insight in the dominant sediment transport directions in parts of the ebb-tidal delta.

The availability of these observations on bathymetry, bedforms, currents, discharges and water levels allows for the construction of a conceptual sand transport model. Secondly, the observations are used for a thorough calibration and validation of a state-of-the-art process-based morphological model application of the Texel Inlet. This model is based on Delft3D Online Morphology (Lesser *et al.*, 2004) and has been used for depth-averaged simulations of the water motion and sediment transports in the inlet domain over a 3-month period in the winter and spring of 1999 (01-01-1999 / 01-04-1999). We will only present a short description of the model basics as the model is discussed in detail by Lesser *et al.* (2004).

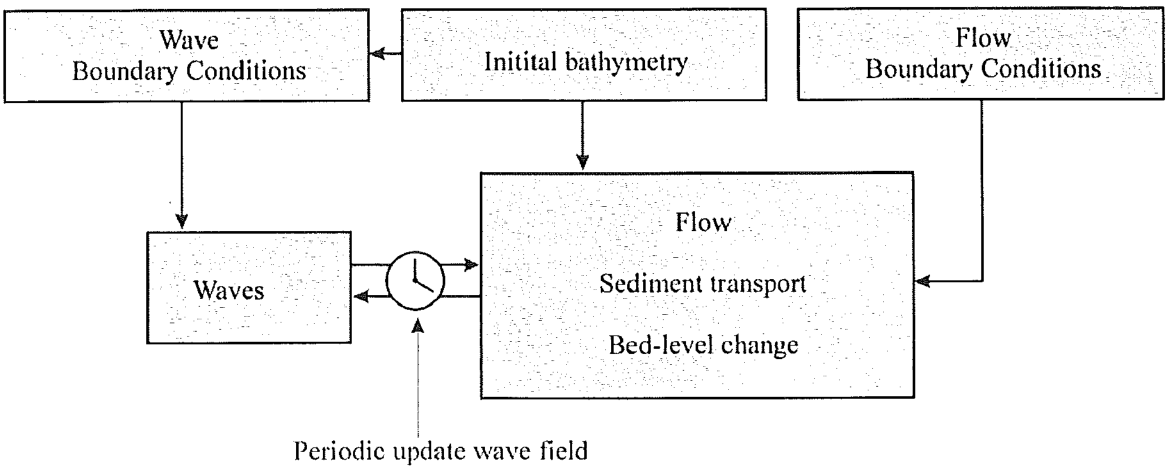

Figure 3.2: Schematic representation of the Delft3D Online Morphology model.

The Delft3D-FLOW module (Version 03.20.00.01) forms the centre of the model, simulating water motion due to tidal and meteorological forcing by solving the well-known unsteady shallow-water equations (Stelling, 1984). The Alternating Direction Implicit method is used to solve the vertical and horizontal momentum and continuity equations (Stelling, 1984; Leendertse, 1987)on a staggered grid. Wave effects, such as enhanced bed shear stresses and wave forcing due to breaking, are integrated in the flow simulation by running the 3rd generation SWAN wave processor (Version 40.11-ABCDEF). The SWAN-model is based on discrete spectral action balance equations, computing the evolution of random, short-crested waves (e.g. Holthuijsen *et al.*, 1993; Booij *et al.*, 1999; Ris, 1999). Physical processes included are: generation of waves by wind, dissipation due to whitecapping, bottom friction and depth induced breaking, and, non-linear quadruplet and triad wave-wave interactions. Wave propagation, growth and decay are solved on a subset of the flow grid. The results of the wave simulation, such as wave height, peak spectral period, and mass fluxes are stored on the computational flow grid and included in the flow calculations.

The Online Morphology version (release 5.25.02.r.1) supplements the flow model results with sediment transport computations. Fig. 3.2 schematically illustrates the model. Each computational time-step sediment transports of non-cohesive sediment are computed within the flow simulation based on the transport formulations of Van Rijn (1993; 2000; 2002), summarized in Walstra and Van Rijn (2003). In the Van Rijn formulation the total sediment transport is obtained from the sum of the bed load and suspended load transports. Bed load transports represent the particle movements in the wave-boundary layer due to currents and waves. Suspended load transports are the transports above the bed layer due to e.g. transport of sediment by high-frequency oscillating flow (cross-shore orbital motion and advective transports by the time-averaged current velocities including wave effects such as wave motion induced reduction of current velocities near the bed and enhancement of near-bed concentrations due to wave stirring). Before bed-level updating sediment transport rates are corrected for bed-slope effects. Testing and validation of Delft3D-Flow Online Morphology is reported in detail by Lesser *et al.* (2004). The study of Grunnet *et al.* (2004) provides a practical example of the model for a shore-face nourishment.

3.4 OBSERVATIONAL DATA ANALYSIS

3.4.1 Wind and waves

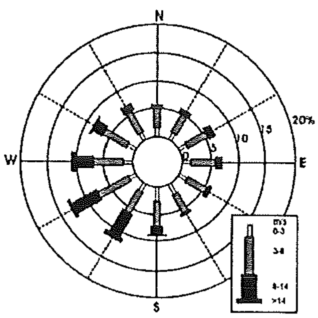

Figure 3.3: Wind rose station De Kooy (based on data 1971-2000 from Royal Netherlands Meteorological Institute).

Overviews of the governing wind and wave conditions along the Holland coast and Texel Inlet are presented by e.g. Roskam (1988), De Ronde *et al.* (1995), Wijnberg (1995) and Coelingh (1996). The wind rose of Fig. 3.3 summarizes the observations for station De Kooy near Den Helder (location see Fig. 3.1) over the period 1971 - 2000. Winds from the sector southwest to northwest prevail. The yearly averaged wind speed is 7 m/s from the west-southwest. These prevailing west-southwesterly winds are the main driving force for the north-east ward residual velocities (5 to 10 cm/s) on the lower shoreface of the Holland coast (Roelvink *et al.*, 2001b). Little information is present on the importance of wind-driven flow in the inlet domain.

The wave climate at Texel Inlet is dominated by wind-generated waves in the North Sea basin. Contributions of swell are minor. Most frequently wave directions (~90%) lie between southwest- and north. Wave-climate measurements, representative for Texel Inlet, are conducted at the nearby Eierlandse Gat wave buoy, located at a water depth of 26 m (Paris co-ordinates X: 106.514, Y: 587.985 km, see Fig. 3.1). Analysis of long-term time-series (1979-2001) reveals a mean significant wave height of 1.3 m from the west, with a corresponding significant wave period of 5 seconds. The highest waves occur from north-westerly directions due to the larger fetch. During storms wind-generated significant wave heights can reach heights over 6 m.

Waves are an important factor governing sand transports on the ebb-tidal delta. Waves contribute directly to the sediment transports via currents due to radiation stresses generated by wave breaking of obliquely incident waves and due to wave asymmetry. Indirectly waves enhance bed-shear stresses and stir-up sediment, allowing more sediment into suspension to be transported by the tidal- and wind-driven flow. To obtain an estimate of the dominant components of the wave climate for sediment transport, we have sorted the Eierlandse Gat observations over the period 1979 - 2001. A sub-division in four wave height classes and four direction classes between 180° and 360° is made. Wave directions between 0° and 180° are generated by offshore directed wind and of negligible height near the Texel Inlet. For each class the representative morphological wave height (Hmor) is determined using,

$$H_{mor} = \left(\frac{1}{n} \sum_{i=1}^{i=n} H_{m0}(i)^k \right)^{\frac{1}{k}} \qquad (1)$$

where, n represents the total number of observations [-], H_{m0} is the significant wave height [m] and k is the power relation between transport and wave height [-], a value of 2.5 is used as in CERC formulation.

The total morphological impact (*MI*) for each class is obtained by multiplying with the probability of occurrence. Table 1 shows the morphological impact for each of the individual wave-height and direction classes.

Table 1: Morphological impact (*MI* in %) of selected wave height and direction classes. Data from the Eierlandse Gat wave buoy over the period 1979 to 2001. The number of occurrences per class is shown in days.

Wave dir (dir)	H_{m0} (m)								Total	
	0-1		1-2		2-4		> 4 m			
	days	*MI*	days	*MI*	days	*MI*	days	*MI*	days	*MI*
180 - 225	361	*1*	420	*4*	155	*6*	1	*1*	936	*12*
225 - 270	616	*1*	690	*7*	404	*18*	24	*4*	1734	*30*
270 - 315	563	*1*	541	*5*	317	*15*	37	*6*	1458	*27*
315 - 360	933	*1*	935	*9*	368	*16*	30	*5*	2266	*31*
total	2473	*4%*	2586	*25 %*	1244	*55 %*	92	*16 %*	6394	*100%*

Waves from the direction classes between southwest (225°) and north (360°) contribute near-equal to the morphological impact. Due to the non-linear relation between waves and sediment transport, especially the larger wave heights are important. Only 20% of the observations exceed the 2 m wave height, but these waves account for 70% of the morphological impact. At the Eierlandse Gat wave buoy the southward component of the morphological impact exceeds the northward component, which results in a net southward directed wave-driven transport (similair to the observations of Sha, 1986a). Note that the littoral drift along the North-Holland coastline is northward, directed towards the inlet (Van Rijn, 1997).

Although measurements of the wave-climate are absent in the direct vicinity of Texel Inlet, studies have shown the effectiveness of ebb-tidal deltas in modifying the nearshore wave-climate and reducing the wave energy on the adjacent coastlines (e.g. Hine, 1975; Fitzgerald, 1988; Van Rijn, 1997). The shallow ebb-tidal delta shoals provide a natural breakwater for the adjacent shorelines. Refraction on the large shoal areas, wave-breaking on the shoals (especially during the high wave-energy events with large morphodynamic impact) and wave blocking by the supra-tidal shoal areas can largely modify and distort the nearshore wave climate. At Texel Inlet it is expected that the larger refraction and wave sheltering of the waves from the north results in a net northward directed transport along the North-Holland coastline, and vice versa, the larger reduction of southern waves results in a southward directed transport along the Texel coastline (also see Steijn, 1997). Thus, on either side of the inlet wave-driven transports are directed towards the inlet.

3.4.2 Tidal flow and transports through Marsdiep

Flow

The tidal motion along the Dutch coast and in the Wadden Sea is generated by the tidal wave entering the North Sea from the Atlantic Ocean between Scotland and Norway in the north, and through the Calais Straight in the south. Interference of these two waves, distortion due to Coriolis effects and bottom friction generates a complicated tidal flow pattern in the Southern part of the North Sea with anti-clockwise rotation and propagation around amphidromic points. Along the Dutch coast the result is a composition of a standing and progressive tidal wave propagating from the south to the north generating maximum tidal velocities in the range of 0.5 to 1.0 m/s. Near, Texel inlet this tidal wave meets a second Kelvin-type tidal wave that propagates from west to east along the Wadden-sea Islands. At Texel Inlet the semi-diurnal tidal movement is the main driving force behind the horizontal water flow through the inlet, with the M_2 constituent being the dominant tidal component. Ebb- and flood-tidal velocities in Marsdiep range between 1.0 and 2.0 m/s. The tidal curve is asymmetrical mainly due to distortion of the M_2 tide by the M_4 constituent, resulting in a faster rise than fall of the tide. The vertical tide has a mean tidal range of 1.38 m in Den Helder, increasing up to 2.0 m during spring tides, while decreasing to about 1.0 m during neap tides.

Tidal discharge measurements in the Texel Inlet have been conducted regularly since 1939. Discharge measurements were traditionally based on "13-hour" observations over a single tidal cycle that were translated afterwards to mean conditions using the ratio of the measured and mean tidal range (e.g. De Reus and Lieshout, 1982; Blok and Mol, 2001). As the accuracy of the measurements and representativeness of the translation to mean conditions are not known, relatively large fluctuations in the residual discharge magnitudes of consecutive measurements are observed; estimates of the residual tide-averaged discharges range between 0 and -200 m^3/tide (export). It is only through analysis of the recent NIOZ-ferry measurements (Ridderinkhof *et al.*, 2002) that a reliable quantitative indication of the residual flow through the Marsdiep inlet gorge could be obtained.

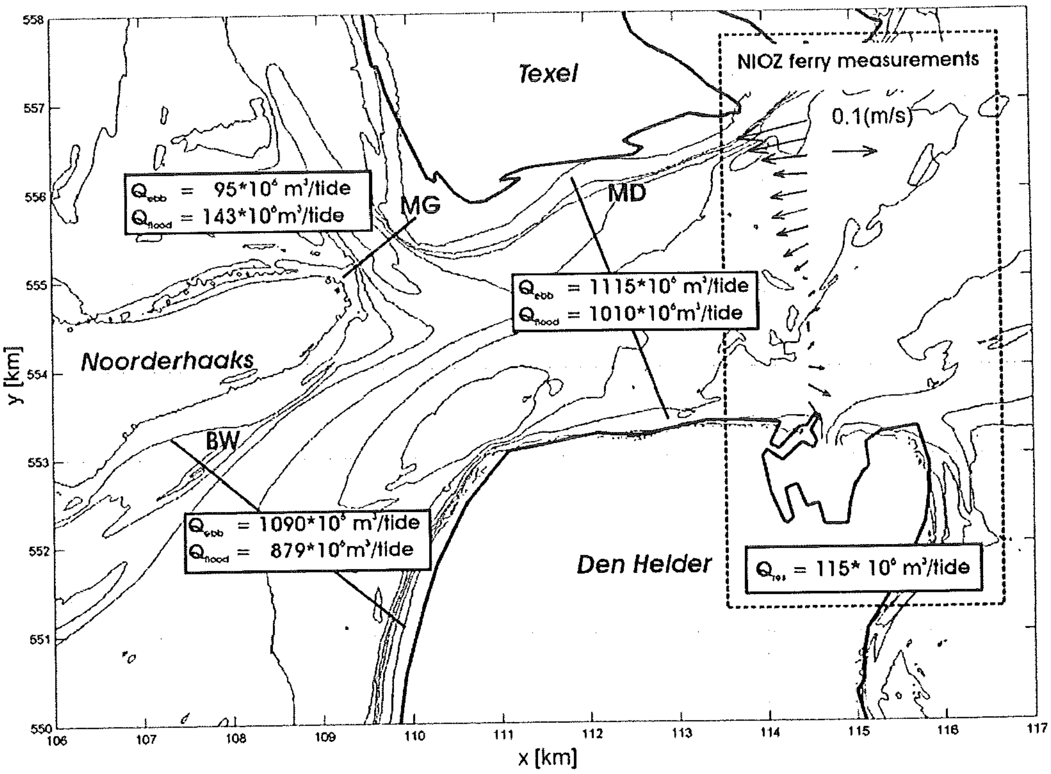

Figure 3.4: Overview of tide averaged discharges in the main channels: Marsdiep (MD), Breewijd (BW) and Molengat (MG). Based on Rijkswaterstaat measurements (lines indicate the sailed transects) and NIOZ-ferry data (1999).

In 1998 the Royal Netherlands Institute of Sea Research (NIOZ) started a long-term high-frequent measuring campaign of e.g salinity, surface temperature and flow through the inlet gorge by attaching a 1.5 MHz Acoustic Doppler Current Profiler (ADCP) to the hull of the ferry from Den Helder to Texel island. During day-time the ferry (ms Schulpengat) that sails the 4.5 km wide Marsdiep every 30 minutes. The raw data are transmitted to NIOZ and processed. Preliminary results of the flow observations have been made available by NIOZ for analysis (Bonekamp *et al.*, 2002; Ridderinkhof *et al.*, 2002).

Flow data have been used to calculate time-series of depth-averaged flow at eighteen equidistantly distributed aggregation points. Harmonic analysis has been applied to these time-series to derive tidal mean flow conditions and amplitudes and phases of the main tidal flow constituents. The residual flow through the inlet of about 115 million (M) m^3/tide is seaward directed (Bonekamp *et al.*, 2002; Ridderinkhof *et al.*, 2002). An important characteristic of the Texel basin is that it does not form a closed system but the basin connects to the neighboring Vlie basin allowing an exchange between the two systems (Fig. 3.1). The residual flow mainly results from throughflow from the Vlie to Texel Inlet, due to the higher amplitude of the vertical tide in the Vlie basin, and tidal residual eddies (Ridderinkhof, 1988).

In the spring of 2001, additional vessel-based ADCP-measurements (13-hour observations) were performed by Rijkswaterstaat (Blok and Mol, 2001). The survey included the simultaneous measuring of velocities through the Marsdiep inlet gorge and the connecting Breewijd and Molengat main channels on the ebb-tidal delta; for transect locations see Fig. 3.4 (note that Breewijd is the name for the combined upper parts of Schulpengat and Nieuwe Schulpengat). Measurements in Marsdiep (a) and Breewijd (b) were executed with a 1200 kHz system of the Broadband type. Molengat (c) was measured at a 600 kHz frequency. Transects were run obliquely on the channel axis, and continuously over a tidal cycle. The raw-data were thoroughly analyzed to remove systematic effects and data outliers, and transect averaged discharges were calculated (Blok and Mol, 2001).

The majority of flow is directed from Marsdiep to Breewijd (~90 %), with only a minor contribution of flow through the Molengat (~10 %). Averaged over the cross-section, Molengat is flood dominated. In Breewijd an internal dominance of ebb flow along the Noorderhaaks and Texel coastline over flood flow along the North-Holland coastline occurs, similar to Marsdiep. The larger ebb surplus in Breewijd compared to Marsdiep indicates a direct exchange of flow between Molengat and Breewijd. Measurements are believed fairly representative for tidal flow as wind and waves were absent.

Transports

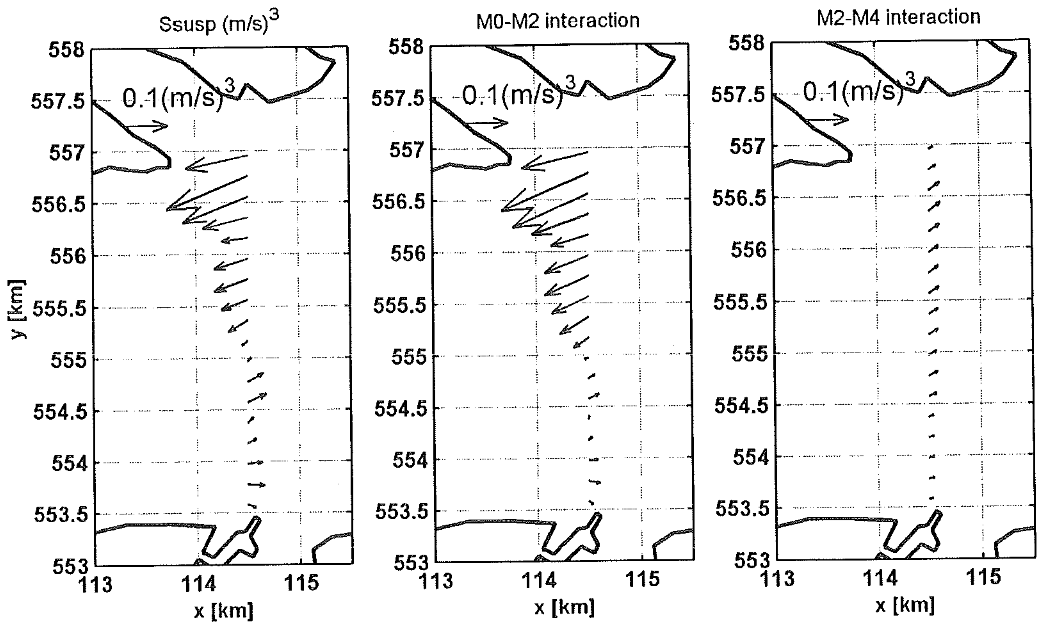

Figure 3.5: Tidally averaged suspended-load sediment transport vectors based on the Groen (1967) transport formulation for harmonically analyzed NIOZ-ferry data (after Bonekamp *et al.* 2002). From left to right: transport vectors for $M_0M_2+ M_2M_4$, M_0M_2, and M_2M_4.

The direction of residual sand transport through Marsdiep and the governing mechanisms for these transports have long been a matter of debate. The large sedimentation in the basin since closure indicates that sediment import into the basin must have occurred through Marsdiep. Previous studies all confirmed that after the closure of the Zuider Sea, large-scale sedimentation occurred in the basin. Studies by Sha (1986a), Dronkers (1986), Dronkers (1998), Louters and Gerritsen (1994), Ligtenberg (1998) and Elias *et al.* (2003) point to a net import of sediment in the order of 1 to 3 Mm^3/year. However, conclusions about the present morphologic state of the basin and the governing mechanism for sediment transport through Marsdiep are more ambiguous.

Dronkers (1986) and Ligtenberg (1998) calculated a net sediment import due to tidal asymmetry. Bonekamp *et al.* (2002)present a qualitative estimate of the tide-driven net sediment transport in Marsdiep using the harmonically-analyzed NIOZ-ferry flow data and a simple sediment transport formulation (Groen, 1967). These authors found a depth-averaged seaward-directed (ebb-dominant) sand transport in the northern part of the inlet and a smaller landward-directed (flood-dominant) transport in the southern part, due to the main tidal asymmetries. The interactions M_0-M_2, and M_2-M_4 were considered in detail (see Fig. 3.5). Since the M_2 flow constituent is dominant over its overtides in first order, the long-term residual sediment transport essentially depends on the interaction of the fundamental constituent and the Eulerian mean current and the interaction of the fundamental constituent and its even over-tides (Van De Kreeke and Robaczewska, 1993). Not shown are the flood-dominant second-order contributions of the triple interaction between M_2, M_4 and M_6 being of near-equal magnitude to the M_2-M_4 interaction.

The work of Bonekamp *et al.* (2002) suggests that the tidal mechanisms cannot explain the observed sediment import into the basin. Possible explanations for this discrepancy include:

- Not the tidal mechanisms (e.g. tidal asymmetry), but the non-tidal (e.g. wind- or wave-driven) mechanisms dominate the net sand transports through the inlet.
- Depth-averaged velocities in the inlet gorge are not representative for the near-bed currents. Observations (see Blok and Mol, 2001) show that during periods of major fresh-water discharge in the basin, secondary circulations due to density differences augment inward flow in the near-bed region.
- The use of harmonic components probably underestimates extreme values. The non-linear dependency of sediment transport on currents may cause relatively large discrepancies if harmonic components are used.
- Sediment concentration dissimilarities during ebb- and flood flow; during flood sand is transported from the ebb-tidal delta where e.g. wind and waves can be important for bringing sediment into suspension and sediment-rich flow is directed into the basin. In the basin wind and waves are smaller and less effective in bringing sediment into suspension; sediment-poor flow is exported. Such concentration differences can possibly lead to sediment import of sediment despite the pre-dominant seaward flow.

3.4.3 Analysis of Bathymetry

The recent ADCP measurements provide a detailed representation of the flow and transports through the inlet gorge and the upper main channels. Unfortunately, no flow or sediment transport measurements are available in the lower domains of the ebb-tidal delta. However, ebb-tidal deltas usually contain typical elements, such as ebb- and flood channels, channel margin linear bars, a terminal lobe and swash-bars. The distribution, shape, and size of these elements, and the analyses of smaller-scale bed form patterns can provide useful insights in the partitioning of flow and sediment transports, see e.g. Hayes (1975), Hine (1975), Hubbart *et al.* (1979), Boothroyd (1985), FitzGerald (1996) and Kana *et al.* (1999).

Echo-sounding based bathymetric surveys of the ebb-tidal delta have been performed every three years since 1991 (digitized measurements are available since 1925, although less frequently measured, see De Kruif (2001)). Recently Elias *et al.* (2003; 2005b) have used these maps to analyze the long-term changes in ebb-tidal delta behaviour. In this study we focus on the recent morphological developments using the bathymetric maps for the years 1986 till 2000. The sedimentation – erosion pattern of the Texel ebb-tidal delta, derived by subtraction of the 1986 bathymetry from the 2000 bathymetry, summarizes the recent morphological changes (Fig. 3.6).

The interaction of tidal, wind and wave-driven flow with the compound ebb-tidal delta bathymetry produces a complex pattern of mutually linked sedimentation-erosion patterns. The asymmetrical shape of the ebb-tidal delta, the larger scale bedforms and bathymetric features, and the updrift-oriented main-ebb channels point to the partitioning of the ebb-tidal delta in a northern and a southern sub-domain (Elias *et al.*, 2003). The northern sub-domain consists of the sub- and supra-tidal Noorderhaaks swash-platforms where the morphologic developments are dictated by wind and waves. In the southern sub-domain the tidally-driven processes prevail. Such partitioning of tide-generated flow, dominating in channels, and wave-driven flow prevailing over the shallow platforms is characteristic for ebb-tidal deltas (Hine, 1975).

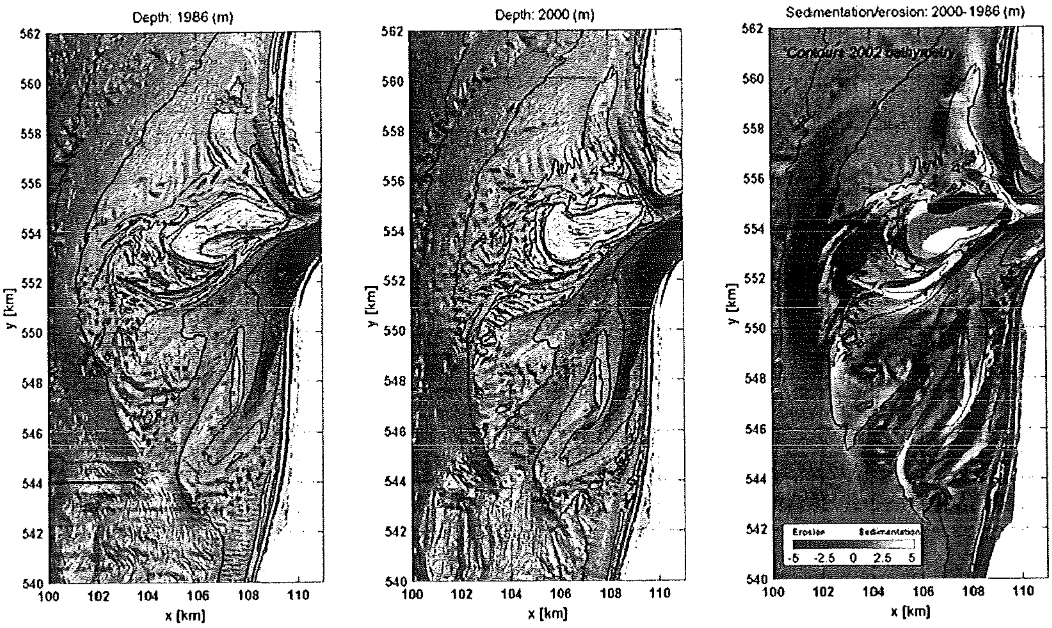

Figure 3.6: Bathymetry of the Texel ebb-tidal delta in 1986 (left), 2000 (middle) and sedimentation-erosion pattern (right).

The largest erosion is observed on the seaward margin of Noorderhaaks due to a re-distribution of sediment in landward direction. The majority of the Noorderhaaks deposits originates from the pre-closure ebb-tidal delta configuration, wherein the main-ebb channel was westward orientated. Noorderhaaks was formed as an ebb shield due to seaward ebb-tidal transports and landward wave-driven transports (Elias *et al.*, 2005c). As a result of the closure-induced updrift development of the main ebb channels, the pre-closure ebb-tidal delta deposits have been reworked. The major closure-induced effects on the ebb-tidal delta and in the basin appeared in a time span of approximately 40 years after closure. Nowadays, smaller rates of erosion still prevail on the western margin of the shoal. Onshore directed wind-driven flow, wave-asymmetry and wave-breaking contribute to these landward transports resulting in e.g. accumulation of sand in the former Westgat and spit formation at the western margin of Noorderhaaks. Alongshore transports due to contraction of tidal flow around the ebb-tidal delta and residual northward directed wind-driven transports redistribute sand to the downdrift lower-shoreface. The sedimentation-erosion patterns, the northward spit developments, the ripple formation and the large number of saw-tooth bars separated by runnels along the seaward northern part of the Noorderhaaks shoal all testify to the wave dominant character of this area.

South of Noorderhaaks the morphodynamic developments are dominated by the presence of the main tidal channels. Although the size of the channels points to large velocities and transport rates, morphological changes in this part of the ebb-tidal delta are relative small. The main developments are: (1) an increasing depth of the channels Nieuwe Schulpengat, Schulpengat and Nieuwe Westgat, (2) a seaward and southward outbuilding of Zuiderhaaks, and (3) changes in Nieuwe Schulpengat and associated ebb-shield due to a small anti-cyclonic rotation and migration of the channel (Elias and Cleveringa, 2003).

3.4.4 Detailed bed-form analysis

Detailed impressions of the dominant transport vectors in Nieuwe Schulpengat, Nieuwe Lands Diep and along the southern margin of Noorderhaaks are obtained through bed-form analysis of multi-beam data. In the autumn of 2002 high-resolution multi-beam surveys were performed (Rab, 2003). After thorough analysis of the recorded data, removing systematic effects and data outliers, a high resolution map of approximately 11 km^2 of seafloor bathymetry was obtained (Fig. 3.7). Nieuwe Schulpengat extends in southward direction, diminishing in depth and curving seaward. The shallow Nieuwe Lands Diep is located landward of Nieuwe Schulpengat along the North-Holland coastline, see Fig. 3.1 for location.

Various studies indicate the direct link between the bedform morphology (viz. size and orientation of ripples), and tidal dominance and flow magnitude (e.g. Boothroyd and Hubbard, 1975; Hine, 1975; Boothroyd, 1985; Ashely, 1990; Lobo *et al.*, 2000). Assuming that the bedforms are created by and in equilibrium with present-day hydrodynamic conditions, the bedform distribution, arrangement and morphology provides information about bottom currents (Boothroyd, 1985). Slipface orientations of large-scale bedforms only, classified as sandwaves and large megaripples or dunes (Boothroyd, 1985; Ashely, 1990) having wave lengths over 50 m and wave heights over 0.5 m, have been determined from bathymetric cross-sections of the bedforms taken perpendicular to the crest. These large bed-forms have long response times, only undergoing minor changes in size and shape when imposed to high-frequency processes such as waves and neap/spring cycles. For that reason, they provide indications of the long term (averaged over periods of several days to months) dominant transport directions. The arrows in Fig. 3.7 indicate slip-face orientations of the major bed-forms.

The margin of Noorderhaaks is dominated by smaller scale bed-forms super-imposed on several larger bed-forms. These largest bed-forms point to an ebb-dominant transport direction (see insert in Fig. 3.7). Sand waves up to 4.25 m in height, having wavelengths over 200 m are observed in the deeper parts of Nieuwe Schulpengat (below -20m) where the highest tidal velocities occur. Up to a certain treshold, the size of the bed form increases with the current velocity. In addition, the existence of coarser sediments in the deeper part of the channel and the channel depth itself allow for the formation of these bed-forms. Slip-face asymmetries point toward a predominantly flood-orientated transport.

In southward direction, as tidal flow velocities diminish and the depth of Nieuwe Schulpengat decreases, the dominant sand waves are smaller in height (maximum of 2 m) and length (up to 100 m). Similar small sand waves are also observed in the shallow region along the coastline and in Nieuwe Lands Diep. The slip-face orientations suggest a net flood dominated sand transport along the coastline and in Nieuwe Lands Diep. In the seaward part of Nieuwe Schulpengat, sand waves are smaller in height, ranging between 0.5 and 2.0 m, and wave lengths vary between 50 to 110 m. In contrast to the nearshore area, the slip-face orientations of the dominant sand waves indicate a net ebb-dominant sand transport.

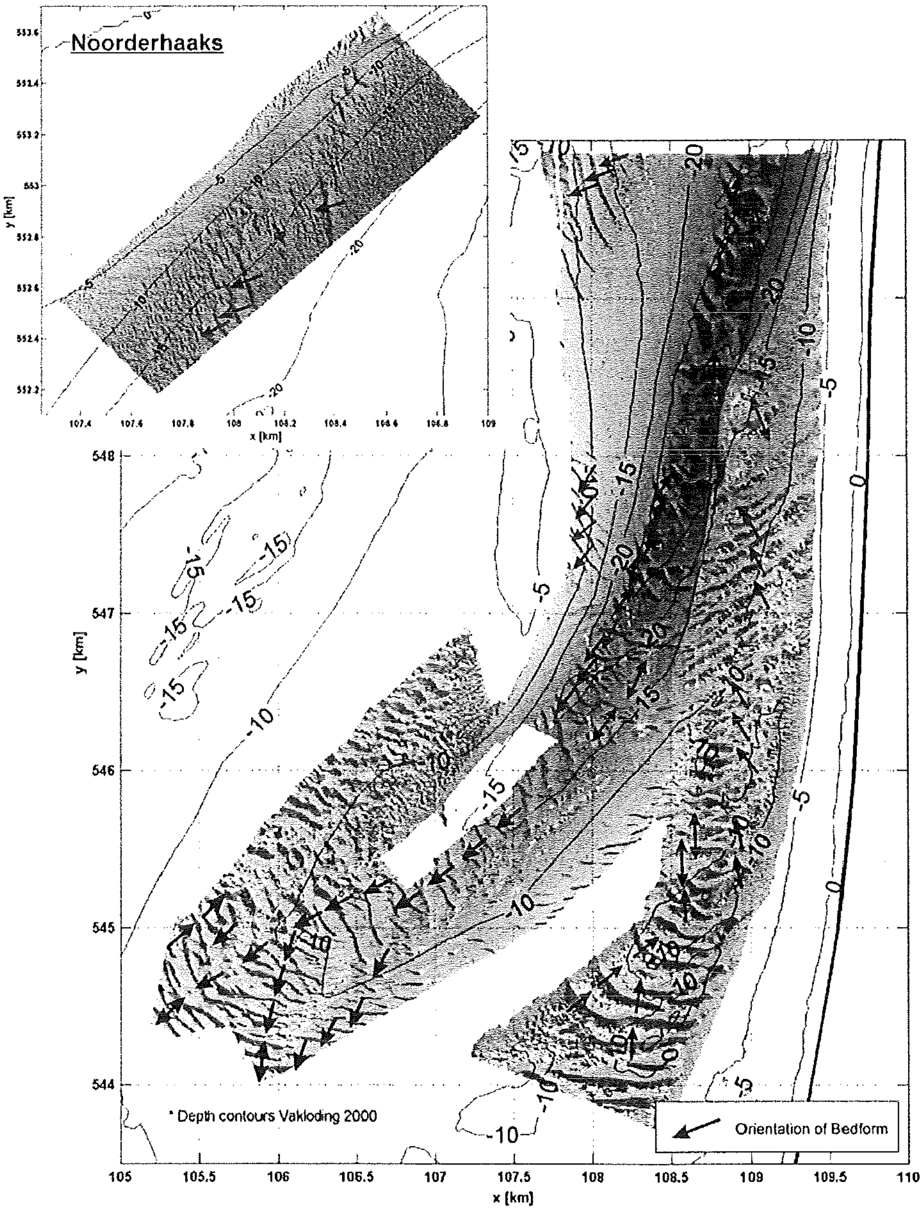

Figure 3.7: Bathymetric map based on multi-beam measurements (arrows indicate dominant slip-face orientation of major bedforms). Left insert shows: impression of the bedforms along the margin of Noorderhaaks (location see Fig. 3.1).

3.4.5 Summary: Observation-based sand transport model for Texel Inlet

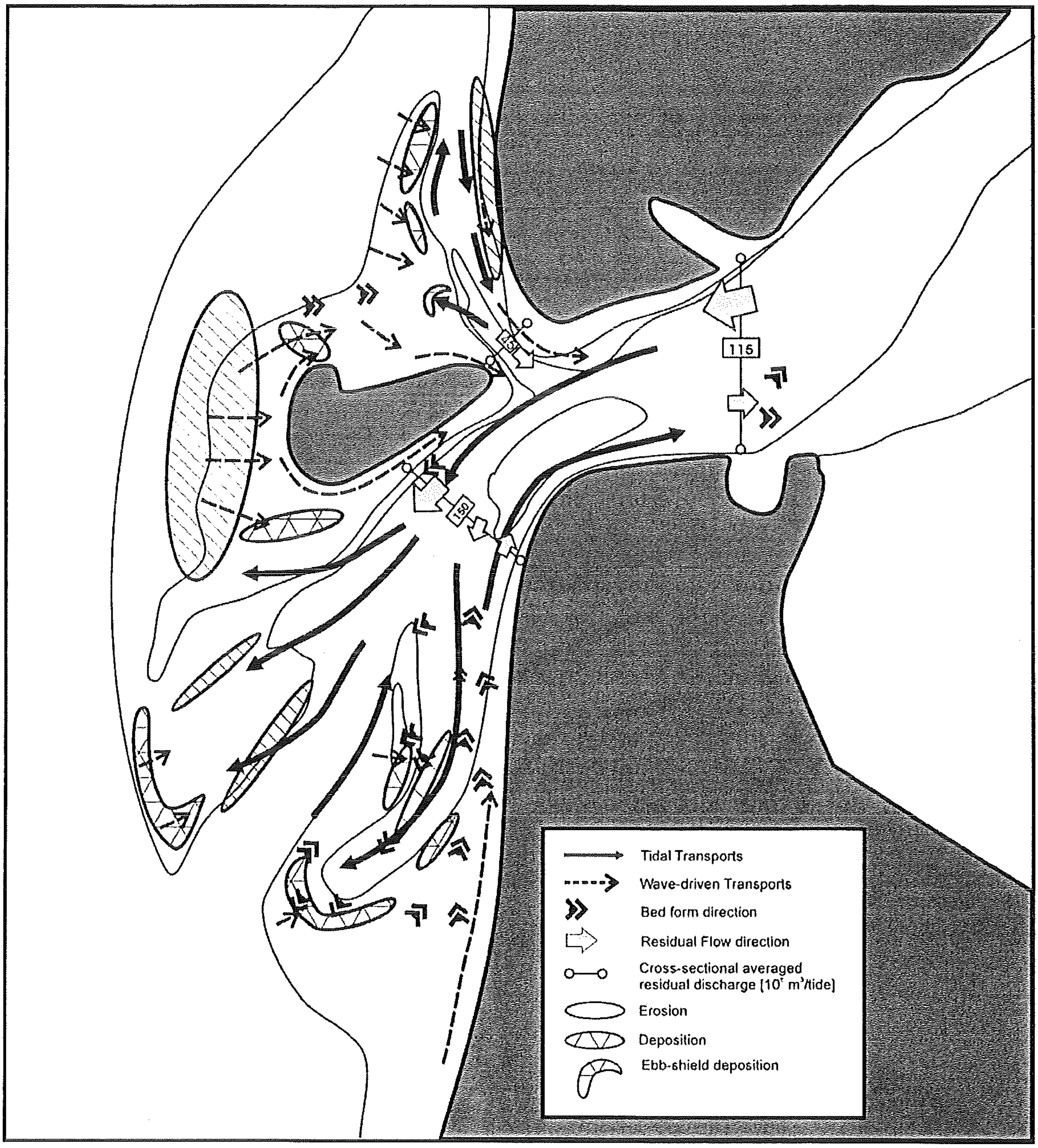

Figure 3.8: Observation-based schematic representation of transport patterns on Texel ebb-tidal delta.

Based on the analysis of flow, bed-forms, sedimentary structures and sedimentation-erosion patterns a schematic sand-transport pattern is presented in Fig. 3.8. The overall patterns correspond reasonably well with the schematic sand-transport model of Sha (1986a). The main differences are observed in the ebb-dominant transport in major parts of the Molengat and Schulpengat channel.

Morphologic developments in the northern wave-dominated domain are governed by a landward redistribution of sediment. Mainly wind- and wave-driven currents transport sand from the seaward margins of Noorderhaaks landward onto the large cuspate spit Noorderlijke Uitlopers van the Noorderhaaks. The major part of the Noorderhaaks deposits is transported along the supratidal margins of Noorderhaaks back through Marsdiep into the basin and directly back onto the ebb-tidal delta. The small ebb-chute and ebb-shield formed on the spit indicate that ebb-tidal transport through Molengat contributes to the formation of the spit. It is assumed that northward sediment transport along the landward margin of the spit retards onshore migration by the wave-driven transports, prohibiting a rapid landward migration of the shoal and extending the spit northward. With diminishing tidal velocities in northward direction, the spit curves towards the Texel coastline. The structural erosion of the Texel coastline points to the loss of sediment due to wave-dominated transports along the Texel coastline into the inlet circulation.

South of Noorderhaaks, tidally-driven flow and transports dominate. Bedform asymmetry suggests prevailing flood-dominant transports along the North-Holland coastline through Nieuwe Lands Diep and the upper part of Nieuwe Schulpengat and Marsdiep into the basin. Ebb-tidal currents redistribute sediment from the basin through the main ebb-channels back onto the ebb-tidal delta. Two main transport pathways have been identified:

- sediments are transported via Marsdiep along Noorderhaaks, through Schulpengat and Nieuwe Westgat onto the Zuiderhaaks. The absence of an ebb-shield facing Schulpengat and the seaward outbuilding of Zuiderhaaks testify to the seaward diversion of ebb transports onto the shoal.
- separated by the Bollen van Kijkduin, the Nieuwe-Schulpengat forms a second transport path. Despite the large flow velocities and transport rates in the adjacent channels; the Bollen van Kijkduin has remained remarkably stable through time. Possibly this stability is related to the recirculation of sediment by evasive ebb and flood transports (Van Veen, 1950; Hibma *et al.*, 2003). Ebb-directed tidal transports distribute sediments through Nieuwe Schulpengat onto Franse Bankje. On Franse Bankje wave breaking induced flow augments flood transports along the seaward margin of the Bollen van Kijkduin shoal, thereby forming a sediment gyre. The location of the Nieuwe Schulpengat channel along the coastline limits the amount of sediment that is transported back onto the coast; a structural sand loss from the North-Holland beaches is observed (Elias and Cleveringa, 2003).

3.5 MODEL DATA ANALYSIS

3.5.1 Introduction

Despite the intensive monitoring of Texel Inlet the spatial and temporal coverage of field observations over the inlet domain remains fairly restricted. Therefore the conceptual sand transport model (Fig. 3.8) is for a major part founded on expert-judgment based interpretations of the available data. One method to obtain synoptic data with sufficient resolution over the entire inlet domain is by numerical simulations using well-validated process-based models. Model studies by e.g. Ridderinkhof (1988), Wang *et al.* (1995), De Vriend and Ribberink (1996) and Cayocca (2001) have shown that nowadays such models can be used successfully even in the complex environments of tidal inlet systems. These previous studies tended to focus on schematized simulations of long-term inlet behaviour and morphological change using a more-or-less 'traditional' approach. The morphological model consists of a number of coupled modules that compute waves, currents, sediment transport and bed-level changes sequentially (e.g. De Vriend and Ribberink, 1996). By applying larger time-steps for the computation of bed-levels then for the computation of flow and sediment transport, morphodynamic simulations over larger time spans could be made. As the frequency of bed-level update is typically in the order of (several) tides to days, these models were forced by representative steady boundary conditions rather than randomly varying input conditions (e.g. Latteux, 1995; De Vriend and Ribberink, 1996; Cayocca, 2001). The construction of representative conditions, but also the correct coupling of the various modules made such models simulations complicated and results sometimes unpredictable. This might be one of the reasons why still only a limited number of process-based model applications have been conducted for complex areas such as tidal inlet systems.

Recently, the computation of sediment transport and bed level change has been fully integrated in the flow module (Delft3D Online Morphology model, Lesser *et al.*, 2004). As morphologic changes are calculated simultaneously with the flow calculations, this allows for the model to be forced as-realistically-as possible (quasi real-time by measured time-series of wind, waves and water levels). In this manner, synoptic 'more or less realistic data' of high spatial and temporal resolution is obtained over the entire inlet domain. Analysis of this data provides valuable information on governing flow and sediment transport patterns in the instrumented and the uninstrumented areas.

3.5.2 Model set-up

The Texel Outer Delta (TOD) model application contains the inlets of Texel, Eierlandse Gat, Vlie and adjacent coastlines. The Eierlandse Gat and Vlie inlet are included in the model domain to enable the simulation of the important internal residual volume transport between Vlie and Texel Inlet. The well-structured, orthogonal curvilinear grid has 38311 points, with a maximum resolution of 80x120 m at the location of Texel Inlet (Fig. 3.9). The North-Holland coastline, the landward coastline in the back-barrier basin, and the island coastlines form closed boundaries (free-slip conditions). The northern basin periphery is chosen on the Terschelling tidal watershed and set as a zero-velocity boundary. Open-sea boundaries are located "far away" outside the sphere of Texel inlet's influence and prescribed as water level elevations.

The bed topography of the nearshore, the inlets and the basin is based on 1997 bed topography measurements (De Kruif, 2001). In the inlet gorge and near-field bed-levels are updated using 2001, 2002 and 2003 observations to obtain an as accurate as possible bottom schematization. Depths in the deeper region are based on Dutch Continental Shelf data supplied by TNO-NITG (Frantsen, 2001). Depending on the resolution of available observations, depth measurements were triangularly interpolated or grid-cell averaged to the curvilinear grid.

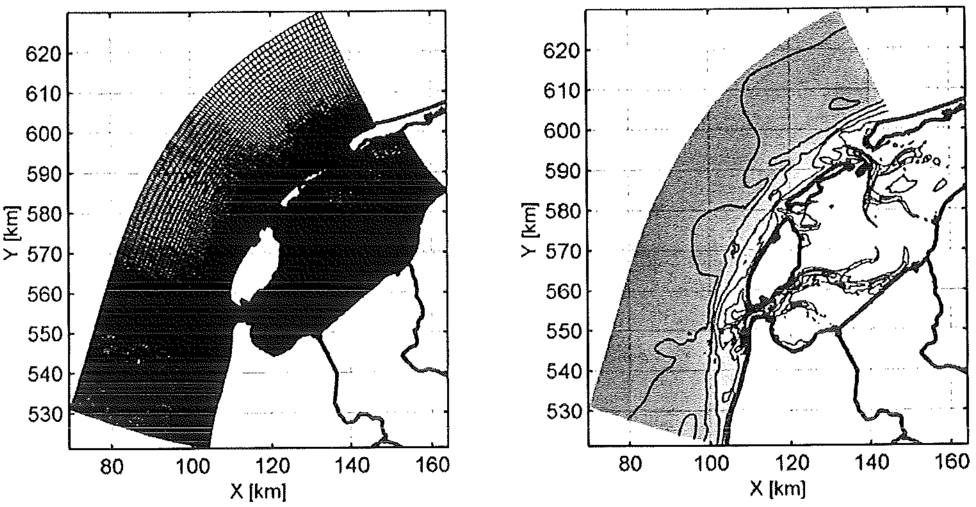

Figure 3.9: Delft3D model grid and bathymetry

Bottom roughness is prescribed by a global Chézy coefficient of 61.5 $m^{1/2}/s$. Enlargement of the bed shear stress due to waves is included using the formulation of Fredsoe (1984). The time step for the flow computations is 60 seconds to fulfill the maximum courant number criterion of 15. Default settings of 1.0 m^2/s and 0.10 m^2/s for the uniform horizontal eddy viscosity and eddy diffusivity coefficients have been applied respectively. The secondary flow option has been used to take into account spiral flow due to stream-line curvature. Computations start from a uniform water level. A two-day spin-up prior to the actual computations is sufficient to dissipate the errors induced by the discrepancy between boundary conditions and initial state.

Quasi real-time model simulations are forced additionally by wind stresses on the free surface based on hourly data of a general wind climate, derived from measurements of station De Kooy near Den Helder (Fig. 3.10 c and d). Waves are forced by spatially constant, time-varying series of wave heights, periods and directions (Fig. 3.10 e and f). These time-series are based on the measurements of the Eierlandse Gat wave buoy and prescribed on the open-sea boundaries using a Jonswap shape of the wave spectrum with a peak enhancement factor of 3.3. Wave propagation, growth and decay due to generation of waves by wind, the dissipation due to whitecapping, bottom friction (Jonswap coefficient of 0.067 m^2s^{-3}), depth induced breaking (Battjes and Jansen breaker criterion of 0.73), and non-linear quadruplet and triad wave-wave interactions (Lumped Triad Approximation with a = 0.1 and b = 2.2) are solved hourly on a subset of the flow grid. The SWAN wave model has been extensively verified in the Dutch coastal area (e.g. the Haringvliet, the Petten sea defences, the Friesche Zeegat and recently the Flyland studies) and worldwide (see e.g. Ris, 1999; Roelvink *et al.*, 2001a; Lin *et al.*, 2002; Rogers, 2003). These studies have proven the SWAN model's capability of reproducing wave-heights and period distributions, even in the complex areas such as inlet systems.

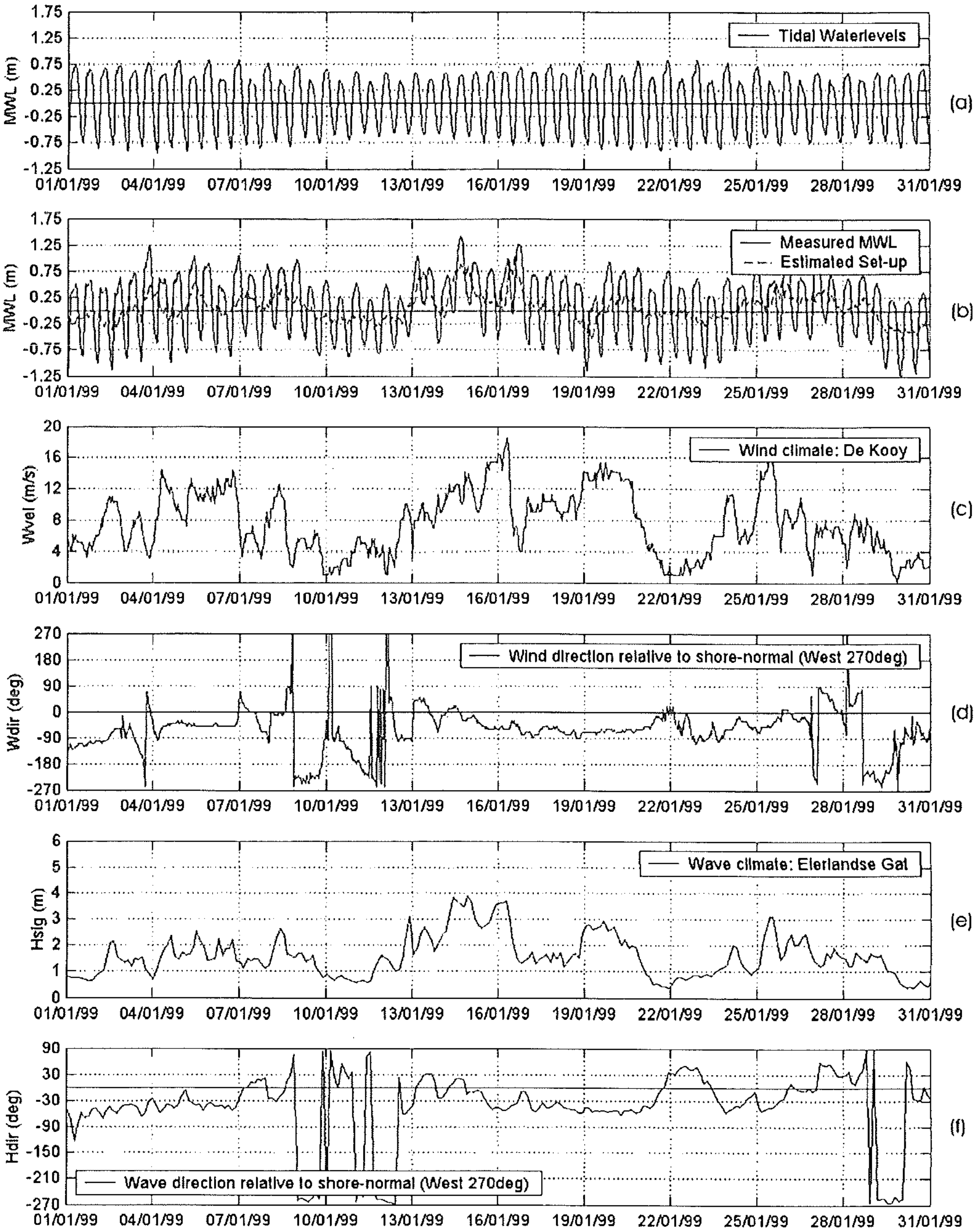

Figure 3.10: From top to bottom: (a) Tidal water levels predicted from harmonic components and (b) observed water levels station Den Helder, (c) wind velocities and (d) directions station De Kooy Den Helder, (e) significant wave heights and (f) directions station Eierlandse Gat, measured period 01-01-1999/01-04-1999.

Default settings for the calculation of sediment transports and morphological change are used (Van Rijn 1993 transport option). Sediment transport computations are initiated after a 1440 minutes spin-up time. A morphological scale factor of 1 is applied; no morphological acceleration, see Lesser *et al.* (2004). Bottom sediments characteristics for the morphological computation include: the use of a single-fraction of non-cohesive sand (d_{50}= 250 μm), no hindered settling, a sediment density of 2650 kg/m^3, a dry bed density of 1600 kg/m^3 and an initial sediment mass at the bed of 16000 kg/m^2.

To obtain feasible computational time only depth-averaged simulations have been made under the assumption that, on the ebb-tidal delta, water is vertically well-mixed and 3D effects are only of minor contribution to the large-scale sand transport patterns (based on De Vriend and Ribberink, 1996)). Local model results are therefore less accurate in areas where 3D currents are important, e.g. shallow shoal areas where wave-induced mass flux and undertow play a role or areas where secondary circulations due to e.g. stratification in periods with major fresh-water discharge develop. The importance and effects of stratification on the modelled sand transport rates and patterns is subject of ongoing research. Even with a depth-averaged assumption, computational time limits the quasi real-time simulations to a period of three months during the winter and spring of 1999 (01-01-1999 / 01-04-1999).

3.5.3 Validation:

Representation of long-term tidally-driven flow

As a first validation step the tidal flow through Marsdiep has been simulated and validated against the harmonically-analyzed NIOZ-ferry flow data and the Rijkswaterstaat flow measurements (see section 4.2). Using a tidally driven TOD-model set-up, flow through the inlet gorge is simulated over a 3-month period (01-01-1998 / 01-04-1999). The open-sea water-level boundaries are prescribed by representative harmonic constituents (frequencies, amplitudes and phases) of the water-level elevations. Tidal boundary conditions have been derived from nesting the TOD-model in the larger-scale PROMISE model that describes the tidal movement of the southern North Sea with a quasi real-time tidal height forcing at the northern boundary and in the English Channel (Gerritsen *et al.*, 2001). Time-series of simulated water levels are parsed on as a forcing at the nested boundaries. The simulated model output for the Den Helder tidal station was harmonically analyzed for water level amplitudes and phases of the main tidal constituents in the same manner as the observed water levels. Main constituents have been selected based on the importance for flow through the inlet (Ridderinkhof *et al.*, 2002). Small corrections on the amplitudes and phases of the tidal components at the open-sea boundaries have been made until a good correspondence is observed in measured and modelled amplitudes and phases (Table 2).

Table 2: Amplitudes and phases of water levels of the Texel Tidal station from observations and model results (period: 01-12-1998 / 01-06-1999)

Tidal comp.	Amplitude (m)		Phase (deg)		Tidal comp.	Amplitude (m)		Phase (deg)	
	Obs	Model	*Obs*	Model		*Obs*	Model	*Obs*	Model
O1	*0.09*	0.09	*98*	97	S2	*0.19*	0.18	*259*	259
K1	*0.05*	0.05	*29*	29	K2	*0.04*	0.05	*130*	134
2N2	*0.02*	0.04	*323*	321	M4	*0.12*	0.12	*79*	78
mu2	*0.08*	0.08	*135*	135	MS4	*0.07*	0.07	*215*	214
N2	*0.10*	0.10	*94*	98	2MN6	*0.03*	0.03	*105*	102
nu2	*0.04*	0.06	*6*	-46	M6	*0.06*	0.06	*134*	137
M2	*0.68*	0.67	*119*	119	2MS6	*0.06*	0.06	*263*	262
L2	*0.06*	0.07	*310*	306					

This model that has been calibrated and validated against tidal water levels is evaluated on the correct representation of the tidal flow through the inlet gorge. Time-series of simulated and observed depth-averaged flow and discharges are calculated at the NIOZ-ferry aggregation points. For each aggregation point the amplitude of the dominant M_2 and its first overtide M_4 , the M_4/M_2 amplitude ratio and the M_4M_2 relative phase difference $(2\varphi_{M2} - \varphi_{M4})$ are plotted. Fig. 3.11a-d shows the good correspondence between model results and observations. As in the NIOZ-ferry measurements relatively strong residual ebb velocities dominate the inlet gorge and upper main channels (Fig. 3.11e). The residual velocities range between 0.1 - 0.2 m/s whereas maximum flow velocities in the inlet are in the order of 1.75 m/s. Qualitatively, the main features of the simulated flow in the inlet gorge represent the observations reasonably well. The internal dominance of flood-flow along the North-Holland coastline and ebb-flow along the Texel coastline and landward margin of Noorderhaaks is adequately reproduced, although the modelled residual discharge of 95 Mm3/tide slightly underestimates the measured value of 115 Mm^3/tide. In correspondence with the Rijkswaterstaat flow measurements (Blok and Mol, 2001) the major part of the simulated flow is directed from Marsdiep to Breewijd with only a minor contribution of flow through Molengat. From Breewijd the major component of flow (≈ 75%) is directed via Schulpengat and Nieuwe Westgat onto the Zuiderhaaks. Nieuwe Schulpengat transports a minor part (≈ 25%) of the total residual flow. Note that the tidal residual flow patterns correspond with the dominant sand transport directions in the southern part of the ebb-tidal delta, but cannot explain the sedimentation-erosion patterns on Noorderhaaks. Hence, in the northern area the non-tidal processes (wind and waves) are the dominant initiators for sediment transport.

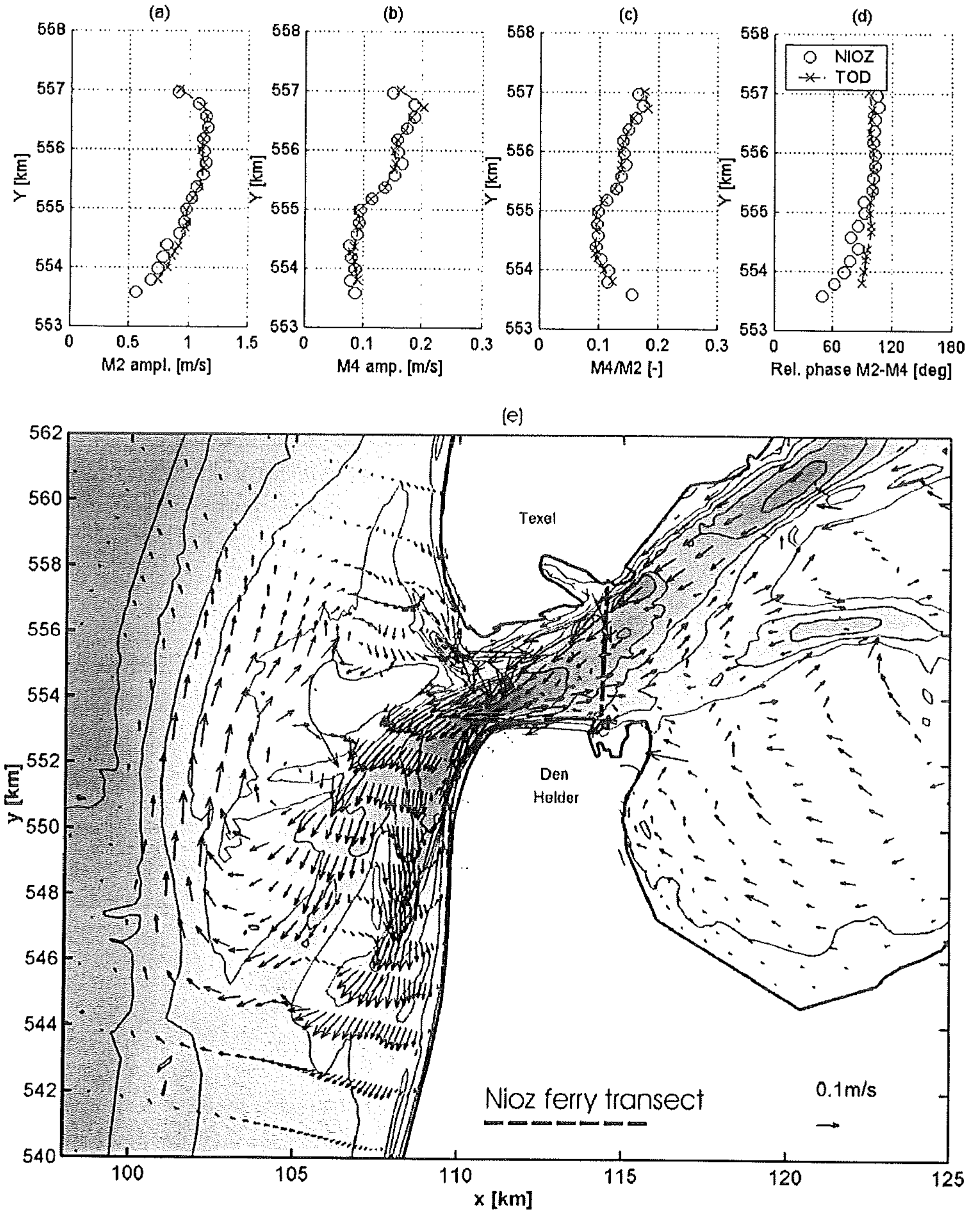

Figure 3.11: Model results of tidal flow simulation. Top: comparison of (a) M_2 velocity amplitude, (b) M_4 velocity amplitude, (c) M_4/M_2 and (d) M_4 phase relative to the M_2 for the modelled and observed tidal flow in the NIOZ-ferry transect. Bottom (e): Residual flow vector field for tidal simulation.

Quasi real-time model forcing

For as-realistic-as-possible estimates of the flow and sediment transport patterns in the inlet and on the ebb-tidal delta both tidal- and non-tidal forcing must be represented correctly. Wind effects are included via hourly data of spatially constant wind stresses on the free surface. The size of the model allows for the generation of the local wind-driven flow, but is too small to generate the larger-scale wind driven set-up, that, due to the funnel shape of the North Sea, can significantly influence the mean water levels during storm events (compare Fig. 3.10 panel a and b). The quasi real-time experiment is forced by 'measured' time-series of water level elevations on the open-sea boundaries. These time-series include the tidal and non-tidal water level elevations and have been derived from a large-scale model that was calibrated on the measured water levels of the tidal gauges along the Dutch coast via Kalman filtering (Plieger, 1999). Small corrections (~ 0.05 m) on the time-series at the open-sea boundaries of the TOD-model have been made until a good correspondence (less than 0.02 m difference) is observed in measured and modelled water levels at Den Helder tidal station. The open-sea boundaries are forced additionally by hourly time-series of wave heights, periods and directions to include wave effects. A 3-month period has been selected (01-01-1999 / 01-04-1999) wherein the wind and wave climate are reasonably representative for the long-term yearly-averaged conditions as can be seen by comparing Table 1 and Table 3.

Table 3: Morphological impact (*MI* in %) of selected wave height and direction classes over the model period 01-01-1999 / 01-04-1999.

Wave dir (dir)	H_{m0} (m)								Total	
	0-1		1-2		2-4		> 4 m			
	days	*MI*	days	*MI*	days	*MI*	days	*MI*	days	*MI*
180 - 225	3	*0*	5	*3*	3	*9*	0	*0*	11	*12*
225 - 270	9	*1*	13	*8*	6	*14*	0	*0*	28	*23*
270 - 315	11	*1*	8	*4*	4	*13*	1	*7*	24	*25*
315 - 360	4	*0*	9	*5*	6	*20*	1	*15*	20	*40*
total	27	*2%*	35	*20 %*	19	*56 %*	2	*22 %*	83	*100%*

Note that the accuracy of the simulated transport magnitudes is unknown due to the lack of field data on sediment transport rates in the area of interest for model validation. The ongoing analysis of NIOZ-ferry measurements might provide an impression of the transport of suspended matter through Marsdiep in the near-future (Merckelbach and Ridderinkhof, 2004). Such datasets would provide an indispensable tool for model validation. For the moment, the simulated transport patterns must be considered in the qualitative rather than the quantitative sense.

Sensitivity analysis of model forcing and model parameters by variations in e.g. boundary conditions, bathymetry, bottom friction and wave parameters showed that the qualitative flow and transport patterns on the scale of the ebb-tidal delta are relative insensitive to small-scale fluctuations. This provides confidence in the modelled patterns.

3.5.4 Model results.

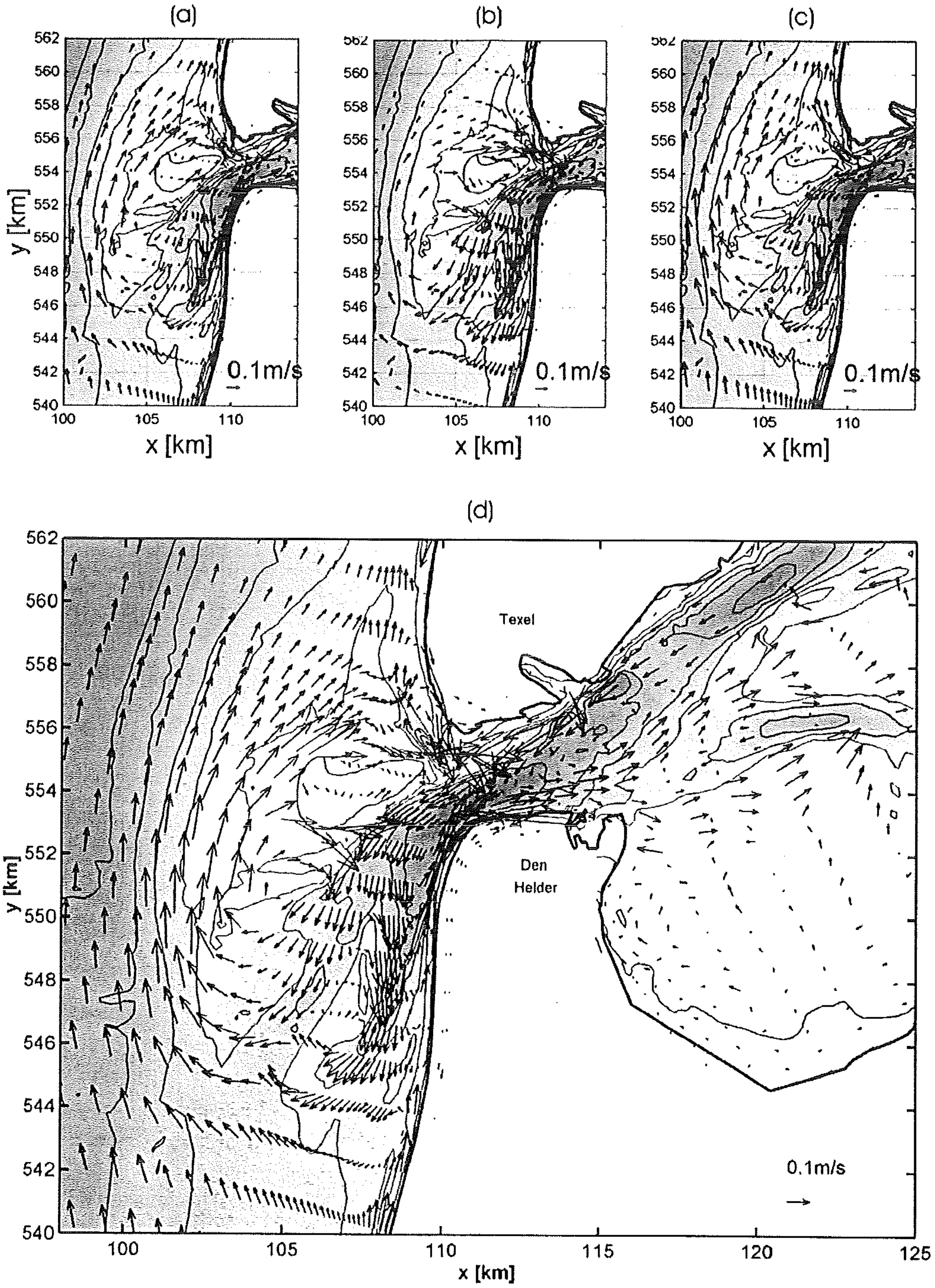

Figure 3.12: Residual flow model results for quasi real-time simulation. Top panels: One-month averaged flow for the months of (a) January, (b) February, (c) March. Bottom panel: (d) three-month averaged flow vector field (January - march 1999).

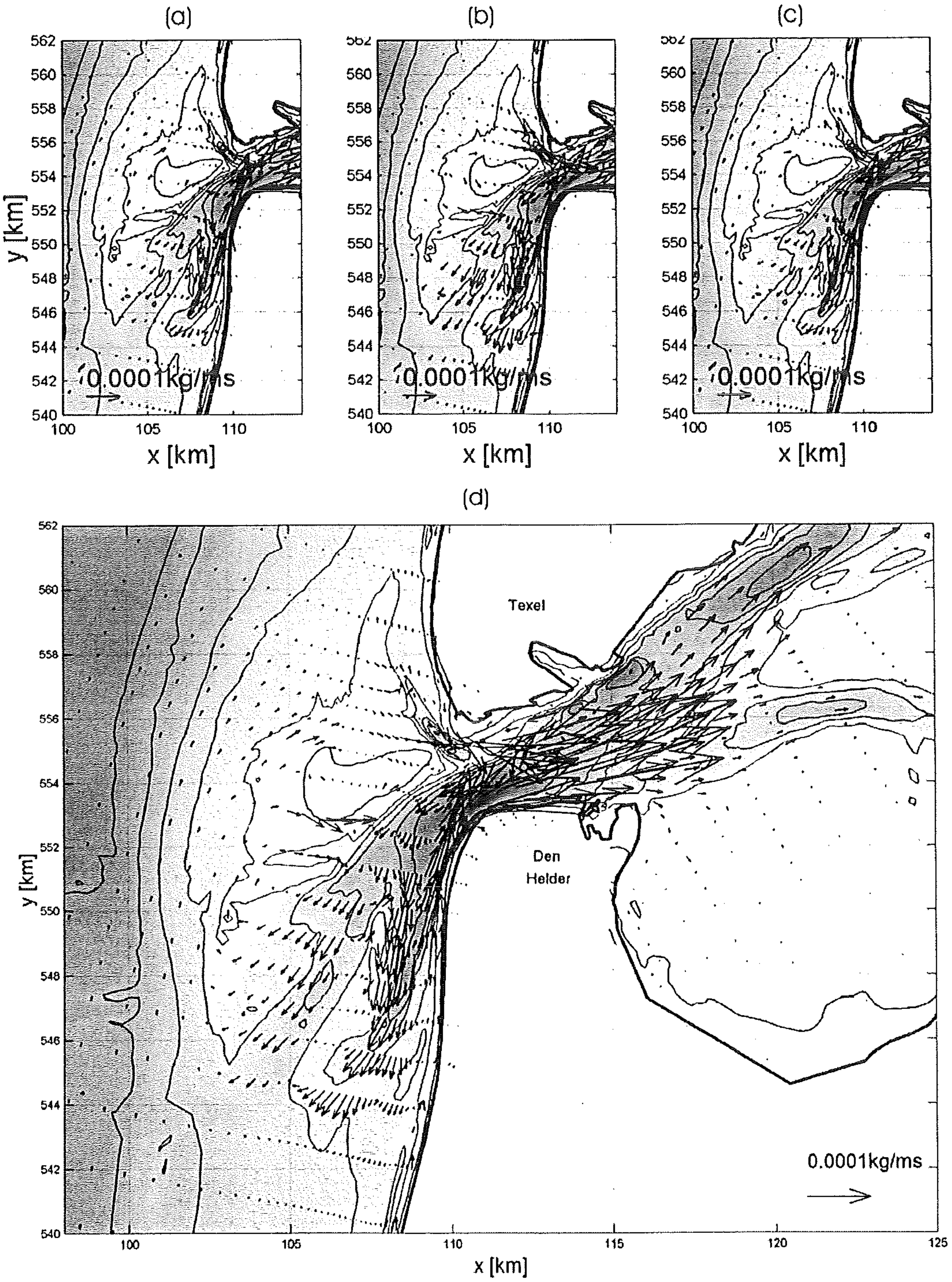

Figure 3.13: Residual transport model results for quasi real-time simulation. Top panels: One-month averaged transport patterns for the months of (a) January, (b) February, (c) March. Bottom panel: (d) three-month averaged transport vector field (January - march 1999).

Flow model results for the quasi real-time experiment are presented as 1-month and 3-month averaged residual velocities (Fig. 3.12a,b,c and Fig. 3.12d respectively). For clarity a limited number of vectors are plotted in x- and y-direction. As in the measurements (and tidal experiment) relatively strong residual ebb velocities dominate Marsdiep along the Texel coastline, while flood flow prevails along the North-Holland coastline. The majority of the ebb flow is directed from Marsdiep into Breewijd via Schulpengat and Nieuwe Westgat onto Zuiderhaaks, and via Nieuwe Schulpengat onto Franse Bankje. Outside the ebb-tidal delta domain, a residual northward current prevails that results from the cumulative effect of small contributions of flow velocities due to a minor but consistent water level setup gradient in the model (Elias *et al.*, 2004). Forcing the model with and without gradients in the model boundaries has negligible effects on the sediment transports due to the small velocity magnitudes involved.

Partly, tidal flow bypasses the inlet directly over the western margin of the ebb-tidal delta, where flow accelerates due to contraction of tidal flow around the delta perimeter. Northward flow is further enhanced by seaward diversion the alongshore flood tidal flow by of the updrift directed currents leaving the inlet. On Noorderhaaks, wave breaking augments the northward and landward residual currents. Partly, flow is directed through Molengat back into the main ebb-tidal delta circulation and partly flow is directed northward onto the Noorderlijke Uitlopers of Noorderhaaks and the Texel shoreface.

The 1-month averaged results for the months of January, February and March 1999 (Fig. 3.12a, b and c) provide an indication of the variability in the flow patterns due to variations in meteorological forcing. In the month of January (plot a) the mean wind speed and wave heights approximately equal the 3-month averaged conditions, but the northward component dominates (Knmi, 1999a). The resulting residual flow pattern shows an increased magnitude of northward flow, while southward flow is reduced. As a result Marsdiep is flood-dominated during January; the residual throughflow is directed from Marsdiep to Vlie inlet. Forcing the model by spatially uniform wind fields ranging between 0 and 25 m/s between South and North, shows that reversal of the throughflow, from Texel to Vlie, occurs for wind velocities between 5 – 10 m/s and wind directions from the south to west. The month of February (plot b) is dominated by above average wind and waves from the Northwest due to the two storm events on the 4th and 17th of February (Knmi, 1999b). Consequently, northward directed flow is retarded, while south-ward (ebb) flow is augmented. In the inlet gorge ebb-flow dominates due to the enlarged throughflow from the Vlie to Marsdiep. The above-average ebb-flow magnitudes during the storm events significantly enlarge the ebb-tidal flow velocities in the main ebb-tidal delta channels. Conditions during the month of March (plot c) are comparable to the 3-month averaged conditions (Knmi, 1999c), although flow magnitudes in the tidal channels generally are somewhat smaller.

Sediment transports (Fig. 3.13) are dominated by the suspended load component. Similar to the flow results prevailing wind and waves from the southwest augment flood and retard ebb transports, and vice versa for wind and waves from the northwest (Fig. 3.13a,b,c). The main transports (Fig. 3.13d) take place in the tidal channels, along the North-Holland coastline and along the seaward margins of the supra-tidal Noorderhaaks. Outside the ebb-tidal delta perimeter, the largest transports are observed in the upper-shoreface and surfzone area of the North-Holland and Texel coast. These transports are mainly wind- and wave-driven and directed towards the inlet. In deeper water, were waves are less important a small northward littoral drift prevails.

The majority of the northward littoral drift is transported along the North-Holland coast into the basin. In the basin sediments partly settles, but the bulk of the sediments is transported back onto the ebb-tidal delta by the ebb currents. Similar to the segregation in flow, the residual transport pattern in the inlet gorge consists of flood-dominant transports along the North-Holland coastline and ebb-dominant transports along the Texel coastline. In contrast to the flow, the flood transports exceed the ebb transports and a net import of sediment is observed (see Discussion).

The majority of the ebb transports is directed from Marsdiep onto the southern part of the ebb-tidal delta (Nieuwe Schulpengat, Schulpengat and Nieuwe Westgat). Major residual transports are observed in Nieuwe Schulpengat onto the ebb-shield Franse Bankje. Deposition of sediment onto Franse Bankje contributes to the formation and maintenance of the shoal. Wave breaking on the ebb-shield augments northward directed transports along the seaward margin and over the Bollen van Kijkduin, transporting sediments back towards the Schulpengat channel.

Ebb transports through Schulpengat and Nieuwe Westgat are directed onto the Zuiderhaaks and contribute to the northward directed transports over the western and southern margin of Noorderhaaks. A minor part of the sediments bypasses the inlet system directly onto the downdrift upper-shoreface. The majority of the sediments is transported back into the main inlet circulation along the northern and southern margins of the supra-tidal Noorderhaaks.

Along the Texel coastline residual transports are directed towards the inlet, mainly due to the absence of waves from southern directions (wave sheltering and blocking by the ebb-tidal delta). Ebb-tidal transports through Molengat deposit sediments onto the Noorderlijke Uitlopers van de Noorderhaaks spit during calm conditions (see Fig. 3.11), while wave breaking induced currents contribute to a landward transport of sediment on the spit. The relatively strong northward directed velocities through Molengat along the landward margin of the spit retards this landward migration by transporting the sediment northward. Thus sand transfer from the spit back onto the coastline is limited, which contributes to the observed structural erosion of the Texel coastline (Cleveringa, 2001). We speculate that, due to diminishing tidal velocities in the lower part of the channel, the spit curves landward.

3.6 DISCUSSION

3.6.1 On the importance of wind and waves for sediment transports in Marsdiep

The direction of residual sand transport through Marsdiep and the governing mechanisms for these transports have recently been a matter of debate. The large sedimentation in the basin since closure indicates that sediment import into the basin must have occurred through Marsdiep. Conclusions about the governing mechanism for the sediment import are more ambiguous. Sediment transports due to the main tidal asymmetries (M_0M_2 and M_2M_4) have so far been assumed to be most dominant (Dronkers, 1986; Ligtenberg, 1998; Bonekamp *et al.*, 2002). Dronkers (1986) and Ligtenberg (1998) concluded to sediment import due to M_2M_4 tidal asymmetry. While Bonekamp *et al.* (2002) found that presently the ebb-dominant tidal residual transport (M_0M_2) dominates over the flood-dominated M_2M_4 tidal asymmetry transport inducing a net sediment export.

In addition to the tidal forcing also non-tidal mechanisms (e.g. estuarine circulation, wind and waves) can contribute significantly to the sand transports in the inlet gorge (see Fig. 3.13). The present relatively large fluctuations in basin volume relative to the long-term trend (Van Marion, 1999; Elias *et al.*, 2003) possibly indicate a correlation of sediment import and export with the meteorological conditions. As yet studies into the importance of estuarine circulation for the transports in Marsdiep are ongoing. An estimate of the significance of wind and waves is obtained by sensitivity studies wherein the TOD model is forced according to three scenarios. These scenarios include: (1) water level forcing, (2) water levels + wind, and (3) water levels + wind + waves (see the quasi real-time simulations of section 5.3.2). For each of the forcing conditions simulations over a 1-month period (01-01-1999 / 01-02-1999) are made. Fig. 3.14a shows the cumulative transport rates for each of the computations integrated over the Marsdiep cross-section. Fig. 3.14b and Fig. 3.14c respectively illustrate the additional transport patterns due to the addition of wind (2 - 1) and waves (3 - 2) to the simulation.

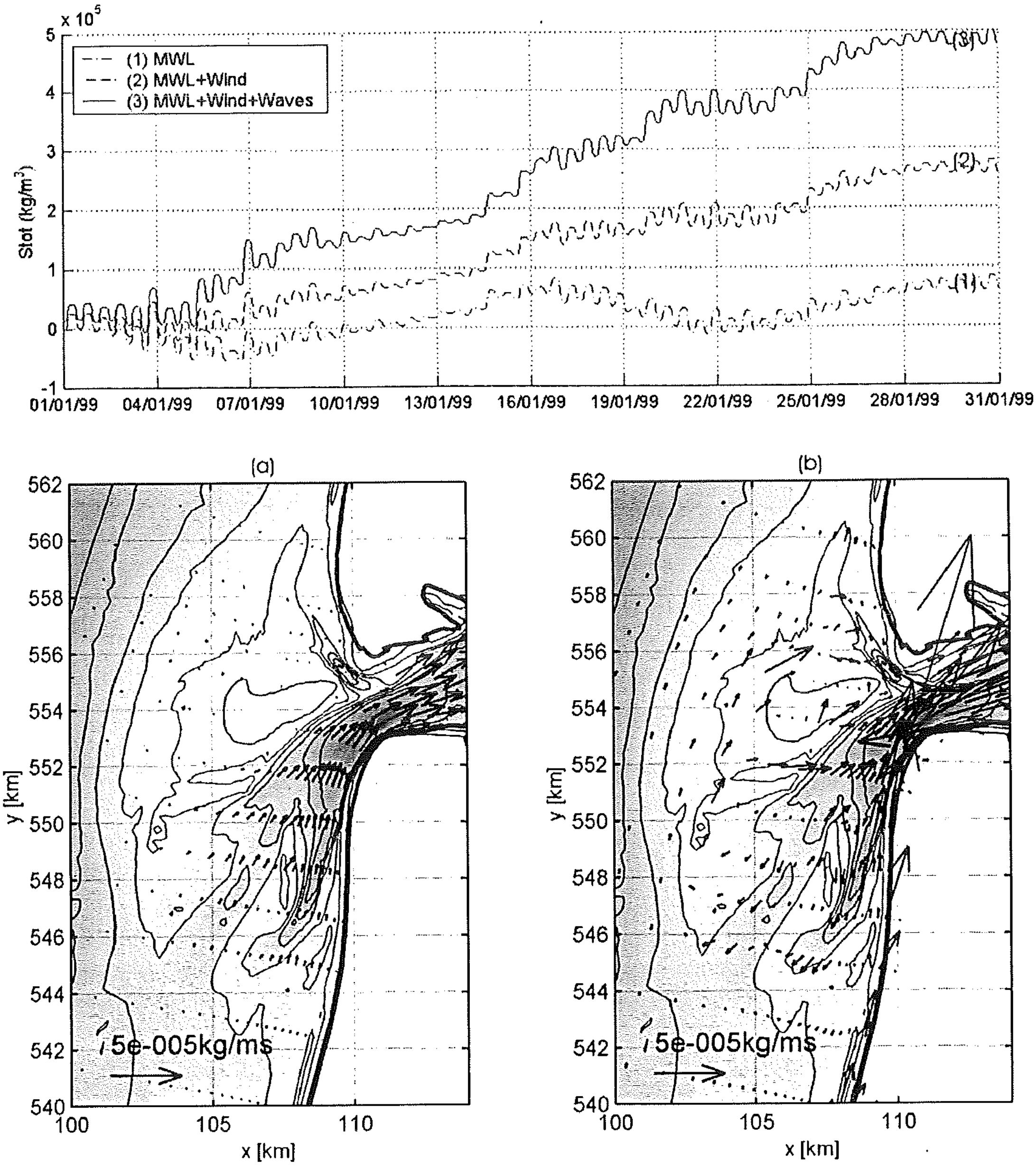

Figure 3.14: Top: total load transports averaged over Marsdiep cross-section for simulations forced by (1) water levels, (2) water levels + wind, and (3) water levels + wind + waves. Bottom panels: (a) wind-driven transports, and (b) wave-driven transports.

In our 1-month model simulation the ebb residual tidal transports along the Texel coastline near-balance the flood residual tidal transports along the Texel coastline and only a small net sediment import is observed (Fig. 3.14 top panel, 1). Observations and model results (Bonekamp *et al.*, 2002) show that these tidal transports through Marsdiep mainly result from the residual transports (M_0M_2 interaction). Tidal asymmetry driven transports (M_2M_4 interaction) are small. Note that we have not been able to calibrate or validate the model on transport rates, it is therefore possible that in reality tides induce a small net sediment export.

An important result is that sediment transports into the basin substantially increase by adding wind and waves to the model forcing (Fig. 3.14 top panel, 2 and 3). Wind contributes directly to the sediment transports due to the generation of wind-driven flow. In the month of January wind from the south-southwest dominates and a northward directed wind-driven flow prevails. This northward current augments flood and retards the ebb flow and transports (Fig. 3.14a).

Waves contribute directly to the sediment transport due to wave-breaking induced radiation stresses and tidal asymmetry. Indirectly, waves contribute by enhancing bed shear stresses and stirring-up of sediment. As a result sediment concentrations and the sediment transported by the tidal flow increase. Fig. 3.14b illustrates that this latter effect might be very important. Waves are larger (more important) on the ebb-tidal delta than in the sheltered back-barrier area and this promotes sediment import into the basin.

In conclusion, these sensitivity studies on dominant forcing mechanisms show that especially the interaction of wind and waves with the tides can contribute significantly to the sediment import into the basin. Wind and waves enlarge flood- and reduce ebb-flow and transport capacities. In the short 1-month simulation wind and waves induce a large sediment import, but also, on the longer-term (year-averaged) wind and waves are primarily landward directed (towards the inlet) and sediment import is expected. Note that this analysis is limited to improving the understanding of the dominant transport mechanisms for sand. For realistic estimates of transport magnitudes, longer term simulations and validation of the modelled transports against field measurements is needed. The long-term simulation of inlet behaviour is topic of ongoing research

3.6.2 Sand transport model for Texel Inlet

Model results (Fig. 3.12 and Fig. 3.13) provide additional evidence for the schematic sand transport patterns presented in Fig. 3.8. Both observational data and model results show that the present morphological developments on the ebb-tidal delta are dictated by sediment redistribution; sediment erodes from the western margin of Noorderhaaks and is deposited in the basin and on the southern part of the ebb-tidal delta. This sediment re-deposition probably results from a second-stage adaptation to the effects of closure of the Zuider Sea. In the conceptual model description of Elias *et al.* (2003) this second-stage development is described as a quasi-equilibrium state. The initial (first-stage) adaptation of the ebb-tidal delta to the effects of closure of the Zuider Sea lasted approximately 40 years. In this period the changed tidal characteristics of the basin forced an asymmetrical ebb-tidal delta development with updrift directed main channels. Due to the large tidal prism and corresponding large tidal transports involved, the channels regained equilibrium in a faster rate than the shoal areas (e.g. the abandoned ebb-shield Noorderhaaks). The present ebb-tidal delta development is dictated by the ongoing adjustment of these shoal areas. We have summarized the resulting dominant sand transport patterns in Fig. 3.15. For clarity we have separated pathways of sediment import, export and circulation.

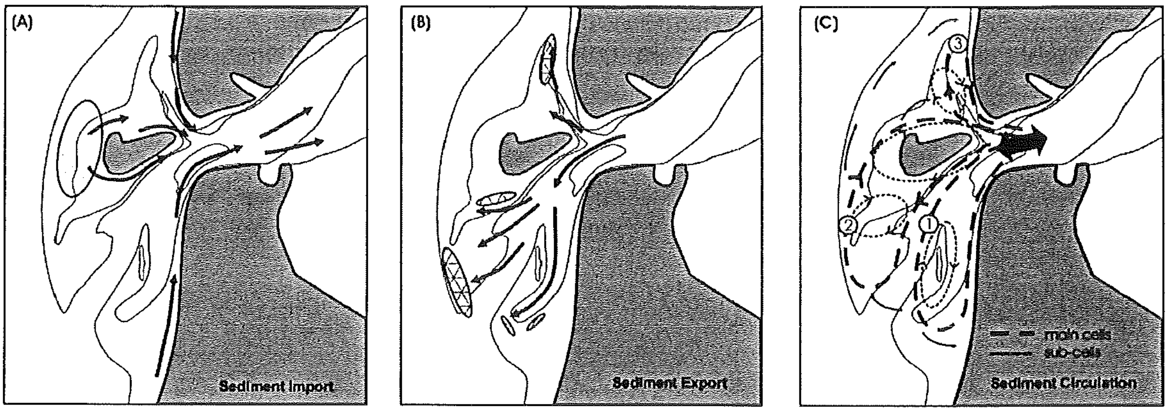

Figure 3.15: Schematic representation of transport patterns on Texel ebb-tidal delta based on observational data and model results, (a) pathways for sediment import, (b) pathways for sediment export, and (c) sediment circulation cells; 1. Nieuwe Schulpengat system, 2. Noorderhaaks system, 3. Molengat system.

Fig. 3.15a summarizes the sediment supply to the back-barrier basin by (1) the northward directed littoral drift along the North-Holland coastline, (2) southward directed transport along Texel coastline and (3) from the abandoned ebb-tidal delta front (erosion of the western margin of the ebb-tidal delta). Mainly landward directed wind-and wave-driven flow transport the eroded deposits along the margins of the supra-tidal Noorderhaaks shoal area. Model results indicate that sediment import into Marsdiep results from the cumulative transports due to tidal asymmetry, landward directed wind- and wave driven flow and the larger flood-transport capacity due to wave effects, that exceeds sediment export due to the residual currents. In the sheltered environment of the basin sediment partly deposits, and partly the ebb currents transport sediment back onto the ebb-tidal delta. The majority is transported onto the southern part of the ebb-tidal delta, contributing to the ebb-shield development of Nieuwe Schulpengat (Franse Bankje and Bollen van Kijkduin), and the outbuilding of Zuiderhaaks (Fig. 3.15b). Only, a minor part of the sediments is transported from the basin through Molengat, contributing to the spit development of the Noorderlijke Uitlopers of Noorderhaaks. The slow rate of spit development and migration, and the relatively small morphological changes in the southern part of the ebb-tidal delta, despite the large residual sediment transport rates in the channels (Fig. 3.13) indicates that sediment recirculation is a dominant process on the ebb-tidal delta. Large gross transport rates but only a small net component due to the counteracting effects of seaward transport by ebb-tidal transports and redistribution due to landward directed wave-driven transports and flood-tidal currents. Three major sediment circulation cells (and associated smaller sub-systems) are identified on the basis of bathymetric change and model results: (1) Nieuwe Schulpengat system, (2) Noorderhaaks system and (3) Molengat system.

- Nieuwe Schulpengat system; the largest sediment transports occur in the Nieuwe Schulpengat system (see Fig. 3.13). Ebb-tidal transports distribute sediments through Nieuwe Schulpengat onto Franse Bankje. Large flood dominant transports prevail along the North-Holland coastline. Despite the large transports only limited sedimentation is observed. Wave breaking on the ebb-tidal delta front induces a landward transport of sediment over Franse Bankje back into the flood circulation, and a smaller northward transport along the seaward margin of the Bollen van Kijkduin shoal. Recirculation of sediment by evasive ebb- and flood transports possibly explains the stable position of Bollen van Kijkduin.
- Noorderhaaks system; schulpengat channel separates the Nieuwe Schulpengat and Noorderhaaks systems. The landward part of Schulpengat is flood-dominated and contributes

to the Nieuwe Schulpengat system. The seaward part of the channel is ebb-dominant and participates in the deposition of sediment onto the Zuiderhaaks. Wind-, waves- and tidal transports participate in the northward displacement of sediment from Zuiderhaaks, eventually, onto Noorderhaaks. From Noorderhaaks landward- directed transports redirect the sediments along the margins of the supra-tidal Noorderhaaks back into the main inlet circulation. A minor part of the sediments is expected to bypass the inlet directly along the margins of the ebb-tidal delta due to contraction of tidal flow, augmented by wind-driven flow.

- Molengat system; only a small portion of the ebb-transports are directed from the basin through Molengat onto the spit Noorderlijke Uitlopers of the Noorderhaaks. It is expected that, eventually, spit breaching will result in the merging of the breached shoal area with the Texel coastline and a new spit will be formed. The structural erosion of the Texel coastline points to the loss of sediment due to wave-dominated transports along the Texel coastline toward the inlet. Therefore, even merged deposits will largely be transported back into the inlet system by the wave driven transports.

3.7 CONCLUSIONS

Qualitative transport patterns for Texel Inlet were derived using an integrated analysis of field observations and model results obtained with a state-of the art process-based model. So far, the quantification of sediment transport patterns in complex domain inlets, and ebb-tidal deltas in particular, has largely relied on expert-judgment based interpretation of measurements of flow, waves, bathymetry, bedforms and tracer experiments. The availability of high-resolution and high-quality observations enabled to reconstruct the dominant sand transport patterns on the Texel ebb-tidal delta. In addition, these datasets provided a unique opportunity to calibrate and validate a morphological model application.

The Delft3D Online Morphology system was used to construct the Texel Outer Delta model. Quasi real-time simulations using measured time-series of water levels, wind and waves were made over a time-span significantly longer than the tidal period (3-months) on the scale of the inlet, O(km). Validation of the model results with the observations showed that the model is able to simulate the dominant features in the flow and transport patterns even in the complexity of the ebb-tidal delta domain. In other words, quasi real-time model simulations can provide a valuable tool in identifying the qualitative transport patterns, especially for inlet systems that are measured at lower resolution and frequency as Texel inlet. Additionally, the possibility to perform sensitivity analyses enables to identify the dominant processes and mechanisms for flow and sediment transport.

Both observational data and model results were combined to provide a qualitative impression of the sand transport patterns in the inlet domain. The present ebb-tidal delta developments are best described as a second-stage self-organizing process of redistribution and recirculation of sediments to obtain a natural equilibrium state, adapted to the changed configuration of the main-ebb channels. Sand is transported from the abandoned ebb-delta front (western margin of Noorderhaaks), along the margins of the supra-tidal Noorderhaaks shoal, into the basin were it partly settles. Sediment is also supplied to the basin by the northward directed littoral drift along the North-Holland coastline, and from erosion of the updrift and downdrift coastlines facing the ebb-tidal delta. Ebb-tidal currents redistribute sand back from the basin mainly onto the southern ebb-tidal delta shoals. Large gross transport rates, but small morphological changes points to sediment recirculation. Model results indicate that especially the interaction of tides with the non-tidal mechanisms wind and waves induces sediment import into the basin. Flood velocities are augmented by landward directed wind-and wave driven flow, and flood transport capacities are increased due to wave effects such as enhanced bed shear stresses and stirring of sediment.

Model simulations could be significantly improved by field observations on sediment concentrations and transport rates. The absence of such field measurements limits our model to a qualitative analysis rather than providing quantitative estimates. So far, limited by computational time, depth-averaged simulations have been made over a three-month period. The results of these three-month simulations might not present a valid estimate of year-averaged conditions. Model simulations are performed to determine long-term trends. Additionally, research is ongoing to identify the effects of the three-dimensional tidal current field and density-driven forcing on the scale of the ebb-tidal delta.

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Kustverdediging (1) en (2)
IK 4 en IK 5

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1 Inleiding

Soms 'gedraagt' een stuk (zandige) kust zich anders dan we zouden wensen. Soms vinden we het 'gedrag' dermate ongewenst, dat we redresserende maatregelen zouden willen treffen. Omdat het bij ongewenst kustgedrag veelal gaat om de gevolgen van 'aanvallen' vanuit zee, wordt er gewoonlijk van de 'verdediging' van de kust (kustverdediging) gesproken bij het treffen van dergelijke redresserende maatregelen. Wij zullen dat spraakgebruik volgen, maar eigenlijk is verdediging soms een wat te eng begrip.
De notie 'ongewenst gedrag' wordt in deze bijdrage zeer ruim opgevat. Alle vormen van kustgedrag, die we als mens als lastig of als hinderlijk ervaren, worden er onder gerekend.

De mens staat dus centraal in dit verhaal. Zeker in het huidige tijdsgewricht is dat niet gebruikelijk en doet het niet plezierig aan dat zo duidelijk te stellen. Natuur en ecologie zijn immers toch veel belangrijker aspecten. Naar mijn mening is het in deze bijdrage over alle vormen van kustverdediging echter wel een goed vertrekpunt.
Gedurende miljoenen jaren hebben kusten zich volledig natuurlijk ontwikkeld. Doorgaande kustaanwas kwam op de ene plaats voor; enige mate van kustteruggang op een andere plaats. Er was niemand die zich er zorgen over maakte; gewenst en/of ongewenst kustgedrag bestond (nog) niet.

Toen de eerste mensen zich min of meer permanent in de kustzones vestigden, was er ook nog niet veel aanleiding om zich veel van ongewenst kustgedrag aan te trekken. Als door doorgaande erosie een hutje wat te dicht aan zee kwam te liggen, werd het hutje afgebroken en werden de schamele bezittingen bij elkaar geraapt en werd het hutje een eindje verder landinwaarts weer opgebouwd en werden de bezittingen weer in gebruik genomen.
Pas sinds relatief korte tijd zijn de belangen en investeringen in de kustzone dicht bij de overgang land-zee zo groot dat de notie ongewenst kustgedrag een rol is gaan spelen. Overigens geldt dat dan nog niet overal aan onze kusten grenzend aan de wereldzeeën, maar in een relatief klein (maar wel een steeds groter) gedeelte van onze kusten. Enerzijds zijn onze belangen en investeringen in de kustzone steeds toegenomen, anderzijds zijn de mogelijkheden om ongewenst kustgedrag te redresseren ook steeds groter geworden. Het gevaar ligt op de loer dat we langzamerhand gaan denken dat kustgedrag wel 'maakbaar' is. Dat is een buitengewoon verwerpelijke gedachte. Verstandig met mogelijke vormen van kustverdediging omgaan, vergt een gedegen inzicht in de diverse natuurlijke processen die zich in de kustzone voordoen. 'Missers' zijn gemakkelijker gemaakt, dan goed functionerende kustverdedigingssystemen.

Over de mogelijkheden en de onmogelijkheden van kustverdedigingssystemen gaat deze bijdrage. Voorbeelden zullen worden gegeven.

Er is een groot aantal verschillende kusttypen; in deze bijdrage zal voornamelijk aan zandige kusten aandacht worden besteed. Kijkend naar een dwarsprofiel loodrecht op de kust gaat het, gaande vanuit zee naar het land, daarbij om een zandige vooroever, een strand en een duinenrij dan wel het vaste land. Soms, zoals in Nederland het geval is, ligt het land achter de duinen relatief laag; zelfs beneden gemiddeld zeeniveau. In dat geval fungeert een duinenrij ook als een echte waterkering. In veel andere gevallen ligt het vaste land veel hoger dan het gemiddeld zeeniveau.

Het ontwerpen van een 'goed' kustverdedigingssysteem, hoort te beginnen met een gedegen analyse van het kennelijk gevoelde 'probleem'. Antwoorden op de volgende vragen (overigens betreft het een niet uitputtende lijst) zijn daarbij van essentieel belang:

- Waarom is het kustgedrag eigenlijk ongewenst?
- Wat is eigenlijk het probleem?
- Wat zouden we eigenlijk willen bereiken?
- Als we een maatregel treffen die 'goed' werkt bij het 'ene' kustproces, voldoet die maatregel dan ook bij het 'andere' kustproces?
- Als we een maatregel treffen die 'goed' werkt in het goed gedefinieerde probleemgebied, heeft die maatregel dan geen ongewenste effecten in de belendende kustgedeelten?
- En als er dan eventueel minder gewenste effecten in belendende kustgedeelten zijn, is dat dan eventueel acceptabel, of eigenlijk helemaal niet?

Kustprocessen en kustproblemen staan verder in deze bijdrage centraal. Pas als de onderliggende processen in voldoende mate zijn begrepen en de problemen goed zijn benoemd, kan aan mogelijke oplossingen worden gedacht. Er is dus sprake van een probleemgestuurde aanpak.
[In dit verband is het eigenlijk wel opmerkelijk dat in ons vakgebied er ook tendensen zijn waar te nemen waarbij de 'oplossing' er al is (vaak een nieuwe, revolutionaire aanpak of uitvinding), en dat het 'probleem' er vervolgens wordt bijgezocht.]

2 Afbakeningen

In deze bijdrage gaat het primair om de toepassing van kustverdedigingssystemen.

> 'Gezien het onderliggende probleem, zou een serie strandhoofden een goede oplossing kunnen zijn.'
>
> 'In dit geval zouden we heel goed een strandmuur kunnen toepassen.'

Aan de zuiver constructieve aspecten van dergelijke toepassingen wordt geen aandacht besteed. Zie daarvoor bijvoorbeeld IK 8, IK 11 en IK 12. We gaan er hier gewoon vanuit dat de toegepaste constructies voldoende stevig zijn op de momenten dat het er op aan komt en ze hun 'werk' moeten doen.

Deze schriftelijke bijdrage kan niet worden losgezien van de bijbehorende Power Point presentatie; de toelichtingen hier zijn dus maar summier. Er worden hier ook geen figuren gegeven.

Er wordt verder vanuit gegaan dat we weten wat begrippen als strandhoofden en offshore golfbrekers globaal inhouden. Gangbare begrippen uit de kustwaterbouwkunde worden bekend verondersteld.

3 Relevante processen

'Erosie van de kust' is een veelgebruikt begrip dat vroeg of laat om (kustverdedigings)maatregelen vraagt. Het is goed om onderscheid te maken tussen twee vormen van kusterosie:

- structurele erosie
- erosie tijdens een zware storm(vloed)

Structurele erosie

Zogenaamde structurele erosie van een kustgedeelte is een duidelijk voorbeeld van 'erosie van de kust'.
Structureel betekent in dit verband dat er sprake is van steeds doorgaande erosie; jaar in jaar uit gaat dat maar door. Structurele erosie van een bepaald kustgedeelte kan een geheel natuurlijke oorzaak hebben, of kan in feite worden toegeschreven aan (gevolgen van) ingrepen van de mens in een eerder stadium.
Structurele erosie heeft vaak betrekking op een heel kustgedeelte (kustvak), maar kan ook heel goed worden duidelijk gemaakt aan de hand van het 'gedrag' van een enkel dwarsprofiel loodrecht op de kust. In een dergelijk dwarsprofiel is het mogelijk een (ruim) balansgebied te definiëren, waarin vervolgens wordt bijgehouden hoeveel zand er in dat dwarsprofiel als functie van de tijd aanwezig is. Bijvoorbeeld in het deel van het dwarsprofiel tussen een vertikale lijn ergens ver in de duinen; een vertikale lijn zeg ter hoogte van ongeveer de NAP - 10 m dieptelijn en een (willekeurige) horizontale lijn op zeg NAP - 15 m. Door structurele erosie blijkt er dan in de loop van de tijd zand te verdwijnen uit dat balansgebied. Uiteindelijk uit zich dat in het landwaarts verschuiven van de waterlijn (de zee rukt op), maar ook van een teruggang van de duinen. Onder water is uiteindelijk ook te zien dat de dieptelijnen in landwaartse richting zijn verschoven in de loop van de tijd. Het zandverlies kan worden opgevat als een onomkeerbaar proces. (Althans op een tijdschaal van zeg orde 10-tallen jaren. Dat is een tijdschaal waar we nu vaak direct mee te maken hebben.)
De mens als belanghebbende ziet dat het strand en de duinen in landwaartse richting verschuiven. Bouwwerken die op enig moment nog op enige afstand van de rand van de duinen waren gebouwd, blijken door de structurele erosie steeds dichter bij zee te komen te liggen. Zonder tegenmaatregelen stort een dergelijk bouwwerk uiteindelijk in zee.
Uiteraard hangt één en ander af van de specifieke omstandigheden, maar een zandverlies van ca. 20 m^3/m *per jaar* is een gangbaar zandverlies voor een structureel eroderende kust. Bij een dergelijk zandverlies 'hoort' een teruggang van de kust met ca. 1 m per jaar.

De tegenhanger van structurele erosie van een kustgedeelte is structurele aangroei. Het volume zand in het balansgebied neemt dan met de tijd toe.
Er zijn ook kustgedeelten die feitelijk stabiel zijn. Gezien over een reeks van jaren verandert er niet veel in de zandhoeveelheid in het balansgebied.

Zelfs als we beschikken over een uitvoerige reeks van gedetailleerde profielmetingen (in Nederland worden er bijvoorbeeld sinds 1965 jaarlijkse metingen uitgevoerd in een groot aantal dwarsprofielen; JARKUS-bestand) dan nog is het lastig om eenduidig vast te stellen of er van erosie of aanwas sprake is, dan wel of het feitelijk een stabiel dwarsprofiel betreft. Zelfs in een groot balansgebied, blijkt de hoeveelheid zand die daarin blijkens de metingen aanwezig, een nogal 'springerig' verloop met de tijd te vertonen. Het detecteren van een betrouwbare trend in de ontwikkeling, wordt daardoor bemoeilijkt.

Erosie tijdens een zware storm(vloed)

Tijdens een zware storm zijn de golven op zee veel hoger dan 'normaal'; door de zwaardere golfaanval verandert de vorm van een dwarsprofiel tijdens zo'n storm. Vaak ook zijn de waterstanden tijdens een storm veel hoger dan normaal; er is dan sprake van een stormvloed. De aanlandige wind stuwt het water op tegen de kust. Hogere waterstanden en hogere golven dan onder normale omstandigheden zijn er de oorzaak van dat de vorm van een dwarsprofiel tijdens een stormvloed wezenlijke veranderingen ondergaat. Er wordt bijvoorbeeld zand van de duinen afgeslagen en op dieper water weer afgezet. Beperken we ons tot het bovenste gedeelte van een dwarsprofiel (de duinen), dan zien we dat die zijn afgeslagen (zijn geërodeerd). Dat zouden we dus ook kusterosie kunnen noemen.

> [Als we overigens, net als bij structurele erosie, naar de zandbalans van een groter gedeelte van een dwarsprofiel kijken, dan zullen we zien dat er weliswaar zand van de duinen is verdwenen tijdens de stormvloed, maar dat de totale hoeveelheid zand in het balansgebied niet wezenlijk is veranderd. Er is slechts sprake (geweest) van een herverdeling van het zand over het totale dwarsprofiel. Overigens zien we ook dat in de periode na de zware stormvloed, de oorspronkelijke toestand langzamerhand weer wordt bereikt; het zand van wat dieper water wordt weer naar de kust getransporteerd; komt op het strand terecht en wordt uiteindelijk door de wind weer de duinen ingeblazen. Het is kennelijk een omkeerbaar proces.]

Uiteraard afhankelijk van de zwaarte van de stormvloed kan de afslag van de duinen groot zijn. Onder ontwerpomstandigheden (de ontwerpomstandigheden hebben betrekking op de 'veiligheid' van het lage achterland achter een relatief smalle duinenrij) geldt dat voor de Nederlandse duinenkust een afslag in de orde van grootte van 80 m kan worden verwacht. Dat gebeurt binnen een tijdsduur van circa *één dag*. De kans dat die 80 m ook daadwerkelijk wordt bereikt is maar klein. Wettelijk is in Nederland vastgelegd dat die kans kleiner moet zijn dan 10^{-5} per jaar. (Dat geldt voor Centraal Holland; voor andere delen van de kust zijn de eisen iets minder zwaar.)

Bouwwerken die dicht bij de rand van de duinen zijn gesitueerd kunnen dus gemakkelijk tijdens een zware stormvloed in zee verdwijnen.

'Door de hond of door de kat gebeten?'

Of een bouwwerk nu in zee verdwijnt door structurele erosie of door duinafslag tijdens een zware stormvloed, maakt voor de eigenaar van dat bouwwerk niet zoveel uit. In beide gevallen is hij of zij het bouwwerk kwijt.
Voor een kustbeheerder maakt het onderscheid tussen structurele erosie en erosie door duinafslag wel degelijk uit. Als de opgave voor de kustbeheerder zou zijn om een wat ongelukkig gesitueerd bouwwerk in voldoende mate te beschermen, is het van groot belang het genoemde onderscheid scherp in het oog te houden bij het afwegen van mogelijke kustverdedigingsmaatregelen.

Dynamiek / deel van een groter systeem
Zelfs in kustgedeelten die stabiel genoemd kunnen worden, vinden voortdurend veranderingen in de ligging van een dwarsprofiel plaats. (Vergelijk de toestand na een stormvloed waarbij enige afslag van de duinen heeft plaatsgevonden. Het strand en duinen liggen er dan heel anders bij dan direct vóór de stormvloed.) Met die inherente dynamiek moet bij het kustbeheer zoveel als mogelijk worden rekeninggehouden als er eventueel aan kustverdedigingsmaatregelen wordt gedacht.
Verder is het van belang om voortdurend te blijven beseffen dat het niet aangaat om een kustgebied waar zich kennelijk een 'probleem' voordoet, min of meer geïsoleerd te beschouwen. In de meeste gevallen is het probleemgebeid een onderdeel van een veel groter systeem. We moeten in dit verband vaak veel verder kijken dan onze neus lang is.

4 Enkele voorbeelden

In deze paragraaf worden enkele eenvoudige cases geschetst (een 'probleem' wordt benoemd). Het is vervolgens uitdrukkelijk de bedoeling dat uitsluitend het 'benoemde probleem' zo mogelijk wordt opgelost. Er wordt niet beoogd om op oplossingen aan te sturen die eventueel ook 'niet-benoemde problemen' kunnen oplossen. Als dat feitelijk de bedoeling was geweest, dan had het 'probleem' beter en uitvoeriger benoemd moeten worden. Er wordt wel zo mogelijk aangegeven wat de consequenties zijn van een gekozen alternatief voor andere bedreigingen.
Er wordt gemeend dat met dit wat formalistische uitgangspunt het keuzevraagstuk dat altijd dient plaats te vinden, zo duidelijk mogelijk wordt neergezet.

Er worden hier slechts enkele cases besproken; tijdens de presentatie volgen er meer.

> *1) In een toeristische kustplaats wordt het 'lastig' gevonden dat de overgang duinen-strand niet goed vastligt. Duinovergangen om het strand te bereiken vergen veel onderhoud. De verdere ontwikkeling van de kustplaats stagneert wat.*

Wat zijn de mogelijkheden?

Nadat zorgvuldig is vastgesteld dat het om een stabiel kustvak gaat, is het heel goed mogelijk om een strandmuur (nagenoeg verticale constructie) en daaraan aansluitend een boulevard (horizontaal gedeelte) aan te leggen. Goede overgangen naar het strand zijn te realiseren; de boulevard kan worden gebruikt om te wandelen en biedt aan de landwaartse zijde ruimte voor bijvoorbeeld restaurants.
De plaats waar deze constructie wordt gesitueerd is op de plaats van de overgang strand-duinen. Het reeds aanwezige strand blijft dan feitelijk op zijn plaats.

De hoogteligging van het boulevardgedeelte dient zorgvuldig te worden gekozen. Er moet daarbij bijvoorbeeld worden vastgesteld hoe vaak per jaar tijdens een storm(vloed) van enige wateroverlast op de boulevard door opspattend en overwaaiend water sprake mag zijn.
Het basisidee van de toepassing van een strandmuur is om een 'vaste' overgang duin-strand te bewerkstelligen. Relatief vaak voorkomende stormvloeden dienen dan dus geen schade aan de constructie op te leveren. Er moet echter een bewuste keus worden

gemaakt hoe 'sterk' de constructie dient te worden. 'Nimmer' schade vergt een zeer dure constructie. Bij de 'sterkte' van de constructie speelt uiteraard het te gebruiken materiaal een rol, maar ook de aanlegdiepte van de strandmuur onder het strand. Als tijdens een stormvloed door de aanleg van de strandmuur het zand niet meer van de duinen kan worden afgeslagen, wordt er juist voor de strandmuur een ontgrondingskuil gevormd die voorziet in (een deel van) de 'zandvraag' van de vooroever.
Met een voldoend sterke strandmuur, wordt wellicht ook de kans op schade tijdens een relatief zware stormvloed van bouwwerken in de duinen, verkleind.

2) Een 5 km lang kustvak vertoont structurele erosie. In de duinen/ op het vaste land van dat kustvak zijn in het verleden diverse bouwwerken opgetrokken en er is infrastructuur in de vorm van wegen en parkeergelegenheden aanwezig. Door de structurele erosie nadert de zee steeds meer en meer. Verwacht kan worden dat op termijn (zeg over 20 jaar) de eerste bouwwerken in zee zullen verdwijnen.
Wat zijn de mogelijkheden om deze ongewenste gang van zaken te keren?

Voor alle mogelijke alternatieven is het nodig om te weten hoe groot het kennelijk zandverlies uit het 5 km lange kustvak is en er dient (in eerste aanleg kan dat nog heel globaal zijn) te worden vastgesteld wat de oorzaak van de structurele erosie is. Veelal zal blijken dat een gradiënt in het langstransport de boosdoener is.

[In IK 10 wordt onder meer de doorgaande erosie van een kustgedeelte nabij de kop van een eiland door een opdringende getijgeul besproken. Ook in dergelijke gevallen is de uiteindelijke oorzaak een gradiënt in het langstransport (maar in dit geval van het transport in de getijgeul).]

Stel dat we hebben vastgesteld dat het gaat om een kennelijk optredend jaarlijks zandverlies van 20 m^3/m. Bij een duinhoogte van ca. 10 m boven gemiddeld zeeniveau [*Mean Sea Level* (MSL); ongeveer NAP in Nederland] en een *closure depth* van MSL - 10 m, levert dat een jaarlijkse teruggang van de kust (en dus ook de duinen) op van 1 m per jaar; een alleszins redelijke mate van structurele erosie. Het totale jaarlijkse zandverlies uit het 5 km lange kustvak bedraagt dan dus 100.000 m^3.

'Zachte' oplossing:
Wordt overwogen om met behulp van kunstmatige zandsuppleties het structurele erosieprobleem op te lossen, dan hoeven we verder niet veel meer inzicht in de precieze oorzaak van het erosieprobleem te hebben. Van tijd tot tijd de kennelijk optredende verliezen aan zand uit het kustvak aanvullen, volstaat. Bijvoorbeeld, in dit geval, elke 5 jaar ca. 500.000 m^3 zand kunstmatig in het kustvak aanbrengen. (Strand- dan wel vooroeversuppletie.) In veel gevallen is dit een prima (de beste) oplossing. (Zie ook bijdrage IK 6.)

De toestand van de kust op het eerste moment van ingrijpen wordt *grosso modo* gehandhaafd; de kust in het 5 km lange kustvak schrijdt niet verder terug. De kans dat de bouwwerken in de duinen tijdens een stormvloed in zee verdwijnen, blijft verder min of meer constant.
In deze case gaat het primair om het 5 km lange kustvak met bebouwing en infrastructuur. Aan het eventueel gewenste gedrag van de belendende kustvakken zijn in deze case geen eisen gesteld. Zijn dat in feite ook structureel eroderende kustvakken, maar bijvoorbeeld zonder bebouwing waardoor er geen noodzaak wordt gevoeld om daar maatregelen te treffen, dan gaat de erosie in de belendende kustvakken gewoon door. Op termijn kan dat betekenen dat het voortdurend gesuppleerde kustvak ten

opzichte van de belendende kustvakken steeds geprononceerder komt te liggen (zandig bolwerk in zee). De (jaarlijkse) zandverliezen zullen daardoor toenemen.
[In Nederland speelt deze kwestie niet omdat het het rijksbeleid is om 'overal' de structurele erosie te bestrijden.]

'Harde' oplossingen:
Als er 'harde' oplossingen (bijvoorbeeld strandhoofden, strandmuren of *offshore breakwaters*) als mogelijk alternatief voor het structurele erosieprobleem worden gekozen, moeten we veel meer achtergrondinformatie hebben over het erosieprobleem dan bij een 'zachte' oplossing, voordat er aan een oplossing gedacht kan worden. Aannemende dat er inderdaad een gradiënt in het langstransport optreedt, en dat het jaarlijkse verlies 100.000 m^3 is, betekent dit dat er aan de ene kant van het 5 km lange kustvak kennelijk jaarlijks 100.000 m^3 méér zand verdwijnt (S_{uit} [m^3/jaar]) dan er aan de andere kant van het kustvak is binnengekomen (S_{in}).

- In dit verband is het noodzakelijk te weten wat de 'ene' en de 'andere' kant van het kustvak is.
- Bovendien moeten we beseffen dat we met een geval met een jaarlijks sedimenttransport van S_{in} = 100.000 m^3/ jaar en S_{uit} = 200.000 m^3/jaar een totaal ander geval bij de kop hebben dan bijvoorbeeld met S_{in} = 300.000 m^3/jaar en S_{uit} = 400.000 m^3/jaar, hoewel de jaarlijkse verliezen uit het 5 km lange kustvak dan precies gelijk zijn (namelijk 100.000 m^3/jaar).
- Verder moeten we weten 'waar' de kennelijk aanwezige gradiënt in het langstransport 'werkt' in het dwarsprofiel. Vindt de gradiënt in het langstransport voornamelijk plaats in het bovenste deel van een dwarsprofiel (zeg in de brandingszone), dan is er sprake van een wezenlijk ander probleem dan waarbij de gradiënt in het langstransport in feite op wat dieper water blijkt op te treden. In het laatste geval eroderen uiteindelijk ook het strand en de duinen, maar is er ook sprake van zeewaarts gericht dwarstransport als een verbindende schakel tussen het bovenste deel van een dwarsprofiel en het diepere profieldeel.

Kortom, voordat we met het genereren van 'harde' oplossingsvarianten aan de slag kunnen, moeten we veel meer kenmerken van het onderliggende systeem kennen dan bij de toepassing van 'zachte' oplossingen.

Aan de basis van het geschetste erosieprobleem liggen zandtransporten langs de kust ten grondslag. Het hoofdidee bij de toepassing van 'harde' constructies is dat effectief wordt ingegrepen in de optredende zandtransporten. Omdat S_{uit} groter is dan S_{in} is er het erosieprobleem; S_{uit} zou dan in feite door de toe te passen constructies moeten worden gereduceerd; als het kan: gelijk worden aan S_{in}.

Strandhoofden?
Stel dat het gaat om een situatie waarin (gradiënten) in het langstransport in het bovenste deel van het dwarsprofiel (voornamelijk brandingszone) blijken voor te komen, dan is het heel goed denkbaar dat een systeem met bijvoorbeeld een serie strandhoofden, zou kunnen werken. Dergelijke strandhoofden zijn immers effectief in staat om optredende langstransporten te beïnvloeden. (Remmen stromingen vlak onder de kust af; drukken de stromingen wat uit de kust.)
Maar wat ook direct duidelijk zal zijn is dat één en ander nogal nauw luistert. S_{uit} moet feitelijk worden gereduceerd tot S_{in}. In het algemeen betekent dit, dat slechts een deel van de optredende langstransporten moeten worden gereduceerd. In principe kan dat door een 'juiste' lengte van het strandhoofd te kiezen. In het voorgaande werd al

aangegeven dat een geval met S_{in} = 100.000 m^3/ jaar en S_{uit} = 200.000 m^3/jaar een heel ander geval is dan een situatie met S_{in} = 300.000 m^3/ jaar en S_{uit} = 400.000 m^3/jaar. In het eerste geval gaat het om een reductie met 50%; in het tweede met 'slechts' 25%.

In de literatuur zijn vaak 'vuistregels' te vinden over bijvoorbeeld lengte/onderlinge afstand verhoudingen van series strandhoofden. Gezien de voorgaande voorbeelden, zal het duidelijk zijn dat dergelijke vuistregels met zeer grote korrels zout moeten worden genomen.

Als een serie strandhoofden in het gedefinieerde probleemgebied goed blijken te werken (nodig en voldoende de optredende transporten beïnvloeden), dan is (versterkte) lijzijde erosie onvermijdelijk. Er wordt immers minder zand aangeboden dan voorheen; de oorspronkelijke sedimenttransporten herstellen zich. Het erosie probleem van het 5 km lange kustvak, wordt in feite verlegd naar het belendende kustvak (wellicht van de buren).
'Uitvinders' die beweren met *fancy* strandhoofd-lay outs (bijvoorbeeld Y-groynes; T-groynes; 'slimme' versterf-constructies; strandhoofden die deels permanent onder water liggen) lijzijde erosie te kunnen vermijden, weten over het algemeen niet waar ze het over hebben.

Strandhoofden (constructies) loodrecht op de kust, hebben nauwelijks of geen invloed op de mate van duinafslag tijdens een zware stormvloed.

Strandhoofden zijn vaak van steen. Soms ook in de vorm van stalen of houten damwanden. Paalrijen worden ook soms toegepast als alternatief voor strandhoofden. De 'werking' van paalrijen is vergelijkbaar met die van strandhoofden; wellicht is de 'werking' zelfs wat inzichtelijker.

Strandmuren/glooiingen?
Bij structurele erosie gaat het land (gaan de duinen) uiteindelijk ook achteruit. Door de aanleg van een strandmuur of glooiing, kan de teruggang heel goed worden voorkomen. Er dient echter te worden beseft dat in eerste aanleg dergelijke constructies niet interfereren in de optredende langstransporten. [Die transporten vinden onder water plaats, terwijl de strandmuur of glooiing (bij eerste aanleg nog) hoog en droog nabij de duinvoet is gesitueerd.] Dat betekent dat de zandverliezen gewoon doorgaan; het strand verdwijnt op termijn geheel; uiteindelijk staat er voortdurend water tegen de constructie. Als het zeewaarts van de constructie uiteindelijk zeer diep is geworden, stopt de verdere erosie wel. (Uiteindelijk interfereert ook een dergelijke constructie in de optredende processen maar wellicht pas na een reeks van zeer ongewenste ontwikkelingen.)
Is dat erg?
Nee, als ons niets kan schelen of we nu een strand hebben of niet.
Ja, als we eigenlijk, bijvoorbeeld voor recreatie, mooie stranden willen hebben en houden. In dat laatste geval is het dus een uitermate slechte oplossing voor een structureel erosieprobleem.

Vaak ondoordacht worden strandmuren en of glooiingen toegepast in structureel eroderende kustgedeelten. Schrijnende gevolgen zijn vaak het resultaat (veelal zijn die gevolgen pas op enige termijn te zien; in de eerste jaren ziet het er nog wel goed uit; de duinen van het verdedigde stukje kust vertoont geen erosie; bij de buren gaat de erosie gewoon door; de ontwerper vindt zichzelf buitengewoon knap.). Het is in feite een schande voor onze beroepsgroep dat 'we' dergelijke constructies aanbevelen in gevallen die daarvoor niet zijn geschikt.

Offshore golfbrekers?
Offshore golfbrekers kunnen een oplossing zijn voor het erosieprobleem. Ze zijn in staat optredende (langs)transporten te beïnvloeden. Net als bij strandhoofden gaat het om een uitermate lastig *'tuning'* vraagstuk.
De 'werking' van *submerged* golfbrekers (golfbrekers waarvan de kruin gewoonlijk beneden de waterspiegel ligt) is wezenlijk verschillend van die van *emerging* golfbrekers (daarbij steekt de kruin gewoonlijk boven de waterspiegel uit.)
(De 'werking' van offshore golfbrekers komt tijdens de presentatie verder aan de orde.)

Bij de summiere bespreking van deze case is het wezenlijk verschil tussen een 'harde' en een 'zachte' oplossingsrichting al even aan de orde gekomen.
Bij 'hard' dient de toe te passen constructie effectief iets te doen met de optredende processen; en als het goed gebeurt dan wordt het doel ook bereikt. Dat kan in medische termen worden opgevat als 'bestrijding van de ziekte'.
Bij 'zacht' gebeurt dat in principe niet. In de optredende processen wordt niet ingegrepen. De erosie gaat gewoon door. Slechts de gevolgen worden bestreden. In medische termen: 'bestrijding van de symptomen'.
In de medische praktijk wordt de optie 'bestrijding van de ziekte' veel hoger aangeslagen dan alleen maar 'bestrijding van de symptomen'. In de kustwaterbouwkunde ligt dat niet zo duidelijk. Veelal is 'zacht' een veel betere oplossing dan 'hard'.

3) Als case 2), maar nu is vastgesteld dat de boosdoener, de gradiënt in het langstransport, zich feitelijk op dieper water bevindt.

Zijn bijvoorbeeld strandhoofden ook in dit geval een serieuze optie?

In dit geval zijn strandhoofden geen goede optie. Het probleem zit op wat dieper water. Tenzij de strandhoofden heel lang worden gemaakt, zullen ze de optredende transporten niet kunnen beïnvloeden. Door het zeewaarts gerichte dwarstransport (dat wordt niet beïnvloed door strandhoofden met een gangbare lengte) zal de erosie van het strand en uiteindelijk de duinen, gewoon doorgaan.
Wellicht dat in een dergelijk geval een onderwater dam zou kunnen worden overwogen (hangend strand).

4) Bestaande bebouwing in een kustplaats 'blijkt' eigenlijk wel wat dicht bij de rand van de duinen te zijn gesitueerd. Achteraf gezien wordt het risico (kans x gevolg) eigenlijk te groot gevonden.
[Het betreft een stabiel kustvak, dan wel een kustvak waar de gevolgen van structurele erosie effectief met zandsuppleties worden tenietgedaan.]

Wat zijn hier mogelijke oplossingen?

Pas sinds enige tijd (ook in Nederland!) is het besef gegroeid dat er indertijd soms bouwwerken in de kustzone zijn opgetrokken, waarvan we nu, min of meer achteraf, vinden dat ze misschien wat ongelukkig zijn gesitueerd. (Vroeger wist men niet beter.) De kans dat het tijdens een stormvloed misgaat, wordt eigenlijk te groot gevonden. Als er door de zee bouwwerken worden verzwolgen, geeft dat een hoop gedoe. Dat moet dus niet te vaak gebeuren.

(Overigens is uit recent afstudeerwerk bij de TU Delft gebleken dat er alle aanleiding toe is om hier niet al te krampachtig mee om te gaan. Dicht bij zee wonen of er een hotel exploiteren, bieden zoveel extra voordelen dat kansen op 'misgaan' van in de orde van grootte van 1/100 - 1/200 per jaar volstrekt redelijk moeten worden geacht.)

Aangezien het hier over een in feite stabiel (gehouden) kustvak gaat, kan er heel goed de constructie van een voldoend sterke glooiing worden overwogen. Maar aangebracht ter plaatse van het huidige steilere beloop van de duinen bij de overgang strand-duin, is meestal niet zonder meer wenselijk in verband met de 'natuurlijke' aanblik van het duin. Een 'verborgen kering' (feitelijk op enige afstand landwaarts van de rand van de duinen aangelegd) heeft dat bezwaar niet. Onder normale omstandigheden zit de kering onder het duinzand; slechts zeer sporadisch bij een zwaardere storm komt de kering dan bloot te liggen.

Soms zou het om min of meer geïsoleerde stukjes kust kunnen gaan waar een dergelijke verdediging nodig wordt gevonden. In de kustgedeelten direct naast de beide uiteinden van de verdediging kan dan meer afslag worden verwacht dan zonder de verdediging het geval zou zijn geweest. Dit soort extra afslag is nog steeds heel lastig te kwantificeren.
Het aanbrengen van een vertikale damwand juist zeewaarts van het object behoort ook tot de mogelijkheden.

De risico's zouden ook verminderd kunnen worden als de afstand zee-object groter zou worden gemaakt dan nu in feite het geval is. Kunstmatig (met zand) verbreden van de duinen aan de zeewaartse zijde is dan een optie. Dat kan echter niet 'straffeloos' gebeuren. In een 'normaal' dwarsprofiel bestaat er min of meer een vaste afstand tussen zeg de hoogwaterlijn (HW) op het strand en de duinvoet (DV). (De hoogteligging van de duinvoet is gewoonlijk zodanig dat niet bij elke geringe verhoging van de waterstand, de duinen worden aangetast.)
Alléén het duin verbreden, zou dan betekenen dat de afstand HW-DV minder wordt. Binnen de kortste keren herstelt het kustsysteem de 'oorspronkelijke' toestand. Uiteindelijk komt het er op neer dat een groot deel van het totale dwarsprofiel in zeewaartse richting opschoven moet worden om ook de duinen te verbreden. Als dat lokaal gebeurt is er in feite sprake van een zandig bolwerk, dat hoe dan ook veel onderhoud zal vergen.

5) Bestaande strandbreedte wordt voor de recreatie wat smalletjes gevonden; graag een breder strand.

Hoe eventueel te bereiken?

Bij case 4) is het al even aan de orde geweest; een normaal dwarsprofiel heeft een bepaalde vorm met karakteristieke afmetingen. Eén en ander hangt nauw samen met de samenstelling van het aanwezige sediment en de globale randvoorwaarden (onder meer golfklimaat en getijfluctuaties). Een dwarsprofiel dat min of meer in evenwicht is 'voelt zich kennelijk wel lekker' bij de gegeven omstandigheden. In een dergelijk dwarsprofiel kunnen weliswaar gemakkelijk kunstmatig verstoringen worden aangebracht, maar het natuurlijke kustsysteem zorgt er heel snel voor dat de verstoring weer wordt tenietgedaan (vergelijk het graven van een kuil in het strand, of de bouw van een zandkasteel op het strand bij opkomend water).
Ten behoeve van de recreatie alléén het droge strand verbreden, gaat dus niet. Ook een deel van de vooroever moet dan worden opgehoogd om een enigszins blijvende

situatie te verkrijgen. (Nadelen: relatief veel zand nodig; veel onderhoud omdat het maar lokaal wordt toegepast.)

Er zijn uiteraard oplossingen te overwegen waarbij met (lange strandhoofden) en wellicht een onderwater dam (hangend strand), het doel kan worden bereikt.
Maar hier wordt enige aandacht besteed aan een methode die zo nu en dan weer in de literatuur opduikt: het draineren van het strand. In een 'gewoon' dwarsprofiel vindt er boven de waterlijn golfoploop plaats (*swash zone*). Water loopt het strand op (en neemt daarmee zand mee naar de kust); een deel van de water zakt in de bodem en stroomt uiteindelijk als grondwater terug naar zee, en het andere deel komt met de terugtrekkende waterbeweging weer in zee terecht. Met dat terugtrekkende water wordt ook zand naar zee meegevoerd. In een dwarsprofiel dat globaal gezien in evenwicht is, wordt er kennelijk evenveel zand naar land getransporteerd, als er naar zee wordt getransporteerd.
Door nu het strand te draineren (geperforeerde buizen evenwijdig aan de waterlijn in het strand) en dus water weg te pompen, is het idee dat er uiteindelijk meer zand naar land wordt getransporteerd dan naar zee. Het bovenste deel van het dwarsprofiel zal dan uiteindelijk steiler komen te staan dan oorspronkelijk het geval was. De gewenste strandverbreding zou het resultaat kunnen zijn.
Misschien dat het 'werkt', maar het zal duidelijk zijn dat zo gauw het pompen wordt gestaakt of dat de pompen uitvallen, de strandverbreding weer wordt tenietgedaan.

In het navolgende worden nog een aantal cases kort beschreven; de uitwerking zal tijdens de presentatie plaatsvinden.

6) Ontwerp van een (relatief beperkt) recreatiestrand op plaatsen waar er nu eigenlijk nauwelijks een strand aanwezig is.

7) Kustverdediging bij grootschalige landaanwinningsprojecten in zee.

8) Eijerlandse Dam aan de noordkust van Texel.

5 Slot

Soms is kustverdediging nodig. Met 'zachte' maatregelen kan men zich nauwelijks een buil vallen. Toepassing van 'harde' maatregelen vergt een gedegen inzicht in het kustgedrag, voordat we daadwerkelijk aan de slag kunnen gaan.
Hoewel het er soms op lijkt dat de natuur gewoon als een 'groot laboratorium' wordt gebruikt ("we weten eigenlijk niet precies wat we aan het doen zijn, maar we zien wel wat er gebeurt"), zijn onze kusten te fraai en te waardevol om daar onoordeelkundig mee om te gaan. Er zijn helaas te veel voorbeelden waar dat kennelijk wel is gebeurd (en nog gebeurt).

Zandsuppletie: ontwerp, ontwikkelingen en ervaringen

Ruud Spanhoff, Rijkswaterstaat RIKZ

Inleiding

Een belangrijk deel van Nederland ligt onder de waterspiegel ("Amersfoort aan zee"). Veiligheid tegen overstroming is dus cruciaal. Op vele plekken dreigt de kustlijn achteruit te gaan, hetgeen niet meteen een lagere veiligheid hoeft te betekenen (als bijv. de duinen zouden meebewegen), maar wel landverlies. Om het laatste te voorkomen is het beleid van Dynamisch Handhaven in 1990 ingesteld. In dit beleid wordt bij voorkeur via zandsuppleties voorkomen dat een (zandige) kust een gekozen grens, de basiskustlijn BKL, landwaarts passeert. Deze BKL is in principe de kustlijnpositie voor het jaar 1990. Aanvankelijk werden de suppleties als strandsuppletie uitgevoerd, maar vanaf de tweede helft van de negentiger jaren meer en meer als onderwatersuppletie. Een voldoende positief effect van deze innovatieve vorm van suppleren was (en is) niet gegarandeerd, daarom worden de uitgevoerde suppleties extra bekeken. Soms wordt een combinatie van een strandsuppletie en een onderwatersuppletie toegepast. De ervaringen tot nu toe zijn meestal goed. Onderwatersuppleties hebben praktische voordelen. Daarnaast wordt daarmee (voor hetzelfde geld veel) meer zand in het systeem gebracht dan bij een strandsuppletie, dus voldoen ze ook aan een nieuwe doelstelling, namelijk om de Nederlandse kust op de langere termijn mee te laten groeien met de stijging van het zeeniveau.

Suppleties tegen zandverlies zandige kust

Van de 432 km Nederlandse kust is 2/3 zandig (duinen, zandplaten). Daarvan bestaat 254 km uit duinen, waarvan 62% onverdedigd is en de rest beschermd wordt door strandhoofden en in mindere mate paalrijen. De Nederlandse kust is niet statisch, maar gaat op sommige plekken vooruit en op andere achteruit. Snelheid en (soms) richting van de verplaatsing wisselen in de tijd. De kust reageert op veranderingen uit het verleden, variërend van zeespiegelstijging tot menselijke ingrepen (zoals het aanleggen van havendammen), maar bijvoorbeeld ook op variaties in windrichting en -sterkte (over jaren tot decaden) en op het passeren van zandgolven op dieper water. Suppleties als middel om ongewenste kustachteruitgang tegen te gaan spelen flexibel in op dit wisselende gedrag. Dit in tegenstelling tot harde maatregelen als strandhoofden en strandmuren.
Zo zijn in het verleden (sinds midden 19^{e} eeuw) vele strandhoofden aangelegd op het moment dat de kust ver was teruggeschreden, en zijn ze later bij een weer aangroeiende fase werkeloos onder het zand komen te liggen. Verder blijken strandhoofden vaak het erosie-probleem te verplaatsen, tot voorbij het jongste hoofd, waar dan weer een nieuw hoofd nodig zou zijn. Strandsuppleties verstoren het natuurlijke systeem veel minder dan harde constructies en blijken relatief voordelig als men de totale kosten van aanleg en onderhoud van harde maatregelen beschouwt. Harde maatregelen worden enkel nog toegepast in gebieden waar zandsuppleties niet effectief zijn, zoals in systemen met getijgeulen dicht onder de kust (bij eilandkoppen).

Dankzij ontwikkelingen in de baggerindustrie werd het uitvoeren van zandsuppleties technisch en financieel mogelijk en aantrekkelijk. Suppleties werden in Nederland sinds de jaren zeventig meer en meer gemeengoed. In het begin ging het daarbij naast strandsuppleties ook om duinverzwaringen en –verhogingen waarbij het zand (meters hoog) tegen de zee- en/of landzijde van het duin of erop werd geplaatst. Doel van de duinversterkingen was de duinen op deltaveiligheid te brengen. Sinds de voltooiing van dat programma werden haast uitsluitend strandsuppleties uitgevoerd, met name sinds 1990 met het invoeren van het beleid van Dynamische Kusthandhaving. In dat beleid wordt, op straffe van een zandsuppletie, niet of slechts tijdelijk getolereerd dat de gemeten kustlijn een gekozen vaste kustlijn (Basiskustlijn) landwaarts passeert (zie onder). In totaal werd sinds 1990 jaarlijks in Nederland circa 6 miljoen m^3 zand gesuppleerd (tegen zo'n 20 miljoen euro). Sinds 2000 gaat het om zo'n 12 miljoen m^3 zand per jaar, tegen 40 miljoen euro. Dat de kostenstijging slechts 50% bedraagt voor de dubbele hoeveelheid zand komt omdat een belangrijk deel van de suppleties nu wordt uitgevoerd als (relatief veel goedkopere) onderwatersuppletie. De verdubbeling is het gevolg van nieuw aanvullend beleid volgens welke de grootschalige zandhoeveelheid in het Nederlandse kustsysteem op de lange duur (orde 50-100 jaar) in de pas moet blijven met de relatieve zeespiegelstijging.

Kustlijnhandhaving en suppletie-ontwerp

In het beleid van Dynamisch Handhaven is allereerst een operationele, meer robuuste (lees: minder invloed toevallige fluctuaties) definitie van de kustlijn geformuleerd. Vervolgens wordt niet naar afzonderlijke jaargegevens gekeken, maar naar de trend van een 10-tal (voorgaande) jaren, om de invloed van toevallige fluctuaties verder te reduceren. Tenslotte is de (te handhaven) basiskustlijn BKL van 1990 gedefinieerd. Deze is meestal de kustlijn zoals hiervoor uitgelegd, dus volgend uit een lineaire extrapolatie tot 1990 van plaatselijk gemeten zandvolumina uit de jaren 1980-1989. Op sommige plaatsen is bewust een afwijkende BKL gekozen, bijv. een enigszins zeewaarts verplaatste om voor extra veiligheid te zorgen.
De kustlijn wordt dus niet meer gezien als de positie van de laag- of hoogwaterlijn of van de duinvoet, zoals die sinds midden 19^e^ eeuw zijn gemeten. De kustlijn wordt nu berekend uit een gemeten zandvolume (in een schijf tussen duinvoet en een daaruit afgeleide diepte onder water), vanuit de notie dat de vooroever en het droge deel van het kustprofiel samen de kust uitmaken en de elementen trotseren. Het kustprofiel kan sterk variëren in de tijd. Na een storm zal veel zand naar dieper water zijn afgevoerd, dat in rustige perioden (hopelijk grotendeels) weer op het droge terug komt. Bij het hanteren van een zandvolume worden toevallige fluctuaties sterk gereduceerd en wordt een betrouwbaarder maat verkregen. Bovendien wordt een te toetsen kustlijn (t.o.v. de zogenaamde basiskustlijn BKL) niet uit een enkele jaarlijkse opname afgeleid, maar uit een extrapolatie vanuit de 10 voorafgaande jaren, indien mogelijk. Daardoor worden toevallige (jaarlijkse) fluctuaties verder uitgefilterd. Soms, na een strandsuppletie bijv., wordt noodgedwongen naar een kortere periode gekeken, namelijk die sinds de suppletie.
De genoemde procedure is mogelijk omdat de hele Nederlandse kust sinds 1965 ieder jaar wordt opgenomen, met schepen (lodingen) in vaste raaien loodrecht op de kust met een onderlinge afstand van 200-250 m, en met waterpassing, later vervangen door luchtfotogrammetrie resp. laseraltimetrie voor het droge (tot ca 200 m landwaarts van de top van de eerste duinenrij). Jaarlijks wordt een overzicht (Kustlijnkaarten) uitgebracht, waarin van de hele kust te zien is waar de kustlijn (zoals boven omschreven) ligt t.o.v. de BKL, en welke verplaatsingstrend ze vertoont.

Aanvankelijk werd een strandsuppletie geplaatst op de plaats waar de kust de BKL dreigt te overschrijden. Het zand wordt met sleephopperzuigers van diep water gehaald (buiten de NAP –20 m lijn). Het benodigde volume volgt (op basis van eerdere ervaringen) uit de

ontwerpleeftijd vermenigvuldigd met het jaarlijkse (uit de regressie) zandverlies uit de BKL-schijf.. Hierop wordt een (gebiedsafhankelijke) toeslag van ca. 10-20% gezet i.v.m. snellere zandverliezen in de eerste fase na de suppletie. Zoveel mogelijk wordt het zand op het strand of tot net onder de waterlijn gelegd. De aangebrachte hoeveelheid zand kan dan zonder schepen worden gemeten. De aannemer kreeg namelijk meestal betaald voor de gemeten aangebrachte hoeveelheid (de contracten zijn nu anders). Typische afmetingen zijn orde 250 m^3 per strekkende meter over een lengte van orde een kilometer. Sinds het succes van de eerste (1993) grote onderwateroeversuppletie (kortweg: onderwatersuppletie) in Nederland (2 miljoen m^3 bij Terschelling) worden dergelijke suppleties meer en meer toegepast, vanuit de regel: "suppleer daar waar het kan onder water en daar waar het moet op het strand". Bij een onderwatersuppletie wordt het zand op de vooroever gelegd. Meestal wordt het direct door de sleephopperzuiger gestort door de bodem open te klappen. Dit is goedkoop, per m^3. De zeebodem wordt dan (in stappen) opgehoogd tot zo'n –5 m NAP, op plekken waar hij eerst zo'n –5m tot –8m diep was. Soms wordt rainbowen toegepast om de bodem op ondieper water te verhogen. Werd bij Terschelling nog tussen de buitenste brekerbanken gestort, latere onderwatersuppleties zijn vooral tegen de zeewaartse flank van de buitenste bank aangelegd, omdat dit praktischer en goedkoper is. Typische afmetingen zijn orde 400 m^3 per strekkende meter kust over lengten van 1 tot een paar kilometer, dus orde 1 tot een paar miljoen m^3 zand. Het effect van onderwatersuppleties wordt nog bestudeerd, o.a. met extra bodemopnamen. Het duurt enige tijd voordat een onderwatersuppletie effect heeft op de kustlijn. Het meeste aangebrachte zand ligt namelijk onder de BKL rekenschijf, en pas bij het aanpassen van de bodem aan deze verstoring komt (hopelijk voldoende) zand in die schijf. Verschillende processen moeten daarvoor zorgen, met name dwarstransport, en zandvangst uit het langstransport in de brandingszone nu de golven extra kunnen breken op de onderwatersuppletie (en het zich aanpassende bankensysteem). In het ontwerp moet met deze tijdvertraging rekening worden gehouden.
Men geeft de voorkeur aan onderwatersuppleties op grond van uitvoeringstechnische voordelen (geen overlast op het strand zoals door pijpleidingen en bulldozers bij strandsuppleties; flexibelere uitvoering, ook bij wat slechtere omstandigheden t.o.v strandsuppleties) en omdat voor hetzelfde geld meer zand bij de kust wordt gebracht. In de regel wordt (meer dan) 2x zoveel zand aangebracht voor (minder dan) de halve prijs per m^3 t.o.v. een strandsuppletie. Dit extra zand is op de lange duur tevens nuttig om de kust van Nederland te laten meegroeien met de stijgende zeespiegel.

Bestuurlijke organisatie en relatie met de aannemer

In 1990 is een sterke decentralisatie van taken betreffende de kust doorgevoerd, met een gewijzigde taakverdeling tussen rijk, provincie en waterschappen. Was voorheen de beveiliging tegen overstroming een rijks(waterstaats)taak, sindsdien is de zorg voor de waterkering overgedragen aan de waterschappen/hoogheemraadschappen. Zij zijn voortaan verantwoordelijk voor onderhoud. Hierbij kan worden aangetekend dat toen de kust eenmaal op deltahoogte was gebracht de kosten daarvan laag leken zolang de BKL gehandhaafd zou worden. De provincie heeft de supervisie over deze taak, terwijl het rijk de algehele landelijke supervisie houdt. De kustlijnhandhaving is een rijkstaak. Jaarlijks wordt door de rijkswaterstaat een suppletieschema voorgesteld, rekening houdend met het totaal beschikbare budget en een prioriteitsstelling. Dit schema wordt afgestemd met de diverse POK's (Provinciaal Overlegorgaan voor Kust), waarin rijk, provincie, gemeenten en waterschappen vertegenwoordigd zijn. Vervolgens wordt het (door de minister van V&W) vastgestelde schema uitgevoerd. D.w.z. de betreffende RWS diensten (bijv. dienstkringen) maken een

bestek waarin alle aspecten rond een suppletie aan bod komen (ontwerp, zandwinning, eventueel uitvoering en periode, controle en kwaliteitsborging, verrekening etc.), waarop aannemers kunnen intekenen. Tevens worden het nodige grote aantal vergunningen en ontheffingen aangevraagd, bij hoogheemraadschap, gemeente etc. Dit is inmiddels een standaard procedure. Ten slotte vindt gunning en uitvoering plaats.
Een goede relatie met de aannemers is van belang voor hen en voor het rijk. De relatie moet zakelijk zijn, maar mag niet te nadelig voor een van de partijen uitvallen i.v.m. de gewenste continuïteit. Goed kunnen inplannen van het dure materieel kan kostenbesparend werken, en soms zijn meerjarige contracten aldus wenselijk. [Bij strandsuppleties werd vooral afgerekend op de gemeten aangebrachte hoeveelheid zand, bij onderwatersuppleties wordt meestal uitgegaan van optiecharter contracten, waarbij het materieel per dag wordt gehuurd en betaald, zij het soms met een bonus/malus regeling. Het zand vervoerd in het beun wordt gemeten. Momenteel wordt gezocht naar een alternatieve vorm van aanbesteden.] Bij de afrekening kunnen makkelijk geschillen ontstaan, gezien de onnauwkeurigheden van de meetmethoden, die bij voorkeur in redelijkheid dienen te worden beslecht. Overleg tijdens de uitvoering is vaak op z'n plaats. De aannemerij investeert in materieel op grond van te verwachten mogelijke opdrachten. Zo worden vooral middelgrote hopper dredgers toegepast nu het suppletiezand meestal van dieper water (buiten de NAP-20 m lijn) moet worden opgehaald. Soms bestaat er bij een onderwatersuppletie behoefte aan een kleiner schip, om bijv. zand hoger in het profiel aan te brengen, terwijl een groter schip bijv. juist kostenbesparend zou kunnen worden om het eerste diepe fundament aan te brengen. In een enkel geval wordt nog werk met werk gemaakt, bijvoorbeeld door een combinatie van vaargeulonderhoud met strandsuppletie. Daarvoor is geen zandwinvergunning vereist. Voor zandwinning op diep water zijn speciale gebieden aangewezen, onder de bevoegdheid van de directie Noordzee (DNZ) van RWS. Voldoet de winning aan bepaalde eisen (bijv. wingebied blijft binnen bepaald oppervlak, kuil wordt niet te diep, zand is nagenoeg slibvrij) dan kan de DNZ toestemming verlenen.

Innovatie en evaluatie

Omdat we in Nederland al enkele tientallen jaren strandsuppleties uitvoeren zouden we haast vergeten dat dit op zich al een innovatie was t.o.v. de harde constructies uit het verleden. Het vereiste een andere instelling, namelijk om zand aan te brengen, als een slijtlaag die weer mocht verdwijnen, terwijl men gewend was aan blijvende zaken als strandhoofden. Strandsuppleties zijn veel natuurlijker en blijken goedkoper omdat harde constructies hoge investerings- en onderhoudskosten hebben. Ook het instellen in de jaren zestig van de jaarlijkse kustmetingen (JARKUS) die later nodig bleken om suppletiebehoeften vast te stellen, is innovatief te noemen. Met deze metingen konden de strandsuppleties ook geëvalueerd worden, wat dan ook redelijk systematisch is gebeurd. Daarbij bleek dat niet te veel zand moet worden aangebracht, vanwege grotere initiële verliezen, maar ook niet te weinig omdat dan de aanloopkosten relatief (te) groot zijn. Als vuistregel geldt nu dat een beoogde levensduur van zo'n 5 jaar optimaal is. Hierbij is levensduur gedefinieerd als de tijd die het duurt voordat men opnieuw moet suppleren om de kustlijn in het gesuppleerde kustvak in stand te houden. Het uit dit vak verdwenen suppletiezand blijkt ten goede te komen aan de aangrenzende kustvakken waar mogelijk vroeg of laat, en dankzij deze suppletie dus later, ook gesuppleerd moet worden, zodat het zand niet echt verloren is.
Eveneens innovatief mag men het invoeren van de BKL en de bijbehorende kustlijndefinitie en toetsingsmethode noemen. In Nederland is het besef relatief sterk dat men vooraf goed moet definiëren wat met een suppletie beoogd wordt en welke kentallen daarbij horen. Er

wordt een duidelijk onderscheid gemaakt tussen bijv. kustlijnhandhaving, veiligheid, recreatie en natuur.
De overgang naar onderwatersuppleties is een volgende, sterk innovatieve stap, waarvan het succes op voorhand niet gegarandeerd was. Het na een eerste succesvolle poging (Terschelling) grootschalig overstappen op deze alternatieve vorm getuigt van durf en visie. Via het bestuderen en evalueren van de effecten van de (meeste van de) uitgevoerde onderwatersuppleties wordt gezocht naar optimale ontwerpregels voor toekomstige onderwatersuppleties, een volgende innovatie.

Vergelijking met het buitenland

Op een paar punten onderscheidt de situatie in Nederland zich wezenlijk van die in het buitenland. Het Nederlandse kustgebied is (met reden) een van de best geobserveerde en bestudeerde kustgebieden in de wereld, hetgeen duidelijk van voordeel was bij het invoeren van het kustlijnhandhavingsbeleid en de suppletiestrategie. Er zijn weinig landen waar zoiets op nationale schaal gebeurt of zou kunnen gebeuren. Dit komt natuurlijk omdat de belangen in Nederland ook nationaal zijn. Immers, half Nederland ligt beneden de zeespiegel en een overstroming zou dan al gauw landelijke proporties krijgen.
In het buitenland is de situatie meestal heel anders, alleen een land als Denemarken komt in de buurt. In de VS, waar overigens felle tegenstanders van suppleren zijn, financiert de federale overheid alleen suppleties die ten doel hebben de veiligheid tegen overstroming te garanderen of te herstellen na bijv. een orkaan. Strandsuppleties t.b.v. de recreatie dienen door de betreffende staat of gemeente te worden betaald. Soms wordt een gecombineerd programma uitgevoerd. Veel landen kennen geen erosieproblematiek of hebben daar nauwelijks hinder van, en suppleren enkel t.b.v. de strandrecreatie en daarmee gepaard gaande inkomsten. Dit gebeurt een enkele keer via een nationaal stimuleringsprogramma maar meestal met regionale of lokale middelen. Omdat de meeste landen niet beschikken over makkelijk winbare grote hoeveelheden zand is men benauwd voor het verliezen van het aangebrachte zand. Daarom worden strandsuppleties vaak gecombineerd met off-shore golfbrekers (bijv. Japan, Spanje). Onderwatersuppleties worden elders ook wel toegepast, maar niet zo systematisch als in Nederland en, in mindere mate, Denemarken.
In het buitenland ontbreekt het vaak aan een kwantitatieve doelstelling van een suppletie en zijn evaluaties, indien al uitgevoerd, vaak van kortere duur en minder systematisch dan in Nederland. Lange (historische) meetreeksen van de betreffende kustsystemen ontbreken gewoonlijk.

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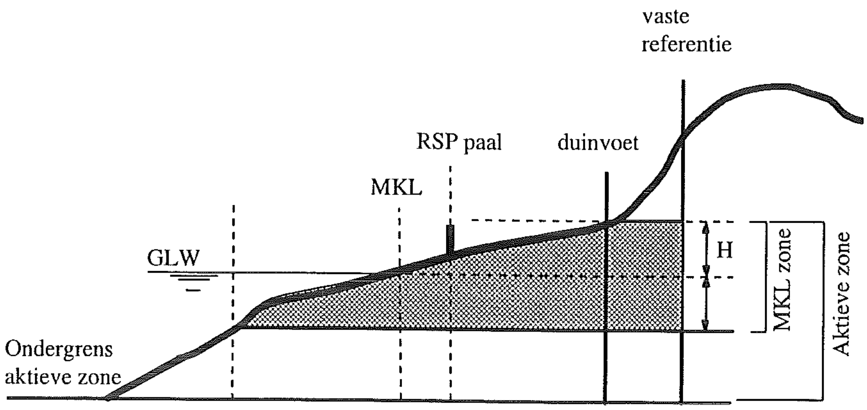
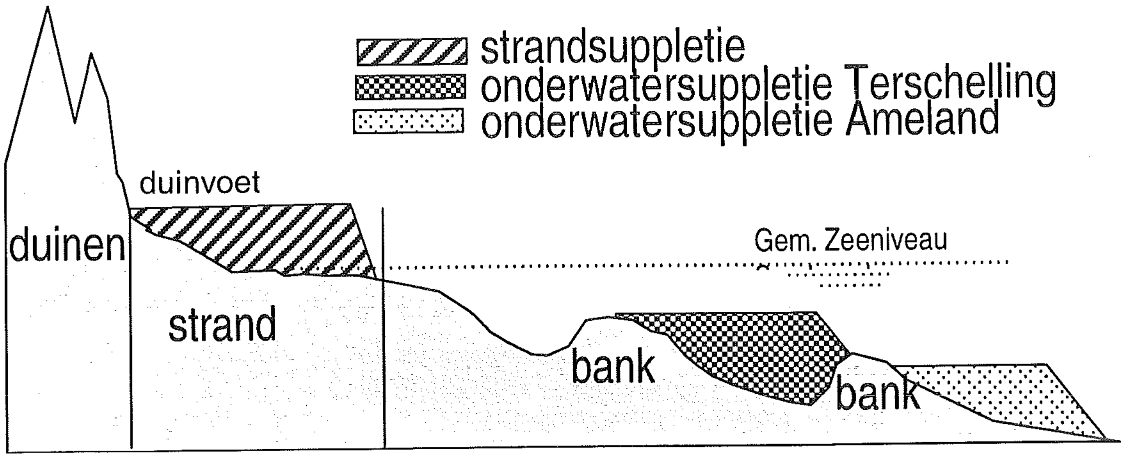
***Figuren*: in de handout bij de cursus worden diverse figuren opgenomen**



Figuur 1. Schets kustprofiel (dwarsdoorsnede) met actieve zone en MKL-zone
In de actieve zone wordt de kust opgebouwd. De ondergrens is het punt waar tussen de diverse lodingen geen veranderingen optreden (binnen de tijdschalen van interesse, typisch enkele decaden). Als bovengrens wordt vaak het niveau van de duinvoet gekozen. Het horizontale zandvolume in de laag (t.o.v. een vaste referentielijn) is een betere maat voor de kustlijn dan bijvoorbeeld de Gemiddeld LaagWater lijn (GLW). In de praktijk wordt niet ver genoeg uit de kust gemeten (JARKUS) en hanteert men het volume in de Momentane KustLijn (MKL)-zone als indicator. Deze schijf loopt van de voet van het duin tot tweemaal het hoogteverschil H tussen duinvoet en GLW. Het is dus denkbaar dat de MKL-zone erosie vertoont terwijl de actieve zone nauwelijks in volume verandert. Ook het tegenovergestelde kan gebeuren, nl. dat de actieve zone meer erosie zou kunnen vertonen dan de MKL-zone aangeeft (bij "versteiling"). De momentane kustlijn (MKL) in één opname volgt uit het delen van het zandvolume in de MKL-zone door diens hoogte 2H, en wordt uitgedrukt in een afstand t.o.v. de Rijks Strandpaal RSP. Omdat de MKL nogal blijkt te variëren over opeenvolgende jaren, toetst men de kustlijnpositie via een extrapolatie (naar het heden) van een regressie over de 10 voorgaande jaren, in plaats van met een enkele opname. Zo is ook de Basis KustLijn (BKL) voor het jaar 1990 meestal bepaald. Dreigt bij de toets de MKL de BKL te onderschreiden, dan dient in het beleid van Dynamisch Handhaven een suppletie overwogen te worden.



Figuur 2. Dwarsprofiel met 3 mogelijke suppleties.

Een conventionele strandsuppletie zoals getoond wordt gewoonlijk tot rond de waterlijn gelegd. Als daar te weinig zand kan worden neergelegd wordt soms ook zand op de aangrenzende vooroever gelegd. “Echte” vooroeversuppleties worden op dieper water geplaatst: bij Terschelling in de buitenste trog, terwijl latere zoals bij Ameland aan de zeezijde van de buitenste bank worden gelegd.

PAO cursus Innovatief denken en doen in kustverdediging
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EUROPEAN LEGISLATION: IMPEDIMENT OR CHALLENGE?

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SUMMARY
Careless handling of nature in coastal areas will lead to unacceptable situations and arising problems for planned developments. Neglecting the environment will confront planned developments with many impediments. These obstacles vary from not foreseen adverse impacts on nature, to resistance to planned projects and delays in project realization, from dramatic budget overruns to projects that will not be realized at all. Moreover many times coastal areas are subject to European legislation. To solve the problems a strategy should be developed and careful planning is needed. In this lecture the following items will be dealt with:

1 Ecological value of coast and tidal areas

Coast and tidal areas comprise salt and brackish water, dunes and beaches, sub tidal sandbanks and underwater soils. All these compartments contain a diversity of types of environment and landscape in its natural state ranging from wet to dry, from poor to rich in nutrients, from sparse to abundant vegetation. In terms of form, dimensions, dynamics and chemical composition the compartments of the coast display enormous differences. A feature of the coast is its highly dynamic character, with ebb an flood tides and fluctuating influence of wind and waves, thus causing fluctuating erosion and siltation.

These differences in abiotic factors are visible reflected in the great diversity in the nature of the ecosystems. Resulting from this dynamic environment very specific ecosystems are created which have become adapted to the special conditions of the coast and marine waters. Neither the banks nor the beaches and dunes are permanent. Caving in of dunes, soil erosions and natural sediment deposition ensure that the form of these areas changes in the course of time, resulting in a rich ecosystem.

It is also through this that different opportunities are created for man. The natural environment is vital for human survival. The importance of natural habitats cannot be overestimated. They provide emotional and aesthetic experiences. They offer outstanding opportunities for recreation. They clean our water, purify our air and maintain our soils. They regulate the climate, recycle nutrients and provide us with food. They provide raw materials and resources for medicines and other purposes. They form the foundation on which we build our societies. Human life depends on a complete and fully functioning ecosystem in which habitats are an important part. Any major reduction in its diversity may restrict the scope for future discovery and development. In other words: Biodiversity matters for ethical, emotional, environmental and economic reasons.

That's why it is essential to understand the response and activity of the ecosystem. The focus is often on the conservation and protection of certain species. Excessive attention for only birds or for larger animals are too often taken as the core activity of nature conservation. However, survival of these depends much more on the total environmental and ecological conditions by which the habitat, the ecosystem and the relationships of that species are determined. Exogenous factors largely determine the conditions within the ecosystem.

2 Biodiversity and risk of extinction

Biological diversity - or biodiversity - is one of the key terms in conservation, encompassing the richness of life and the diverse patterns it forms. Biodiversity forms the web of life of which we are an integral part and upon which we build our societies and economics, locally, nationally and globally. It is the result of billion of years of evolution which is increasingly affected by humankind. The Convention on Biological Diversity defines biological diversity as "the variability among living organisms from all sources including, inter alia, terrestrial, marine and other aquatic ecosystems and the ecological complexes of which they are part; this includes diversity within species, between species and of ecosystems".

Biodiversity is often understood as the great variety of plants, animals and micro-organisms. At present it is still very much unclear how many species exist globally. So far about 1.7 million animal and plant species have been described scientifically. However, extrapolations about the total species number vary between 2 and 50 million (sometimes even up to 100 million). Most biologists consider an estimate of about 5 to 15 million to be realistic. Biodiversity encompasses the variety of ecosystems such as those that occur in oceans and coastal areas, rivers, lakes and wetlands, mountains, deserts and forests as well as in agricultural landscapes. Biodiversity describes the variability among living organisms of different origin, including terrestrial ecosystems, marine ecosystems and aquatic ecosystems, including the ecological complexes of which they are part of and the diversity within species and between species.

Diversity provides the ecosystem with a built-in control system to maintain sufficient genetic capacity to allow for adaptation and timely adjustments of the ecosystem to changing environmental conditions and system fluctuations. Disruption of the ecosystem by reducing biodiversity can result in an increased dependency on human input and management efforts to keep the system running and to maintain it on a desired output level. Biodiversity is not just a matter of a number of species found in an area. First of all the relations within and between species and the relations between the various habitats are important. Apart from these relations the valuation of a good functioning ecosystem depends of:

- The uniqueness and significance of the ecosystem
- The presence of rare species
- The contribution to biodiversity
- The provision of habitat for wildlife

Various types of biodiversity can be distinguished:

- α the number and variety of species in one habitat
- β biodiversity, that is the variation of different habitats in a larger area
- γ biodiversity, that is related to the differences of comparable habitats in a region

Alpha or habitat diversity is strongly related to the variations in geomorphology and hydrology of the site. It describes the number of species, the number of populations as well as the genetic diversity of the population in the habitat. It will be clear that alpha diversity on a tidal flat or floodplain meadow is much lower than that in a humid tropical forest. However, the beta diversity within 1 km from tidal flat to salt marsh and dunes and cliffs is rather high. In a tropical forest the differences over 1 km are rather low. Gamma diversity is of importance when some species are becoming extinct and when re-colonisation from comparable habitats might occur.

If environmental conditions are subject to fluctuations the ecosystem needs to mobilise those species and life forms with a wide tolerance and strong adaptive capacities in order to sustain functional ecosystem processes. Stable conditions favour the dominance by a smaller number of species, which all have a very specialised and efficient resource use. Changing conditions favour the increasing appearance of at first marginal genetic varieties and populations or species. Both trends are of importance for the overall stability and existence of ecosystems.

Europe hosts a unique set of natural diversity, including hot spots like the Mediterranean and the coastal areas round the Wadden Sea and the Dutch islands. The 10 new Member States bring new biodiversity riches to the EU. However, biodiversity loss has accelerated to an unprecedented level, both in Europe and worldwide. It has been estimated that the current global extinction rate is 1000 to 10000 times higher than the natural background extinction rate. In Europe some 42% of European mammals are endangered, together with 15% of birds and 45% of butterflies and reptiles.

Extinction of species might occur if:

- Populations are too small
- Habitats are too small or fragmentized
- Species are isolated
- Habitat destruction

Fragmentation of habitats and landscapes often leads to the isolation of populations and parts of an original population. The consequences are illustrated by the following example. A few patches of remaining dry areas originally housed small sub-populations of lizards. These where in contact with each other by small corridors. In this way genetic drift, inbreeding and decreasing genetic variety were frequently corrected once roaming young male lizards moved from one dry area through the corridors to another dry area. Flooding of the corridors would mean that the dispersion corridors would disappear and that every single small sub-population in each island would become isolated and highly susceptible to extinction.

This is part of the islands theory. On very small islands the number of different habitats is low. In other words the beta diversity is low. So genetic variety and input from out breeding is reduced and speciation is hampered. As a result the small island can only have small populations and thus a higher risk of extinction. The further the island is away from large populations the lower the chance of immigration of new species. The bigger the island more species can be introduced and the risks of extinction will be reduced. Small nature reserves are like islands far in the ocean. The risk of extinction can be reduced by corridors between the reserves to prevent isolation and fragmentation. In this way an ecological network is created which supports dynamic processes at systems scale and overarches and connects specific intertidal sites. That is the main reason why an ecological infrastructure in the form of the Natura 2000 network aims at providing a stable ecological system. This will create foraging, spawning and breeding grounds for different species by connecting water and terrestrial areas.

3 Sustainable development and European legislation

3.1 Sustainable development

In Rio de Janeiro in 1992 the countries of the world agreed upon some basic principles of nature conservation. The agreements were laid down in the so-called Convention on Biological Diversity. After many discussions the following definition of sustainable development was accepted:
"Sustainable development is a development which meets the needs of the present without compromising the ability of future generations to meet their own needs".

This concept of sustainable development reflects the core of human efforts. Not only you and me, but also our children and grandchildren have the right to live in a sound natural environment. The principle of sustainable development has lead to a series of international treaties and conventions. These are continually being updated and developed as scientific knowledge increases. The countries that are contracting members to the Convention have to comply with the obligations of the Convention by implementing these in national legislation. It is for the sake of the interests of future generations that important nature areas, like wetlands and areas with important cultural heritage must be protected.

3.2 European legislation

European policy for the marine and coastal environment has been embedded in policy for nature conservation and strategy for the marine environment. Common policies like that for fisheries or agriculture or chemicals and waste are incorporated in the policy. The water framework directive and the directive on the transport of dangerous goods have their influence too. The European policy should comply with international agreements and conventions. This is illustrated in Figure 1.

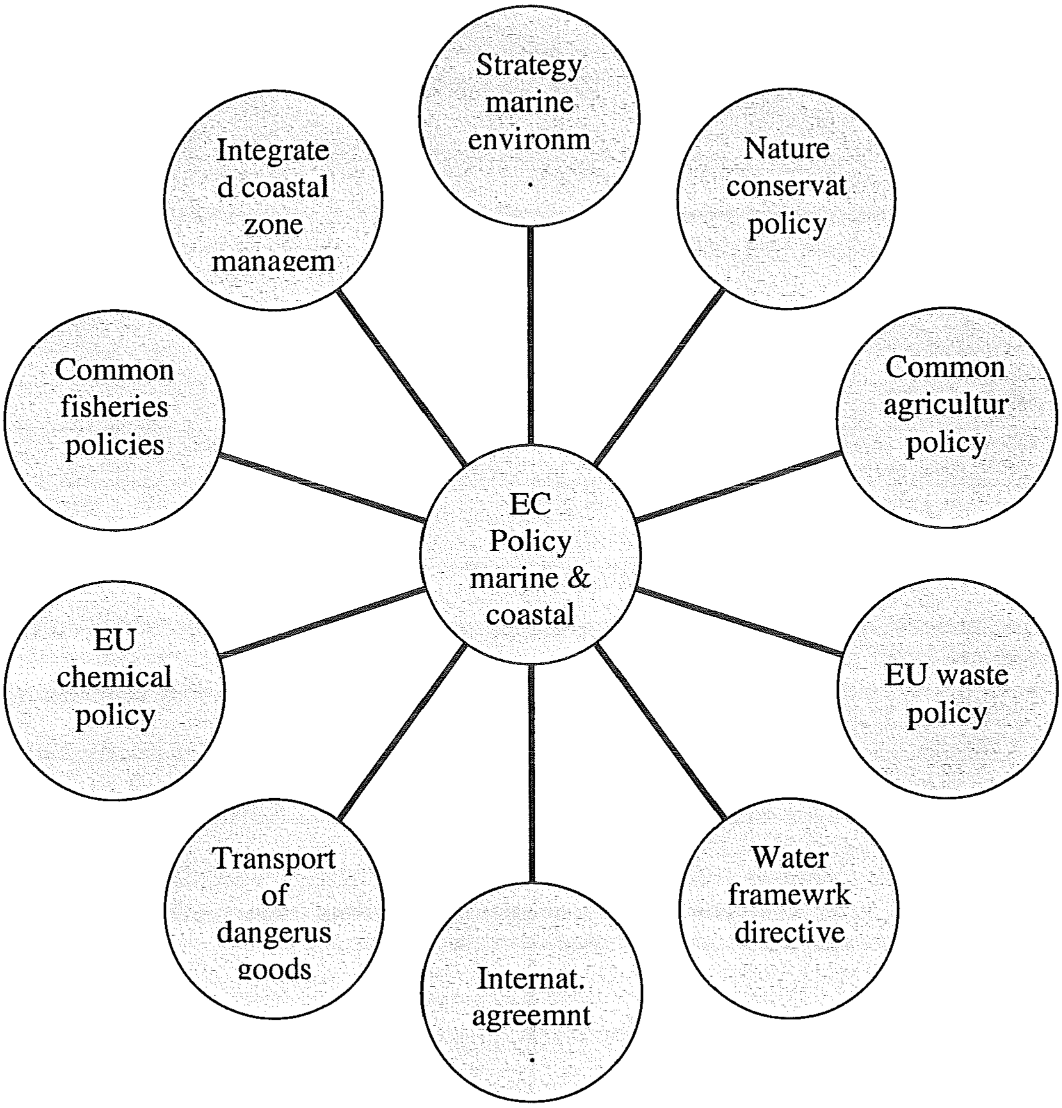

Figure 1: EU policy on marine & coastal protection

The result is that the European Union has launched many directives related to the environment. Directives must be implemented by the laws or regulations of the member states within a designated time limit. The directive is the main tool of Community environmental policy. It empowers the Community to define objectives, standards and procedures but allows the member states some flexibility in that implementation must take place through national legislation and regulation. That is however, also the main reason why in the various member states some difference in the interpretation of the legislation exists. An overview of the most important

3.2.1 Birds Directive

The birds directive mention a number of wild birds to be the subject of special conservation measures concerning their habitat in order to ensure their survival and reproduction in their area of distribution. The aim of the Directive, which was adopted by the environment council as part of the second environmental action program of the EU, is to maintain, or restore, an adequate diversity and area size for all wild bird species native to the territory of the European Union member states.

Suitable measures shall be the designation of protected areas and the ecologically sound management of habitats within the protected areas and outside as well as the creation of new habitats. Many species are migratory birds whose protection is a trans-boundary task. In the context of BirdLife International, Important Bird Areas (areas of worldwide importance) have been selected and described with harmonised criteria. These criteria refer to:

- Populations of migratory birds of international importance using these areas for breeding or resting, or hibernating.
- Areas selected according to these criteria are also wetlands of international importance according to the Ramsar Convention.
- Populations of species under threat worldwide.
- Species of European Conservation Concern.

The list of Important Bird Areas constitutes an appropriate basis for the selection of Special Protection Areas for Birds as stipulated in the Birds Directive.

3.2.2 Habitats Directive

The aim of the Habitats Directive, which was adopted in 1992, is the conservation of natural habitats as well as of wild fauna and flora, and thus, the conservation of the diversity of species and habitats in Europe. The obligations of the Bern Convention have thus become embodied in a legal instrument of the EU. To ensure the conservation of species and habitats, there are bans on the killing, catching and disturbing of strictly protected animal and plant species.

The habitat directive aims both at protection of natural habitat as well as protection of species. In the annexes are listed:

- Annex 1: Natural habitats of community interest whose conservation requires the designation of special areas of conservation,
- Annex II: Animal and plant species of community interest whose conservation requires the designation of special areas of conservation,
- Annex III: Criteria for selecting sites eligible for identification as sites of community importance and designation as special areas of conservation,
- Annex IV: Animal and plant species of community interest in need strict protection,
- Annex V: Animal and plant species of community interest whose taking in the wil and exploitation may be subject to management measures.

3.2.3 Water Framework Directive

The Water Framework Directive

- Protects all waters - rivers, lakes, coastal waters, and groundwaters.
- Sets ambitious objectives to ensure that all waters meet "good status" by 2015.
- Sets up a system of management within river basins that recognises that water systems do not stop at political borders.
- Requires cross border co-operation between countries and all involved parties.
- Ensures active participation of all stakeholders, including NGOs and local communities, in water management activities.
- Ensures reduction and control of pollution from all sources like agriculture, industrial activity, and urban areas, etc.
- Requires water pricing policies and ensures that the polluter pays.
- Balances the interests of the environment with those who depend on it.

3.2.4 Natura 2000

To ensure the conservation of species and habitats, there are bans on the killing, catching and disturbing of strictly protected animal and plant species. This important legal instrument for nature conservation at EU level stipulates the development of a Europe-wide network entitled "Natura 2000". A series of informal working groups with member states and stakeholders groups study specific issues relating to this unique European network.

3.2.5 Ramsar Convention On Wetlands

Though not a European directive the Convention on Wetlands of International Importance should be mentioned as well. The Convention was accepted in 1971 in Ramsar in Iran and thus the Convention was the first of the global intergovernmental treaties on conservation and wise use of natural resources. The wetlands convention lays down the principles of wetlands conservation and wise use. The contracting parties to the Ramsar Convention commit themselves, among other things, to the following obligations:

- List the own locations of particular interest on the basis of its ecology, botany, zoology, limnology or hydrology.
- Designate at least one site that meets the Ramsar criteria for inclusion in the List of wetlands of international importance.
- Include wetland conservation within their national land-use planning and promote the conservation of wetlands in their territory.
- Develop national wetland policies. Any loss of wetland should be compensated for by creation of new habitat.
- Establish nature reserves on wetlands and promote training.
- Consult with other contracting parties about the implementation of the Convention, about transfrontier wetlands, shared water systems, shared species and development aid for wetland projects.

3.2.6 Environmental Impact Assessment

The Directive aims to ensure that environmental consequences of projects are identified and assessed before development consent is given. The public and environmental authorities can give their opinion and the results of consultations and the information gathered pursuant the Directive's articles are taken into account in the authorisation procedure of the project. The Directive outlines which project categories are subject to an Environmental Impact Assessment, which procedures must be followed and the content of the assessment. The provisions on public participation have been aligned in accordance with the Aarhus Convention by the Directive 2003/35/EC.

3.2.7 Commission Communication on the 'Integrated Coastal Zone Management

This document presents a series of conclusions and recommendations that are based on the results of the EU Demonstration Programme on ICZM. It addresses the problems that European Coastal Zones are faced with and calls for an integrated, participative territorial approach to ensure that the management of Europe's coastal zones is environmentally and economically sustainable, as well as socially equitable and cohesive. The overall role of the EU is to provide leadership and guidance by establishing a framework to enable action at other levels by:

- Promoting ICZM within the Member States and at the "Regional Seas" level;
- Making EU policies compatible with ICZM;
- Promoting dialogue between European coastal stakeholders;
- Developing best ICZM practice;
- Generating information and knowledge about the coastal zone, and
- Diffusing information and raising public awareness

4 Impacts of mining and beach nourishment

Dredging for the purpose of sand extraction and beach nourishment can be performed in various ways. These activities should be carried out in a sustainable way. The emphasis of the activities of sustainable dredging is related to management of the dredged material in such a way as to prevent disturbance as much as possible and to derive environmental benefits of the dredged material.

Impacts of dredging activities might be:

- Disturbance of habitat in the mining area,
- Turbidity in sensitive areas as a result of the dredging operations itself,
- Changes in hydrologic and morphologic properties of the area as a result of the works carried out,
- Burial of benthos and food areas of fish and migrating birds,

Disturbance
Sand mining will kill the organisms living on and in the sediment, the benthos. These organisms can be divided according to their feeding type in deposit feeders, suspensions feeders, filter feeders or predators. When dredging stops new species will enter the disturbed area. The first species that colonise the area are pioneer species that are able to adapt to the newly created environment. These species will make it possible for other species to colonise the area afterwards. These species in succession will create food possibilities for other species and so forth. During time there is an increase in succession. The time needed for recolonisation of the area depends on various factors.

Turbidity
As a result of sand mining operations sediments will be suspended causing turbidity. This will influence the light penetration and reduce the productivity of the aquatic life. In other cases the turbidity caused by dredging must be controlled when the suspended sediments disperse and settle down on sensitive aquatic ecosystems (e.g. coral reefs, oysters or other shellfish). Concern may be expressed that the reef structure or the shellfish production will be permanently damaged. Many dredging techniques have been developed to reduce turbidity levels.

Morphologic and hydraulic changes
After completion of the mining activities the water depth and other hydraulic and morphologic properties of the area will be changed. These changes will have impact on the environmental dynamics. Fluctuations in circumstances will influence the development, grow, reproduction and survival of species. In natural circumstances too, the number of species fluctuates depending on environmental conditions. In a stable situation oscillations of the population occur. By feedback and control mechanisms the number of species fluctuate around a mean value. The ability of the eco-system to survive mining depends on the rate of disturbances by dredging and other human activities.

Burial of benthos
Placing fine grained dredged material onto beaches and sandbanks before the coast results in the short tem in the loss of an ecologically important habitat. Benthos, the flora and fauna of the bottom of the sea play a crucial role in biochemical processes. Beaches and coastal areas are rare and specialised habitats, which are vulnerable to man-made as well as natural influences. Recovery of macro-invertebrates from dredged material placement on the intertidal occurs via a combination of 3 main processes (Bolam et al). Each of these processes becomes more or less important depending upon factors such as the depth of the deposited sediment layer, properties of the deposited sediment, spatial area of placement and the time of placement.

- Planktonic recruitment will be the predominant mechanism if the material becomes suitable for recolonisation shortly prior to the main annual recruitment phase for most species (i.e. spring).
- Lateral migration, primarily by post-juvenile stages will be the main recovery process during other times.
- Burial survival can be a main recovery process if the amount deposited is not too great.

5 Stakeholders and their interests

Apart from the conservation goals of coastal areas a number of interested parties can be identified which have their own goals. In case of coastal areas it is obvious that safety and recreation are important. However, some other parties are important as well. It is essential to distinguish between the economical value of coast and tidal areas and the ecological value of these. In this lecture, the ecological function is defined by the dynamic physical, biological and chemical processes that give rise to its overall structure and function, whereas the economical value is judged in terms of end user needs. The economical value of coastal and intertidal areas are determined by many social and recreational values. Economic values that can be identified are:

- Safety. The first priority of coastal areas is to protect land from erosion by storms and currents.
- Water intake for urban or industrial purposes.
- Drainage of sewage and waste water.
- Fishery. Both commercial and recreational fisheries.
- Recreation and nature experience. One of the most important social-environmental items is the valuation of environmental experience in nature. Coast and dunes are extensively used by the public for bathing, walking, digging bait, sailing, surfing and bird watching.

- Navigation and industry. The coast hosts harbours and access channels to various ports, having commercial and industrial functions.
- Bird watching. Many people enjoy the many birds feeding in dunes and beaches.
- Agriculture and housing. Some areas in the coastal zone have been claimed for the purpose of civilisation.
- Research and education.

Many times the interests of the various stakeholders are conflicting with each other. This urges for ways to solve the problems resulting from these controversies. A strategy should be developed to deal with these items and to derive a policy in which the views and interest of NGO's and other stakeholders have been integrated. Figure 2 shows the steps of the strategic plan. It also shows two different lines of interaction. Those of the politicians, policymakers and other regulatory parties on the one hand and those of the stakeholders, NGO's and other interested parties on the other hand. Consideration of the pros and cons against each other should be incorporated in each step of the strategy. This is not an easy process. The challenge is to reach an agreement between the various economic, social and ecological interests. Involving interested parties at an early stage will allow each party to bring in their concerns and experiences. This helps to ensure that steps are taken and studies are made in accordance with the requirements of stakeholders and decision makers. In this way the management of coastal areas gets the opportunity to develop and resolve issues in a constructive manner rather than possibly provoking a formal confrontation at a public enquiry.

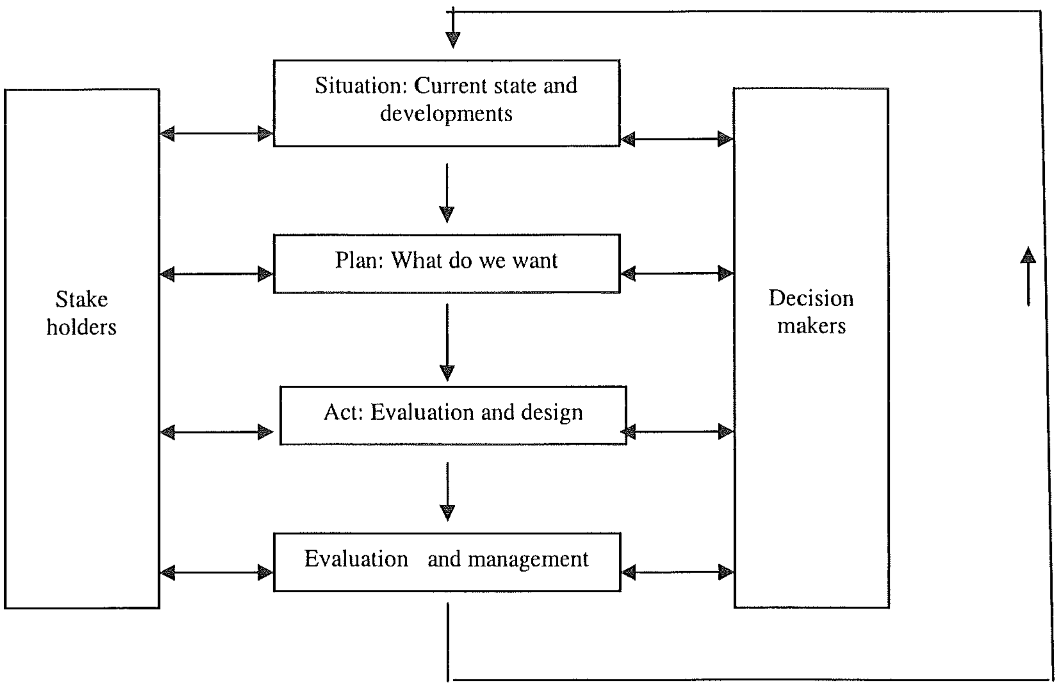

Figure 2: Steps in a strategic plan

6 Steps of a strategy

A strategic plan is an excellent means to assure the balance between coastal defence and environment. It provides an overview of the realistic goals, the means to reach these goals, the planning time and the resources needed. It incorporates the results of one or more Environmental Impact Assessments and other related studies. In this way a strategic plan is also an essential element in obtaining finance and all legal permits, which are necessary for implementation. Another important function of a strategic plan is that it lays the foundation for communication to the public and it is the basis for an agreement between the parties involved. All relevant interested parties should support the strategy. We should not forget that a favourable judgment of the plan and agreement between the various interested parties will prevent infringement on existing operations and future developments.

PIANC-Envicom working group 7 has studied ways to develop a strategy to solve these problems for wetlands. The work was based on the international standard for environmental quality ISO 14000. Though coastal areas have their own specific problems the strategy developed can also be used for coastal and tidal areas. The working group distinguished the following steps:

6.1 Overview of current situation

The first step in strategic planning is the making of an overview of the existing condition and the impact of human activities on the environment. Knowledge of the facts, data and quantification of the current state form the baseline of each project. Both the economic dynamics of the site as well as the ecological dynamics are important. There are items related to the a-biotic environment, defined as the physical and chemical conditions. These are important in relation to pollution control and to aspects of health and safety. Important too is the biotic environment. So an overview of the ecological values and the process dynamics of the area is necessary. These are related to the protection of natural resources and habitats. And finally we should not forget the human environment and the quality of life of the people who live, work and have their leisure time on beaches, dunes and in the sea. Management plans should involve the local stakeholders considering their requirements and regularly evaluating and revising these as necessary. National and international legislation plays a role too. Analyses and evaluation of the situation leads to the identification of constraints and bottlenecks to overcome. It gives insight into the required situation to strive for.

6.2 Setting goals and objectives

The second step of strategic planning is: Plan the required situation. That means setting goals and objectives, while taking into account the coastal functions and values. A realistic problem definition, as defined in the foregoing stage, is essential to set goals and objectives. In this stage communication with decision makers and stakeholders is essential, too. Various participants should agree on prioritized objectives and the ways to reach them. Moreover, objectives must be feasible, testable and verifiable. In this stage, several alternatives should be developed and feasibility studies should be carried out in order to assess the economic, technical and environmental feasibility of the various alternatives. The identification of the goals and objectives to implement must be related both to the socio-economic functions, as well as to the defined ecological functions and values of the area. In addition the legislative obligations must be taken into account. Setting the objectives is one of the most difficult activities of strategic planning. There is not one approved manner to set goals. That depends largely on the local situation. Different situations can exist:

- Are natural characteristics disturbed in the current state; is there a need for improvement of the existing situation? The objective should be focused on a better management of the wetland and a better integration of navigation purposes and ecological purposes.
- Will natural characteristics be disturbed in the future when navigation developments take place; will the disturbance be permanent or temporary? The objective should be focused on measures to be taken for temporary protection during construction and for permanent management of the wetland afterwards.
- Are possible measures not sufficient to compensate the impact of navigation development by good management? In this case managers should prove which social needs urge for these navigation developments. The objective should be focused on minimizing adverse effects by limiting the degree of action and on alternatives with less impact on the existing wetlands.
- Are there local, national or international legislative requirements relevant? The objective should be focused on compliance with all legislative requirements.
- Are there social reasons that necessitate developments adversely affecting a wetland? The objective should be focused on rehabilitating or restoring measures, on preservation or maintenance operations that could reduce or eliminate adverse effects. Studies and research should incorporate an environmental impact assessment. An alternative should be selected that assures that all possible measures are taken.

- Can't adverse effects be avoided; after all appropriate and practical measures have been taken? The objective should be focused on compensation measures to be taken by replacing or providing substitute resources or wetlands.

When all the work has been done the result will be a program that meets the quantitative and measurable requirements and has a series of prioritized goals. This plan has two ultimate aims: to produce an environmental strategy for the area that has the full agreement of all parties involved and that is achievable within the commercial and socio-economic possibilities of the organization and the locality. Emphasis must be placed on a transparent decision procedure in which the wishes of the community and the potential of the nature in the area must be coordinated. One of the most important aspects is the introduction of good effective lines of communication.

6.3 Act: design

Implementation of the program includes determining methods to reach the goals and to incorporate them into land-management strategies. In other words: Design, how to do it and Act: how to construct it. The key process of the design stage is the implementation of the planned measures, the draft of the required hydrology, the creation of the layout and landscape and its visual quality, the training of the people involved and the procedures to monitor and control. The large variety of environmental conditions connected with coastal areas requires that the local conditions be taken as a starting point for the technical design of coastal defence structures.

The key of nature development is that nature itself cannot be constructed. Therefore the design should address the conditions for the living species to survive, rather than the ecosystem itself. The environment should be created so that the desired ecosystem can function and regenerate in a healthy manner. The ideal goal is a self-sustainable ecosystem with minimal maintenance. By taking advantage of natural developments the cost will be minimized. Monitoring the natural developments and adapting the design according to the findings is good practice.

The deliverables of the design stage will lead to a project plan. This consists of a detailed design plus guidelines for the management for construction, operation and maintenance. The management plan should be focused on the required quality, the time table and the budget.

6.4 Act: construct

The key process in the construction stage is the realization of the required situation. Implementation of the activities mentioned in the project plans will lead to the construction of the defence structure and nature areas. Monitoring and control of the environmental impact during construction should be an integrated part of the program of supervision. A clearly defined organization structure with training and procedures for the workforce to follow should be established. The principles of adaptive management should be applied to the projects in order to be able to accommodate to unforeseen developments and to take advantage of newly acquired knowledge or resources. At the conclusions of the work the project completion report will be prepared. It will contain data on the environmental impact on the nature area, both before and during construction. The completion report will enable review of the management plan for long-term stewardship of the nature area. Relevant training and ensuring good effective lines of communication must be introduced.

6.5 Evaluation of success

And that brings us to the final stage of the strategy: The post-construction site management and the evaluation of success. An environmental assessment and feedback to the result of mitigation measures and other activities must be carried out. If the results of the review indicate that the methods are not achieving the required goal, measures should be taken to improve performance to the required level. This feedback loop can continue until the environmental audit indicates that the goals have been achieved. A regular time interval for this feedback is recommended. Attention should be given to an approach making use of reference and target situations. This provides a policy tool for translating the ecological data from the past and present into practical and testable objectives. Such references and ecological starting points may also give others a more objective picture of the choices made.

The process of management should however, not be to strict. It should be regarded as a continuous, long-term process. Managers must adopt a flexible approach that will allow them to respond to the legitimate interests of stakeholders and adapt to the uncertainties of the nature world. The planning process has to be adaptive and dynamic. The plan must change or evolve to meet varying factors both on and off the site. The performance criteria developed should be based on sustainable development with a minimum maintenance. This type of adaptive management is a good example of sustainable development.

7 Conclusions

The above steps of the strategic plan lead to a set of guiding principles to be considered during planning, design and execution of the works:

- Sustainable development. The aim is to integrate sustainable development of navigation infrastructure and wise use of wetlands. That is a development which meets the needs of the present without compromising the ability of future generations to meet their own needs.
- Balance between ecology and economy. The ambition is an equal balance between ecological and economic objectives. If we don't achieve the balance that we need, the outcome might become disastrous.
- Participation from the very beginning. The process of strategic planning must involve communication. The plan is a mean of presenting information in an accessible format. The process should be recognized as a forum for resolving conflicts. Participation of stakeholders at the first onset will resolve issues in a constructive manner.
- Prevention is better than cure. Avoid destruction or degradation of important wetland functions and values. This holds for waste dumpsites and other polluted wetlands, for the consequences of salt intrusion, for the uncontrolled use of pesticides, but also for the over-exploitation of wetlands.
- Ecological values benefit socio-economic values. Restoring ecological values will benefit socio-economic values. Any major reduction in environmental diversity may restrict the scope for future discovery and development.
- Comply with the requirements of conventions. Where large infrastructure works are to be undertaken, national and international legislation require that mitigation and compensation be considered. Ramsar convention, European law and other conventions set guidelines and principles that must be followed.
- Show and evaluate all impacts. Communicate in a transparent way the outcomes of studies and assessments of the impacts with the interested parties.
- Sound management. A good organizational structure and sound management is essential for all stages of wetland restoration. This is of particular importance in the planning stage during the development of the goals and objectives.
- Strategy is a must. A good strategy is the key to success.

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Algemene principes van functioneel en technisch ontwerp van kustverdediging

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Inleiding
In deze bijdrage zal ingegaan worden op een aantal algemene aspecten. De focus ligt op constructieve oplossingen, oplossingen in zand worden elders in deze cursus besproken. Ook wordt de term "functioneel ontwerp" in dit kader beperkt tot de constructie zelf. Er wordt dus vanuit gegaan dat in een kustmorfologische analyses (zie IK4 en IK5) is al geconcludeerd is dat een "harde" kustverdedigingsoptie noodzakelijk is.

Voor dat aan het functioneel en technisch ontwerp begonnen kan worden moet er wel het nodige voorwerk gedaan zijn. Zo moet het project onderdeel zijn van een bredere visie over intergraal kustbeleid (Integrated Coastal Zone Management). Uit het kustbeleid zou een integrale visie van de ontwikkeling van de kustzone moeten volgen. Dit is meer dan alleen het kustbeleid van een Ministerie van Verkeer en Waterstaat. Ook de andere ontwikkelingen in de kustzone horen daarbij, zoals stimulering van het toerisme en natuurbescherming, maar soms ook wat buitenissige belangen als die van een schietterrein of een Hoogoven. Het blijkt dat in de meeste landen dit soort integrale afwegingen moeilijk is, omdat ze niet bij één ministerie onder te brengen zijn.
Meestal wordt het probleem opgelost door één ministerie in deze zaken een leidende rol te geven, maar dat geeft onherroepelijk ook aversies bij die ministeries die niet leidend zijn.

Maar als gevolg van zo'n visie wordt besloten om bijvoorbeeld een kustlijn op z'n plaats te houden en kan er een afweging volgen welke methode daarvoor het beste is. Uit zo'n analyse kan volgen dat de meest aantrekkelijke optie zandsuppleties is, maar er zou ook uit kunnen volgen dat een harde verdediging wat meer geschikt is.

Bij harde verdedigingen kan onderscheid gemaakt worden tussen verdedigingen die vast met de kust verbonden zijn, en offshore constructies. In IK12 en IK 13 zullen deze nader uitgewerkt worden. In deze cursusbijdrage wordt ingegaan op de algemene principes van het functioneel en technisch ontwerp van beide soorten harde verdediging.

Randvoorwaarden
Zoals bij ieder project, zal er ook bij een kustverdediging eerst gekeken moeten worden naar de verschillende randvoorwaarden. In deze bijdrage worden de volgende randvoorwaarden nader beschouwd:

- Hydraulische randvoorwaarden
- Ecologische randvoorwaarde
- Sociale randvoorwaarden
- Kosten

Deze zullen in het navolgde wat meer in detail besproken worden. Verder moet er rekening mee gehouden worden dat de uitvoering van het werk zijn eigen randvoorwaarden meebrengt. Een zware breuksteenconstructie of zware betonunits toepassen bij een verdediging voor een oud kustplaatsje betekent meestal in de praktijk dat e.e.a vanuit zee moet worden aangelegd, omdat het niet toegestaan wordt om met zwaar materieel door het plaatsje te trekken. Een voorbeeld hiervan is de kustverdediging die pas uitgevoerd is bij Scarborough in Engeland. Voor deze verdediging was zware breuksteen uit Noorwegen nodig. Omdat er geen werkverkeer door het stadje mocht rijden is alle breuksteen per ponton aangevoerd uit Noorwegen, op zee overgeslagen op kleinere pontons, en die zijn op hun beurt weer tijdens hoogwater boven het strand gelost. Bij laag water kon de stenen vervolgens van het strand worden weggehaald om in het werk geplaatst te worden. Voor de aanvoer van de Accropodes is een vergelijkbare operatie toegepast.

Hydraulische randvoorwaarden
De belangrijkste hydraulische randvoorwaarden zijn waterstand en golven. Voor beide grootheden geldt dat zij altijd beschikbaar komen in de vorm van overschrijdingslijnen. Dat betekent dus dat er eerst een kans van voorkomen gegeven moet worden voordat de juiste ontwerp golfhoogte, cq. waterstand bepaald kan worden. Deze overschrijdingskans is gerelateerd aan de te accepteren kans op schade, de te accepteren kans op bezwijken en de te accepteren kans op vervolgschade. Over het algemeen kan de te accepteren kans op schade en de te accepteren kans op bezwijken berekend worden uit een optimalisatie. De gekapitaliseerde kosten van aanleg en onderhoud (reparatie van de schade) en de gekapitaliseerde kosten van aanleg en herbouw (reparatie na bezwijken) kunnen relatief eenvoudig bepaald worden. In beide gevallen kan een minimum waarde gezocht worden, en dat punt is dus de optimale ontwerpwaarde.
Bij vervolgschade ligt dit vaak veel gecompliceerder. Vervolgschade is vaak niet alleen uit te drukken in geld, maar vaak komen er ook andere "kosten" bij, zoals schade aan het milieu, aan historische gebouwen, emotionele schade van bewoners achter de constructie, en wellicht ook mensenlevens. In dat geval is het vrijwel niet mogelijk om een economisch optimum te bepalen en zal er een politieke keuze gemaakt moeten worden. Tijdens dit besluitvormingsproces zal de politiek moeten bepalen hoeveel geld zij over hebben voor het beveiligen van cultuurwaarden, milieuwaarden, mensenlevens, etc.

Alle waarden in bovengenoemde berekeningen zijn natuurlijk nooit absoluut. Het zijn altijd verwachtingswaarden met een spreiding. Soms is de onzekerheid groot (bijv. in de te verwachten golfhoogte die met een kans van 10^{-5} per jaar optreedt), soms is die onzekerheid vrij klein (bijv. de afwijking in de soortelijke massa van zeewater). Als al die onzekerheden in rekening gebracht moeten worden kan een eenvoudige probabilistische som gemaakt worden. Een voorbeeld van zo'n som is gegeven in bijlage 1.
De conclusie uit dit voorbeeld is dat een zeer zware steenverdediging het beste is, maar als het gebouw minder kostbaar is, gaat deze conclusie niet meer op.

Uit de probabilistische analyse volgt ook dat de onzekerheden in de hydraulische randvoorwaarden overheersend zijn over de onzekerheden in de sterkteparameters. Dit betekent dat het wel belangrijk is om de variatie in hydraulische randvoorwaarden in de berekening mee te nemen, maar dat een gedetailleerd onderzoek naar de variaties in de sterkteparameters meestal niet lonend is.

Bij de hydraulische parameters is de golfhoogte in de meeste gevallen de doorslaggevende parameter; die moet dus zo zorgvuldig mogelijk bepaald worden. Helaas zijn daar vaak juist geen goede gegevens over beschikbaar (en zeker niet voor zeer kleine overschrijdingskansen). Bij kustwerken is vaak sprake van ondiep water. In die gevallen is de golfhoogte meestal gelimiteerd door de waterdiepte. Dan kan volstaan worden met de statistiek van de waterstanden (en die is in de meeste gevallen veel betrouwbaarder) en een berekening waarbij de maximale golfhoogte bepaald wordt uit de waterdiepte. Er wordt dan gebruik gemaakt van een formule in de vorm: $H_s = \gamma\, h$, waarin h de waterdiepte is (en die is heeft meestal een bekende kansverdeling) en γ is de brekerindex, die in orde van 0,5 is voor random golven. De brekerindex heeft natuurlijk wel een standaardafwijking. Voor een probabilistische berekening zou een σ_γ van ongeveer 0,1 aangenomen kunnen worden.

De periode is ook van belang, vaak wel iets minder relevant dan de golfhoogte, maar belangrijk genoeg om aandacht aan te besteden. Met name omdat door brekingsprocessen de golfhoogte sterk verminderd, veranderd ook het golfspectrum. De langere golfperiodes in het golfspectrum worden dan relatief belangrijk. Om deze langere periodes meer gewicht te geven wordt daarom aanbevolen om niet met de gemiddelde periode of de piekperiode van het spectrum te rekenen, maar met de periode die bepaald is uit het eerste negatieve moment van het spectrum. Deze waarde wordt meestal aangegeven met $T_{m-1,o}$.

In de bijdragen van Van der Meer en van Van Gent (IK11 en IK 17) zal veel dieper ingegaan worden op deze parameters. Voor standaard spectra kunnen de volgende omrekenfactoren gebruikt worden:

Goda:	$T_p=1.1\ T_{1/3}$
Pierson Moskovitch:	$T_p=1.15\ T_{1/3}$
Jonswap:	$T_p=1.07\ T_{1/3}$
TAW (vdMeer):	$T_p=1.1T_{m-1,0}$

Belangrijk is wel om te realiseren dat deze factoren gelden voor standaard spectra en standaard oevers. Juist omdat veel oevers niet standaard zijn, wordt geadviseerd om de $T_{m-1,0}$ te bepalen voor een bepaalde oever aan de hand van een berekening met een spectraal model (bijv SWAN, door het diepwater spectrum door te rekenen naar de teen van de constructie, en dan uit het rekenmodel het ondiepwater spectrum aan de teen af te lezen.

Kosten en baten

Voor een simpel probleem geldt dat de gekapitaliseerde som van kosten en baten geminimaliseerd moet worden. Echter, dit is alleen waar als:

- kosten en baten in vergelijkbare eenheden uit te drukken zijn;
- kosten en baten uit één pot komen;
- aan het eind van de 'hypothecaire levensduur' de constructie geen meerwaarde meer heeft.

Bij de praktische uitvoering van een kosten/baten analyse ontstaan een aantal problemen. Zo zijn vaak de baten heel moeilijk te kwantificeren in eenheden die goed meegewogen kunnen worden. Door de aanleg van een kustverdediging wordt de kans op erosie kleiner, en vaak ook de kans op overstromen. Hoe moet die verhoogde veiligheid nu meegewogen worden in een kosten/baten analyse? Dat is niet zo makkelijk omdat iedereen een andere perceptie heeft over wat veiligheid waard is. Ook zijn er waarden die moeilijk in geld uit te drukken zijn, zoals bijvoorbeeld de waarde van historische panden, een waardevolle biotoop of heeft iets een hele grote emotionele waarde. Dit speelt bijvoorbeeld bij een begraafplaats. In onderstaande foto is een begraafplaats langs de kust van Senegal weergegeven. De lokale vissers zijn gewend hun doden dicht bij zee te begraven. Maar toevallig is het een eroderend kustvak. Het gevolg is dat graven op het strand komen te liggen. Om dit de voorkomen is een hele kostbare langsverdediging gebouwd. Blijkbaar was de emotionele waarde van een graf voldoende hoog om deze hele grote investering te doen.

Strandmuur en langsverdediging bij een begraafplaats in Senegal

Overigens zou in dit geval het natuurlijk veel goedkoper geweest zijn om de begraafplaats een paar honderd meter landinwaarts te plannen. Er is in Senegal land genoeg; een goed ruimtelijk beleid had dit probleem dus kunnen voorkomen.

Een ander veel voorkomend probleem is dat er sprake is van verschillende geldstromen. Deze geldstromen worden vaak beheerd door verschillende organisaties of bestuurslagen en er is geen uitwisseling tussen deze stromen. Het gevolg hiervan is dat bijvoorbeeld de lasten bij een overheid liggen, terwijl de baten bij particulieren liggen. Als die baten ten gunste zijn van het algemene publiek is dat vaak niet zo'n probleem, maar als de bij een bepaalde groep ondernemers komt te liggen (bijv. bij strandtenteigenaren) dan kan dit vaak wel fricties geven. Ook komt het vaak voor dat voor aanleg en onderhoud verschillende fondsen gebruikt worden. Zo werd in het verleden in Zeeland de aanleg van strandhoofden voor 80% door het rijk betaald, 10% door de provincie en 10% door het waterschap. Een wezenlijke verandering van het dwarsprofiel werd gezien als nieuwbouw. Onderhoud was voor 40% voor rekening van het waterschap. Ontwerp en directie over de uitvoering was volledig in handen van het waterschap. Het gevolg hiervan was dat veel ontwerpen zodanig gemaakt werden dat de onderhoudspost minimaal werd. De aanleg werd daardoor vaak duurder dan de optimale waarde die volgt uit de gekapitaliseerde kosten van aanleg en onderhoud samen.

Gerelateerd aan het bovengenoemde probleem is de problematiek van het overdragen van een kustverdedigingsconstructie van de ene overheid naar de andere overheid. Van belang daarbij is de dekking van de onderhoudskosten, van het risico van schade aan direct betrokkenen en de eventuele claims van schade van de buren.

Ook speelt de politieke besluitvorming en de publieke opinie vaak een doorslaggevende rol. Alle bebouwing in en buiten de zeereep staat daar op eigen risico van de eigenaar. Dat betekent dat er juridisch geen enkele basis is voor welke vergoeding dan ook bij stormschade aan standpavil-joens. Toch is na zware afslag er grote politieke druk om toch financieel bij te springen in dat soort gevallen.
Ook een probleem bij de besluitvorming is dat een oplossing voor een bepaalde partij vaak leidt tot extra lasten voor een andere partij. Door het aanleggen van strandhoofden ontstaat bijna altijd erosie aan de lijzijde van het strandhoofdenvak. Het probleem is dus feitelijk afgewenteld op de buren.
Zeker in het geval van publiek private samenwerking, of in geval van zuiver private ontwikkeling moet er zekerheid zijn dat de initiatief nemende partij ook in staat is om zorg te dragen voor mitigerende maatregelen. Als bijvoorbeeld een initiatiefnemer failliet gaat, dan is er niemand meer om de kosten op te verhalen, en feitelijk draait de gemeenschap dan op voor de kosten van de gevolgen van het project.
Dit is niets nieuws. Als voorbeeld kan genoemd worden dat de Koloniale Overheid in 1095 een particulier mijnbouwbedrijf op Sumatra verbood om een haven bij Bengkulu aan te leggen. He argument van de overheid was dat de overheid niet verwachtte dat dit bedrijf op termijn levensvatbaar zou zijn, en de overheid wilde niet een haven bij Bengkulu in stand houden die commercieel absoluut niet rendabel zou zijn.

Tot slot een korte opmerking over de "hypothecaire levensduur". Bij veel economische beschouwingen worden de totale kosten gekapitaliseerd met de methode van de netto contante waarde tot één bedrag. In deze berekening zit een rentevoet. Bij een realistische rentevoet blijkt dat investeringen die nu gedaan worden, eigenlijk financieel gezien niet meer renderen na ongeveer 30 jaar. Het omgekeerde is dus ook waar, een hypotheek moet in zo'n 30 jaar afbetaald zijn, omdat het financieel niet aantrekkelijk is de looptijd langer te maken. Maar het gevolg van deze methode is dat alle investeringen feitelijk afgeschreven worden in een periode van zo'n 30 jaar. Een bij grote infrastructurele werken (bijv. inpoldering Haarlemmermeer, de Zuiderzeepolders, een Betuwelijn) zijn de nuttige inkomsten van belang over een veel langere periode. De inpoldering van de Haarlemmermeer heeft de eerste 50 jaar niet het verwachte geld op gebracht, en als je het 50 jaar na aanleg het rendement van deze inpoldering zou analyseren, dan zou je kunnen concluderen dat je dit gebied niet had moeten inpolderen. Maar of je dat na 150 jaar ook moet concluderen is natuurlijk nu een heel andere vraag. Zeker als je ziet dat de Haarlemmermeer nu één van de motoren van de Nederlandse economie is.

Effecten van kustwerken

In de Europese landen is het besluitvormingsproces voor grote werken steeds meer beïnvloed door effectrapportages. Voordat een besluit genomen wordt over een bepaald proces moet heel precies bekend zijn wat de effecten van dat project zijn en hoe de initiatiefnemer daar mee om denkt te gaan. Het gaat dan vaak over milieueffecten in de breedste zin van het woord. Het gevolg is dat een MER heden ten dage een kostbaar en tijdrovend proces is. Met alle beschikbare modellen wordt getracht een zo goed mogelijk beeld te schetsen van de mogelijke effecten van het project. Vaak is het zo dat de modelwerkelijkheid een grotere waarde krijgt toegemeten dan de echte werkelijkheid. Dit komt omdat veel modellen resultaten op een zeer overtuigende manier kunnen presenteren. Echter, in werkelijkheid zijn bijvoorbeeld de ingevoerde randvoorwaarden veel meer gevarieerd dan in het model. Dit verminderd de voorspellende waarde van modellen sterk.
De aanpak in een aantal Arabische staten is heel anders. Hier wordt een groot plan gelanceerd, en er wordt met name gefocussed op het ontwerp zelf, en in veel mindere mate op de effecten van het project op de omgeving. Dat betekent niet dat die effecten verwaarloosd worden. Men

gaat er vanuit dat in een latere fase, als de effecten zichtbaar worden, mitigerende maatregelen genomen kunnen worden. Zo zullen bijvoorbeeld de "Palmen" in Dubai gegarandeerd op bepaalde plaatsen kusterosie veroorzaken. En in plaats van nu gedetailleerd te proberen te voorspellen waar precies hoeveel erosie optreedt, heeft de Municipality van Dubai een gedetailleerd meetsysteem opgezet. Men kan dus waarnemen wat er gebeurt, en als er een ontoelaatbare erosie van een kustvak dreigt, is het uitvoeren van een zandsuppletie een snelle en doeltreffende maatregel. Natuurlijk moet er dan wel geld beschikbaar zijn, en moet er geen verschil van inzicht zijn tussen de eigenaar van het project zelf en de verantwoordelijke dienst voor de kustlijn. En in de politieke verhoudingen van Dubai op dit moment is dat ook niet zo.

Bijlage 1:
Voorbeeld van een probabilistische berekening van een oeververdediging
(Uit: G.J. Schiereck, Bed, bank and Shorelineprotection [2004] Delft University Press, ISBN 90-407-1683-8)

To illustrate the use of a traditional deterministic approach compared with the various levels of probabilistic approach, the following example are used, see Figure 0-1. A building is situated in the vicinity of the coastline and is protected by a rip-rap revetment, sufficiently high and including filter layers and a toe protection.

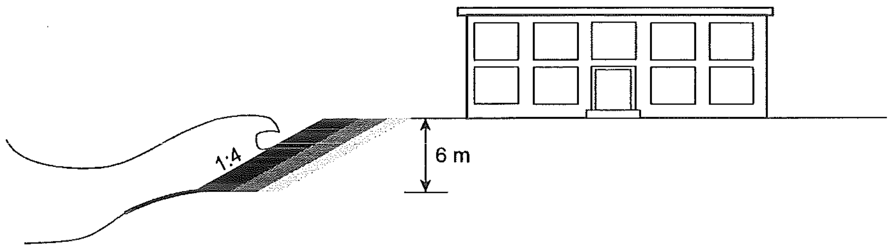


Figure 0-1 Example structure for comparison of probabilistic and deterministic approach

The idea is the following. Firstly, a design is made with a "traditional" deterministic approach (level 0). A characteristic wave height is chosen from the available wave observations and the threshold of motion is taken as a characteristic strength,. Subsequently, a fully probabilistic method (level III) is used to show what is a more realistic approach. Next, the results of level II and level I methods are shown to illustrate the merits of these methods. Finally, a level 0 approach is applied again, using the results of the probabilistic methods, showing that common sense always pays. The example is simple and is meant as an educational tool, not as a practical application.

Ten years of wave observations are available, see Table 0-1. Only waves > 0.5 m have been processed, as they are representative for "storms", lasting several hours. In these ten years, the highest recorded wave height was 1.62 m.

Table 0-1

Wave height interval (m)	**Occurrences in 10 years**	**Exceedances in 10 years**	**Return period (yrs)**
0.51-0.6	48	98	0.1
0.61-0.7	29	50	0.2
0.71-0.8	21		
0.81-0.9	6	20	0.5
0.91-1.0	4		
1.01-1.1	3	10	1
1.11-1.2	2		
1.21-1.3	2	5	2
1.31-1.4	1		
1.41-1.5	1	2	5
1.51-1.6	0		
1.61-1.7	1	1	10

Deterministic approach

With the highest recorded wave height, the stone size for the top layer is calculated with the Van der Meer formula, see chapter 8:

$$d_{n50} = \frac{H_{sc}\xi^{0.5}}{\Delta 6.2 P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}} \tag{0.1}$$

Swell does not play a role at this coast, so for the wave steepness a value of 0.05 can be used. With a slope angle of 1:4 this means that only the plunging part of the formulas has to be used. The permeability, P, for a revetment on sand ≈ 0.1, $\Delta \approx 1.6$ and the number of waves, N, is 7000 (maximum). The damage number, $S = 2$ is chosen, as it is representative for the threshold of motion. This leads to $d_{n50} = 0.56$ m and a choice of rock class 300-1000 kg ($d_{n50} \approx 0.6$ m).

This can be seen as an example of a classical deterministic approach. Now, several probabilistic methods will be used to establish the risk of failure of this structure.

Probabilistic approach

Probabilistic calculations will be done with the VaP package from ETH Zürich (see Petschacher,1994). In a probabilistic approach, a limit-state function has to be defined. For this example the Van der Meer relation is rewritten:

$$Z = 6.2\,P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi^{-0.5} - \frac{H_{sc}}{\Delta d_{n50}} \tag{0.2}$$

Note that it is not strictly necessary to separate strength and load factors. If this Z-function < 0, the structure fails. With the values from the deterministic approach we find $Z = 0.15$, slightly positive since we used a larger stone (0.6 instead of 0.56 m).

Probability distributions for all parameters are used in the computation of the total probability. So, firstly, these distributions have to be estimated. The wave height distribution is determined from the available wave observations, see Table 0-1 and Figure 0-2.

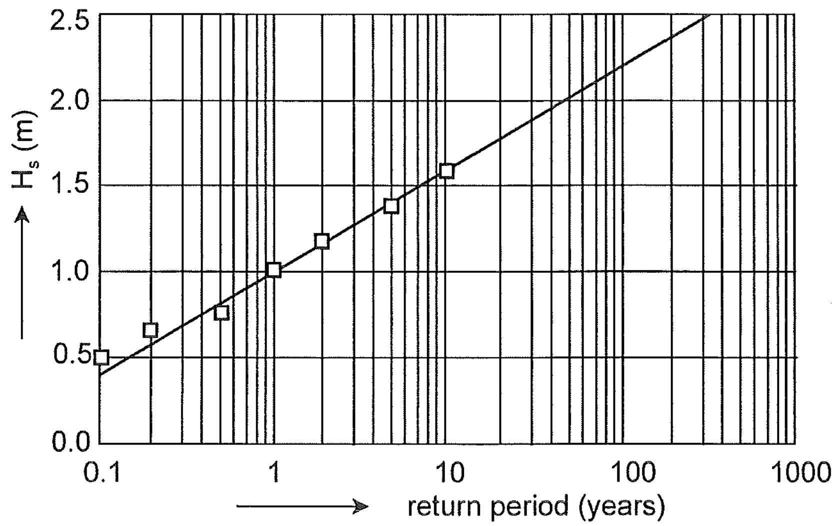


Figure 0-2 Long-term wave height distribution

In chapter 7 (Appendix 7.7.2) wave statistics were given for an irregular wave field. Such a wave field could be represented by the significant wave height, H_s, while the wave heights in a registration were described with a Rayleigh-distribution. So, H_s represents the wave condition at a certain moment, or better for a short period of one or more hours and the Rayleigh-distribution can be seen as the short term wave height distribution. All registered values of H_s give a distribution for the long term. This distribution is normally described with a Weibull-distribution, but often an exponential distribution gives reasonable results. Figure 0-2 shows an exponential distribution. This distribution has to be described mathematically to be used as input for the VaP-model.
The general expressions for the exponential distribution are:

$$\underset{\text{probability density}}{f(x)=\lambda\exp(-\lambda(x-\varepsilon))} \rightarrow \underset{\text{probability } X<x}{F(x)=1-\exp(-\lambda(x-\varepsilon))} \rightarrow \underset{\text{return period}}{(1-F(x))^{-1}=\exp(\lambda(x-\varepsilon))} \tag{0.3}$$

Note: *the function is defined only for $x \geq \varepsilon$ since negative probabilities are impossible, see second equation in (0.3).*

Figure 0-2 gives the return period of the wave heights, so the parameters λ and ε have to be derived from the third equation of (0.3). This can be done by taking two values of the line in Figure 0-2, e.g. for return periods of 1 and 10 years:

$$\left.\begin{array}{lll} 1=e^{\lambda(1-\varepsilon)} & \rightarrow & \ln 1=0= \quad \lambda-\lambda\varepsilon \\ 10=e^{\lambda(1.6-\varepsilon)} & \rightarrow & \ln 10=2.3=1.6\lambda-\lambda\varepsilon \end{array}\right\} \rightarrow \varepsilon=1,\ \lambda=3.83 \tag{0.4}$$

The parameter ξ is a function of H_s, T and tanα. T cannot be used as input parameter, because the period is a little bit correlated with the wave height. Therefore the wave steepness is used as input parameter.

For d_{n50}, Δ, $\tan\alpha$ and s, a normal distribution is assumed. The mean values for these parameters are equal to the ones used in the deterministic approach: 0.6 m, 1.6, 0.25 and 0.05, respectively. The standard deviations are estimated as: 0.05 m, 0.1, 0.0125 and 0.01, respectively. P is assumed to have a log-normal distribution, to avoid errors caused by negative values of P in the calculation. The mean value is 0.1 and the standard deviation is 0.05. N is given a deterministic value of 7000.
Note: *The normal distribution for d_{n50} is not a distribution curve within a stone class (e.g. a sieve curve). It represents the deviations in characteristic diameter for a whole mass of stones. Compare the Rayleigh-distribution within a wave record (characterized by H_s) and the long-term distribution of H_s*

This all results in the following input for the computation:

parameter	Distribution type	mean	σ
H_s	Weibull	$\varepsilon = 1$	$\lambda = 3.83$
Δ	Normal	1.6	0.1
D_{n50}	Normal	0.6	0.05
S	Determin.	2	
N	Determin.	7000	
P	Lognormal	0.1	0.05
s	Normal	0.05	0.01
$\tan\alpha$	Normal	0.25	0.0125

Level III

Numerical integration of Z is one of the available level III methods but will not be used here. Another method is the Monte Carlo method. The basis of this method is quite simple, see Figure 0-3. For all parameters, a random number is drawn, taking into account the probability distribution. This means that a value with a high probability density will appear more often. So, after many draws, the histogram of a normally distributed parameter will show the well-known Gauss-shape.

When all parameters have a value, the resulting value for Z is computed from equation (0.2). This whole procedure is repeated N times after which P_F simply is N_F/N (N_F being the number of times that $Z < 0$). The procedure is simple but the number of repetitions is very high, which makes the Monte Carlo method a computer job par excellence.

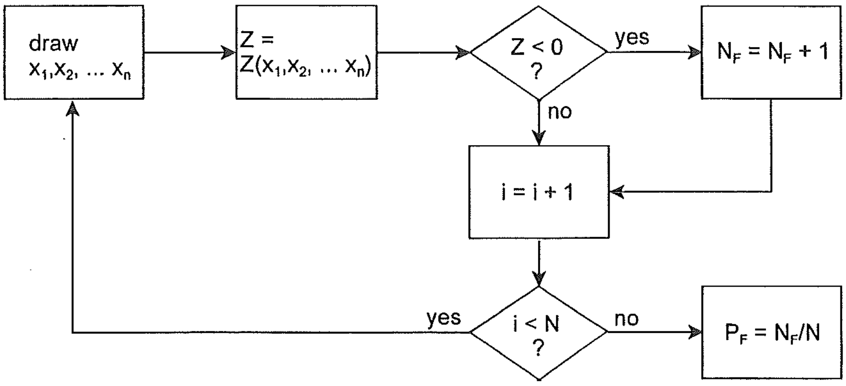


Figure 0-3 Procedure of Monte Carlo method

Computations with S = 2

Figure 0-4 shows two realisations of the Monte Carlo procedure for $S = 2$, resulting in $P_F = 0.095$ and 0.091, respectively. The number of draws, N, determines the accuracy of the method. A rough estimate of the necessary value of N is: N > $400/P_F$, so N has to be checked after P_F has been determined. An average, after three calculations, $P_F = 0.094$, which is the probability per year since the wave heights have been introduced as numbers of exceedance per year, see Figure 0-2. This value seems logical: the deterministic design was made with a 1/10 year wave (which has a probability of exceedance in one year ≈ 0.1) and a slightly larger stone was chosen (d_{n50} = 0.6 instead of 0.56 m).

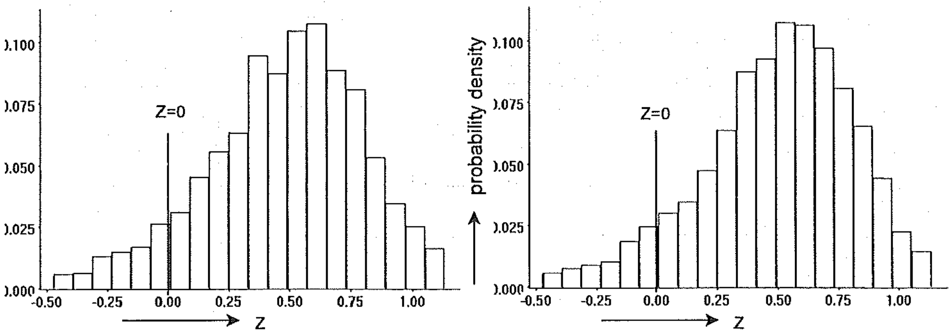


Figure 0-4 Realisations of Monte Carlo simulation

$S = 2$ was used in the deterministic approach. This can be seen as a Serviceability Limit State (SLS), beyond which some repair can be necessary. So, there is a 10% chance per year that some damage to the revetment will occur. This result is not really astonishing. But how safe is our building really? To judge the safety, it is necessary to determine the probability of damage to the building. This requires determination of the Ultimate Limit State (ULS).

Computations with S = 10

The ULS will be approximated here simply with $S = 10$. The reason for this is as follows: $S = 10$ indicates the damage when the top layer (which is about $2 \cdot d_{n50}$ thick) has been removed completely. Once the top layer removed, the filter layers, which have much less resistance against wave loads, will be removed as well. Underneath the revetment there is only sand. Equation 8.2 gives an idea of how far the coast will erode. With a wave of 1.6 m and fine sand, this will be about 40 m. This means collapse of the building near the shore. Implicitly it is assumed that the storm lasts for several hours, which is reasonable. So, although roughly, S = 10 can indeed be seen as total failure.

The Monte Carlo simulation for S=10, with all other parameters equal to those in the previous case, gives $P_F = 0.011$. So there is a 1% chance per year of total collapse of the building. Is this a problem? The lifetime of a building is normally many decades, say 50 years. The probability of collapse is then given by:

$$P_F \ in\ 50 years = 1-(1-P_F / year)^{50} = 0.42 \qquad (0.5)$$

So, there is almost a 50% chance that this building will be destroyed during its lifetime! It should also be noted that even the best maintenance policy can not prevent this, since a storm that causes a damage level of 10 will also do so when the armour layer is still completely intact. The capitalized risk is:

$$R = \sum_{n=1}^{50} P_F D \frac{1}{(1+r)^{50}} = P_F D \frac{1-\left(\frac{1}{1+r}\right)^{50}}{r} \qquad (0.6)$$

in which D is the total damage and r is the interest rate. D, (including the economic activities related to the building) when $S = 10$ is set to $10 \cdot 10^6$ € and r is assumed to be 5%. The capitalized risk is then $0.011 \cdot 10 \cdot 10^6 \cdot 18.25 = 2 \cdot 10^6$ €.

The final answer to this dilemma has to come from econometric considerations. These considerations are presented very simply. Revetments with various strength will be compared with a focus on costs and risk. For the involved risk, Table 0-2 gives the results of Monte Carlo computations and equation (0.6):

Table 0-2

Armour layer (kg)	d_{n50} (m)	P_F per year (-)	P_F per 50 years (-)	Risk (10^6 €)
60 - 300	0.4	0.189	0.999	34.5
300 - 1000	0.6	0.011	0.42	2.0
1000 – 3000	0.85	0.001	0.049	0.18
3000 – 6000	1.1	0.00017	0.0085	0.03

This has to be compared with the costs of the different revetments. Again, some simple assumptions will be made. The length of bank necessary to protect the building is assumed to be 200 m. The evolved length of the protection along the slope of the bank directly follows from the geometry of Figure 0-1: $\sqrt{(6^2 + 24^2)} \approx 25$ m. The layer is taken $2 \cdot d_{n50}$ thick. Only the costs of the top layer are different and one extra filter layer for the largest stone classes is needed. The costs of the other activities to construct the revetment (excavating, creating slope, filter layers, toe protection) are assumed to be $1 \cdot 10^6$ € for all revetments. Table 0-3 gives the costs:

Table 0-3

Armour layer (kg)	Cost per m^3 (€)	Volume (m^3)	Costs extra filter layer (10^6 €)	Costs (incl. extra filter) (10^6 €)	Total costs revetment (10^6 €)
60 - 300	20	4000	0	0.08	1.08
300 - 1000	24	6000	0	0.14	1.14
1000 – 3000	30	9000	0.02	0.27	1.27
3000 – 6000	36	11500	0.02	0.42	1.42

The difference in costs is small: the initial costs to construct the revetment ($1 \cdot 10^6$ €) are dominant. Comparison of costs and risks now shows the following picture, see Figure 0-5. It is obvious that stone class 1000-3000 kg instead of 300-1000 kg is a good choice, since the risk decreases with a factor 10 for just a small amount. Another solution could be a thicker top layer, permitting a larger S before the structure is endangered.

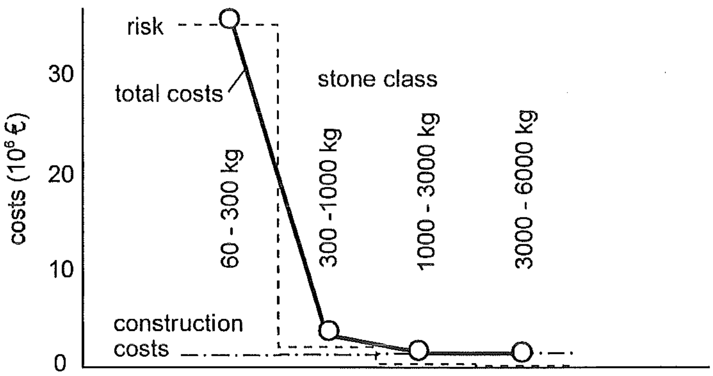


Figure 0-5 Risk and costs for various top layers

This choice becomes completely different when there is no expensive building directly on the shore. For example, if there is a protection, only to prevent further meandering of an estuary channel, the cost-risk ratio is completely different and the deterministic approach of table 0-1 is perfectly adequate (see also the evaluation in section 0).

Level II

The probabilistic level II approach is a collective term for approximate solutions of the failure probability by means of linearization around a well-chosen point, the so-called **design point**. The limit-state function, Z, is described with a normal distribution just like all parameters that make up Z. This means that a deviating distribution of a parameter will be replaced by a normal distribution, which has the same value and slope in the design point as the original probability function. The failure probability, finally, is determined from the properties of the normally distributed Z-function, μ_Z and σ_Z via, see also Figure 0-6:

$$\beta = \frac{\mu_Z}{\sigma_Z} \tag{0.7}$$

The failure probability P_F and β are directly related in the normal distribution and can be found in standard tables. So, if β can be derived from the known parameter distributions, P_F can also be known.

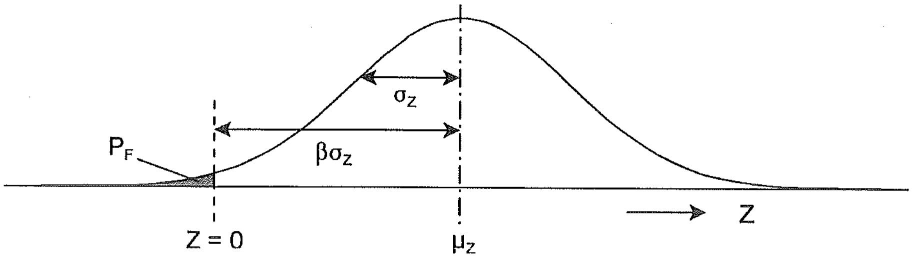


Figure 0-6 Failure probability in level II approach

The **design point** is the point on the line $Z = 0$ where the probability density of the combination of load and strength has its maximum. When a structure fails, load and strength will probably have values near the design point values. The distance between the average value of a parameter and its design value is an indication of its importance with regard to σ_Z and, hence, to the failure probability of the structure. This importance is indicated with α_i for each parameter.

The VaP results for the original Z-function, equation 10.5, are:

Table 0-4

$\beta = 2.33$, $P_F = 0.0099$			
Parameter	α-value	α-value^2	Design value
H_s	0.90	0.81	2.054 m
d_{n50}	-0.24	0.06	0.571 m
P	-0.25	0.06	0.068 -
Δ	-0.18	0.03	1.557 -
$\tan\alpha$	0.07	0.01	0.252 -
s	-0.15	0.02	0.046 -
		Σ=1	

The influence of the wave height variation on the failure probability is clearly dominant. This is often the case; the load variations are more important than the strength variations.

Comparing the failure probability from the level II-analysis with the results of the Monte Carlo approach of section 0 gives an idea of the reliability of the level II method as a whole. $P_F \approx 0.011$ is found with the Monte Carlo approach compared with $P_F \approx 0.01$ from the level II approach. A difference of 10% is acceptable, since computations like this only serve to obtain an indication of the failure probability.

The advantage of a level II method is the resulting α_i-values, which indicate the relative importance of a parameter in the total failure probability. The combination of a large σ_i and a large power in the Z-function lead to a high α-value for a parameter. So, with good engineering judgement, a large α-value does not come as a surprise and an engineer intuitively chooses a conservative value for such a parameter.

Level I

A level I approach adds nothing new to what has been said above. It is actually an application of the results of a higher-level method, especially level II methods. The approach requires a design value and a safety coefficient to be established for every parameter. The safety coefficients are derived from a level II computation, using the α-values and β-value. The β-value stands for the required safety and the α-values stand for the relative importance of each parameter. The partial safety coefficient for each parameter is then given by:

$$\gamma_i = \frac{\mu_i - \alpha_i \beta \sigma_i}{\mu_i} \tag{0.8}$$

α-values are negative for loads and positive for strength, leading to $\gamma > 1$ for loads and < 1 for strengths when used as a multiplier in both cases. Other definitions are possible and can be found in literature. The safety factors in equation (0.8) are defined with regard to the average values. When using other characteristic values, the safety factor changes correspondingly. This approach can be seen as the application of engineering judgement as mentioned in the previous section.

Evaluation

On the first page of this appendix we started with a deterministic approach and compared the result with various levels of probabilistic methods. A deterministic method results in a certain strength (in the example the necessary stone class), given a certain load (in the example the wave height). A probabilistic approach results in a failure probability, given the distributions of load and strength. So, a probabilistic method never leads directly to dimensions for a structure. Given an acceptable failure probability, the dimensions have to be determined iteratively.

The risk analysis in section 0 showed that the deterministic approach led to an unacceptable high risk in the case of a building near the shore line. Would it have been possible to avoid this high risk without sophisticated probability models?

The deterministic approach was based on a wave height with a return period of 10 years and a low damage level, $S = 2$. From the risk analysis it became clear that the real issue is an acceptable low probability of failure of the building, equivalent to $S = 10$. Comparing the cost of a revetment, which is in the order of magnitude of 0.5 mln €, with the risk of collapse of a building of 5 mln €, the failure probability should be less than 10% during the lifetime of the building, which is about 50 years, say 5%. A convenient formula to deal with this issue is the Poisson-equation, which is an approximation of equation (0.5):

$$P = 1 - \exp(-f\,T) \tag{0.9}$$

in which: P = probability of occurrence of an event one or more times in period T

T = considered period in years
f = average frequency of the event per year

For $P = 0.05$ and $T = 50$ years we find $f = 1/1000$. In other words: a wave height with a return period of 1000 years should be used. From Figure 0-2 we find: $H_s \approx 2.8$ m. With S = 10 in equation 10.4, this gives: $d_{n50} \approx 0.7$ m. This would lead to a stone class 1000-3000 kg instead of 300-1000 kg. The same procedure could be applied to the maintenance. In that case a 10% chance of failure (SLS) or, in other words, the need for maintenance, is reasonable. This would lead to the same calculation as carried out in the deterministic approach. (once in 10 years wave height and $S = 2$). In this case, however, the ULS is dominant. In the case where there is not an expensive building on the shore, maintenance is the only criterion.

The Poisson equation (0.9) combined with common sense as outlined above can be seen as a semi-probabilistic approach which can serve as a very useful tool in a preliminary design.

Of course, to establish the dimensions of a complex and important structure, like a storm surge barrier to protect a large and densely populated area, a probabilistic approach is an important design tool. Since probabilistic computations can only be done with some given structure, deterministic calculations are always carried out first and the final dimensions result from iteration between both approaches. A level III method is the best way to determine the failure probability. Level II methods are useful to get an insight into the relative importance of the various parameters involved.

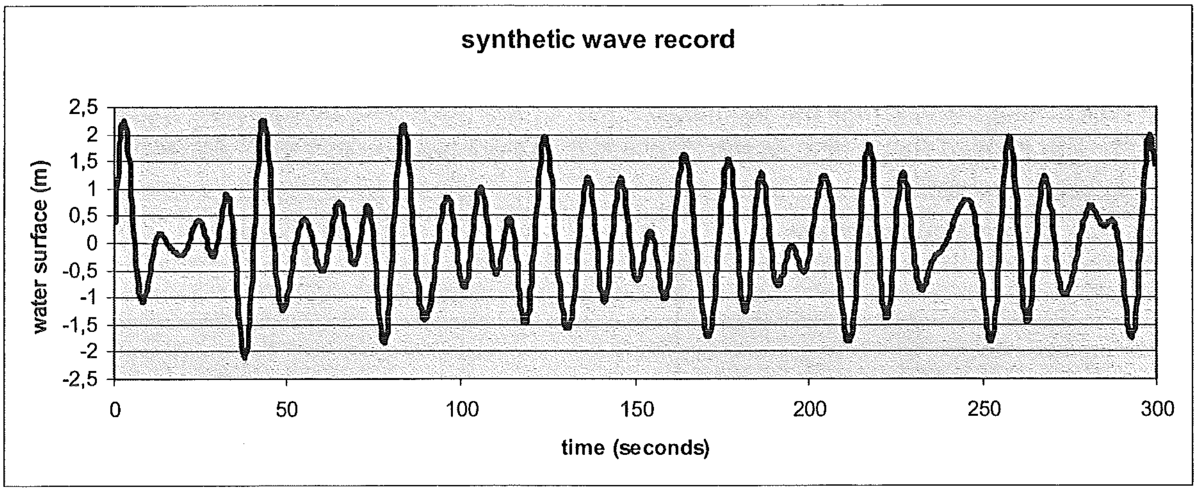
Equation (0.9) can also be used to show other elementary statistical aspects:

- A common mistake is to design a structure with an envisaged lifetime of 5 years (e.g. a temporary situation during construction) with a load condition with a frequency of 1 per 5 years. Equation (0.9) shows that the probability that this load is reached or exceeded is 1-exp(-1) ≈ 0.63. So, there is a 63 % chance that conditions will be worse than assumed, which is usually not acceptable.

In the case of a sea defence structure, to protect a low-lying area, it is impossible to define the lifetime of the structure since it is supposed to protect "forever". Assuming an acceptable probability of 1% in a human life (e.g. 75 years), instead of using the lifetime of the structure, equation (0.9) gives the frequency for the design conditions: $f \approx 1/7500$ year which is quite a normal number for dangerous flooding hazards.

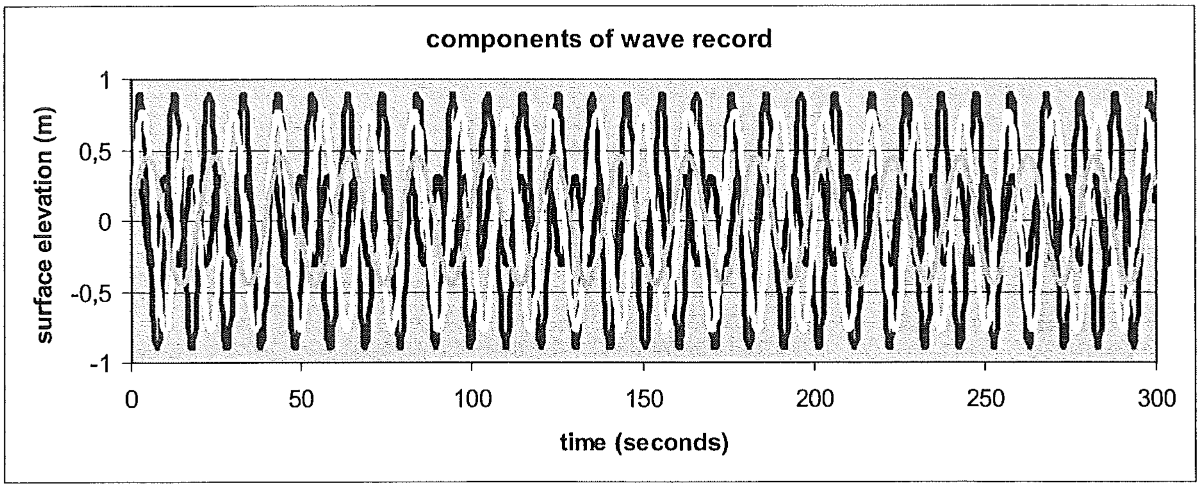
Bijlage 2: Spectral Wave periods

In order to describe the interaction between waves and structures, especially in shallow water, it seems that there is a stronger influence of the low frequency part of the spectrum. In order to take this into account, the spectral period $T_{m-1,0}$ is used. In order to explain the meaning of this, a very simple wave record is used.



This record contains 28 waves in 300 seconds, so the average wave period is in the order of 10 seconds. The significant wave height is (assuming a Rayleigh distributed wave it is the wave exceeded in 13% of the cases) is the fourth biggest wave (0.13 * 28 = 4). This wave is approx. 3.8 m high.

The sample record seems natural, but is a synthetic one. In fact it is composed of four components.



The record is built up from the following components:
H_1 = 0.63 m T_1= 4 sec
H_2 = 1.80 m T_2 = 5 sec
H_3 = 1.55 m T_3 = 6.67 sec
H_4 = 0.90 m T_4 = 10 sec

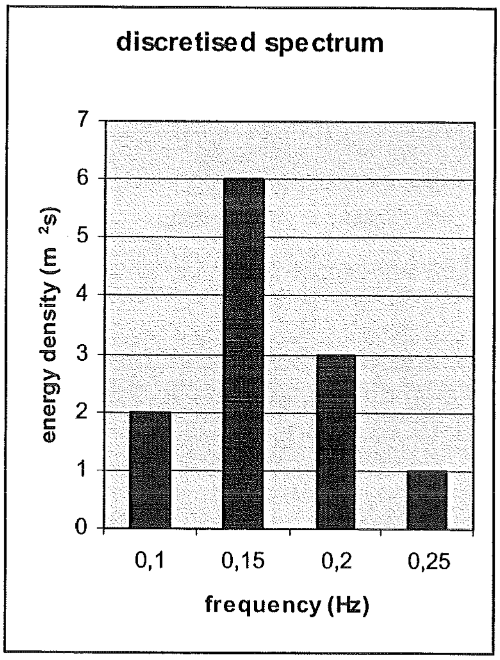
A spectrum describes the energy as a function of the frequency. The energy of the wave is given by:

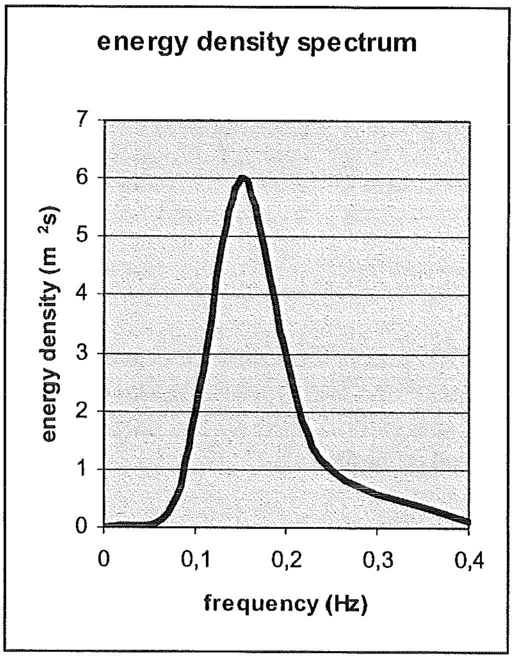
$$E = \frac{1}{2}a^2$$

in which *a* is the amplitude of the wave (half of the wave height). The energy density *S* can now be defined via:

$$\frac{1}{2}a^2 = S \bullet \Delta f$$

in which *Δf* is the width of the frequency band. In our example we have four wave heights, each with its own frequency, so the discretised and continuous spectrum are defined as follows:





dist * SΔf	
0.10*0.10	0.010
0.15*0.30	0.045
0.20*0.15	0.030
0.25*0.05	0.013
	0.098

This can also be written as:

$$H = \sqrt{8S \bullet \Delta f} \qquad S = \frac{H^2}{8\Delta f} = \frac{1.55^2}{8 \bullet 0.05} = 6\ [m^2 s]$$

A spectral moment is the product of the area of a part of the spectrum, multiplied with the distance of that part of the spectrum to that value. The first order moment m_1 can be calculated for the discrete spectrum simply as indicated in the adjacent table. In formula this is:

$$m_1 = \int_0^\infty f \cdot S(f)\,df$$

This moment is called the first order moment, because the distance *f* is the distance to the first power. In the same way one can also define the zero order moment. In fact, the zero order moment is equal to the surface of the spectrum.

$$m_0 = \int_0^\infty f^0 \cdot S(f)\,df = \int_0^\infty S(f)\,df$$

In our example this value can be calculated as is indicated in the table on the right. It is obvious that $4\sqrt{m_0} = 3.1$ m is the wave height according to this spectrum.

0.05*2	0.10
0.05*6	0.30
0.05*3	0.15
0.05*1	0.05
	0.60

Following the same reasoning one can also define a second order spectrum:

$$m_2 = \int_0^\infty f^2 \cdot S(f)\,df$$

Filling in the values for this case leads to the adjacent table, and the value of the second order spectrum is $1.69 * 10^{-3}$.

$dist^2 * S\Delta f$	
$0.10^2 * 0.10$	$1.00\ 10^{-3}$
$0.15^2 * 0.30$	$6.75\ 10^{-3}$
$0.20^2 * 0.15$	$6.00\ 10^{-3}$
$0.25^2 * 0.05$	$3.12\ 10^{-3}$
	$1.69\ 10^{-3}$

From the second order one can calculate the spectral wave period as follows:

$$T = \sqrt{\frac{m_0}{m_2}} = 10\sqrt{\frac{0.60}{1.69}} = 5.69\,\text{sec}$$

Following the same reasoning one can also define a first order negative spectrum:

$$m_{-1} = \int_0^\infty f^{-1} \cdot S(f)\,df$$

Filling in the values for this case leads to the adjacent table, and the value of the first order negative spectrum is 3.95

1/dist * SΔf	
1/0.10*0.10	1.0
1/0.15*0.30	2.0
1/0.20*0.15	0.75
1/0.25*0.05	0.20
	3.95

In the same way as above, one can define the $T_{m-1,0}$ period with the following equation:

$$T_{m-1,0} = \frac{m_{-1}}{m_0} = \frac{3.95}{0.60} = 6.58\ \text{sec}$$

This period gives somewhat more weight to the longer periods of the spectrum.

As an overview, for our example the following values have been found:

H_{m0} = 3.1 m (1.55+1.10+0.90+0.63=4.18)

T_{m0} = 5.69 sec

$T_{m-1,0}$ = 6.58 sec

T_{peak} = 6.67 sec

$$\frac{T_{m-1,0}}{T_{m0}} = \frac{6.58}{5.69} = 1.16$$

$T_m = T_{1/3} = T_{m0}$

These values are only valid for this synthetic spectrum. For real spectra the following factors are given:

Goda:	$T_p=1.1\ T_{1/3}$
PM:	$T_p=1.15\ T_{1/3}$
Jonswap:	$T_p=1.07\ T_{1/3}$
TAW (vdMeer):	$T_p=1.1T_{m-1,0}$
Old Test (vdMeer):	$T_p=1.04\ T_{m-1,0}$
Also:	$T_{m-1,0}=1.064T_{1/3}$

Syllabus

Prepared for PAO Course on Innovative Design of Coastal Defense

Ecological considerations for sound design

April 2005

Contents

1. Designing hard defence structures

1.1 Introduction

The design of a defence structure, and the materials used can be selected based on ecological requirements. The design concepts for sea defences with regard to shape and materials used will affect the local hydrodynamic conditions (e.g. waves, currents, local turbulence, etc.). This will in turn affect the suitability for benthic marine species and communities on or in the substrate and the attraction of pelagic species to the new ecosystem. The incorporation of different designs for the sea defences provides a chance to create a diverse new ecosystem.

1.1.1 Main driving factors

Four major environmental gradients are responsible for a variety of shore types, ranging from hard rocky shores, via sandy beaches to (estuarine) mudflats. These are:
1. The vertical gradient
2. The horizontal gradient of exposure to wave action
3. The particle size gradient
4. The salinity gradient

The vertical gradient
The vertical gradient can result in emersion of organisms and that will result in fluctuations in temperature, desiccation and food supply. On lower levels depth may result in a decrease in light availability for plants. Tidal and wave movements will create variations in the physical parameters and vertical zonation is the result. Species are ordered along this gradient according to their abilities to cope with physical factors and variation in their responses to biological processes (e.g. competition and predation).

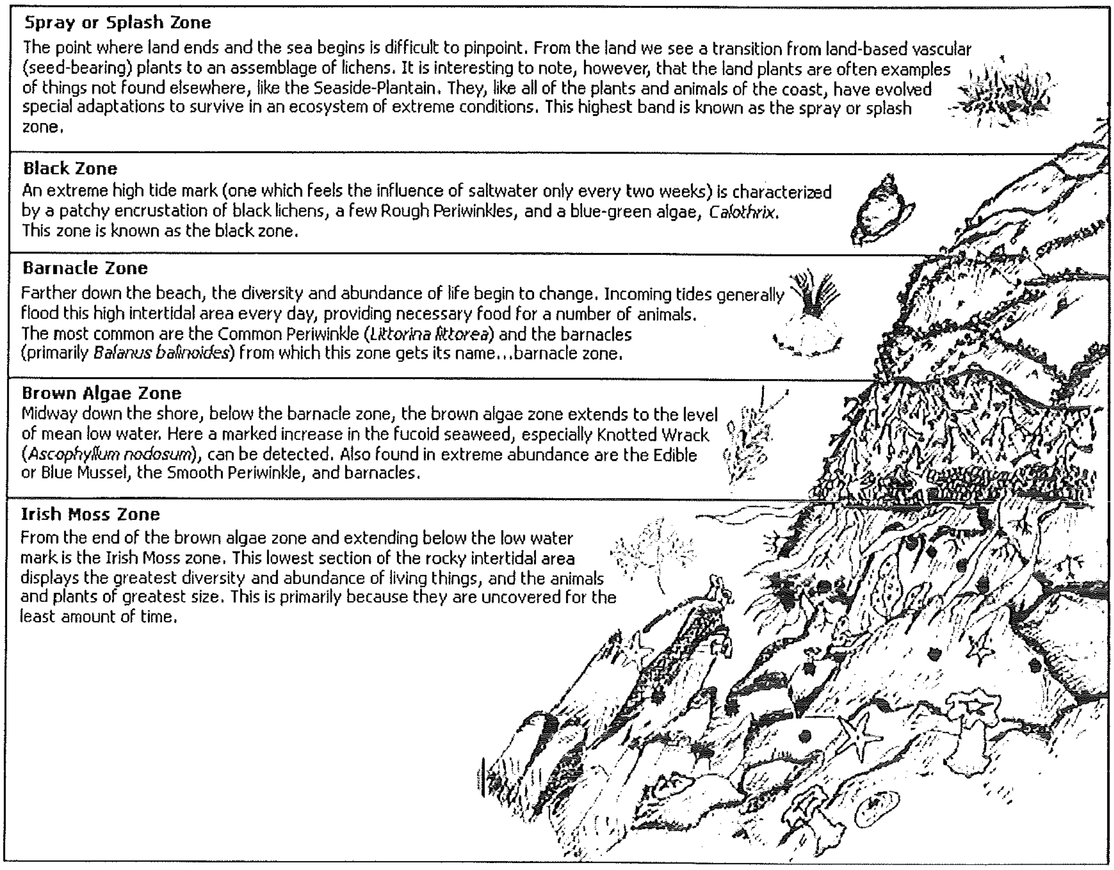
The horizontal gradient
The horizontal gradient that is mentioned here is the gradient in wave exposure going from the sheltered bays to the exposed rocks on the outside. This gradient is determined by the wind speed and fetch. Wave action has a profound effect on the biological characteristics of a shore. Wave action can pose stress on animals on the one hand or transport food on the other hand. Areas with net siltation or high turbulence will show very different species, adapted to the conditions.

The particle size gradient
Particle sizes of shores range from a few micrometers on mudflats, several hundred micrometers on sandy beaches, a few centimetres in gravel and shingle habitats, tens of centimetres on boulder shores and metres for massive boulders and breakwaters. The average size of a natural shore reflects in part its exposure to water movement due to wave action and currents, and in part the geological history of the area. The different particle sizes of shores result in different habitats for different species.

The salinity gradient
Fluctuations in salinity may occur along the vertical gradient, for example where pools of sea water are trapped in rock depressions and hollows. Salinity stratification may also occur. The sensitivity to salinity ranges differ for different species.

Zonation on hard substrate
The influence of the gradients in the above mentioned variables will result in a typical zonation of marine flora and fauna. The shape of the sea defences, the type of material and other factors will

affect this zonation and may encourage certain species and discourage others.



1.1.2 Material properties

Several properties of the material used for the defence system affect the growth of organisms:
The **smoothness** affects the possibilities for settling, particularly in systems with high current velocities or wave attack.
The **placement** affects the open space in between the stones. Rectangular stones, placed closed together have less holes and cavities.
The **water retention capacity** is particularly important in the tidal zone. Limestone retains water better than basalt blocks, and is found to have a higher species diversity and density.
The colour determines the temperature. Dark stones show more extreme temperature differences.
The **size** is determinant for the species diversity. Larger stones offer a wider gradient of abiotic conditions and a larger species diversity.
The **chemical composition** might effect the species diversity, especially when furnace slags are applied for coastal protection.

Conclusions:

- Asphalt is a bad substrate for organisms, both in the intertidal and subtidal zone.
- A large water retention capacity is favourable in the intertidal zone.
- Using larger elements (diameter 30 cm or more) is recommendable.
- Placing the elements loosely is better for the diversity

In the 1980s considerable research has been undertaken by Rob Leewis, see the reference list.

2. Ecological impacts of coastal defence structures

Background knowledge and tools for prediction of ecological impacts of coastal structures

Prepared in EU-DELOS project
April 2004

www.delos.unibo.it

2.1 Definitions of main factors influencing the distribution and abundance of species and assemblages (biotopes) on natural- and rocky bottoms

2.1.1 Broad-scale – geographic variation

The species pool in a particular locality, is determined by its biogeographic context. This is the result of past events on tectonic / evolutionary time scales (100 million years – 1 million years P., e.g. Mediterranean compared to Atlantic) and more recent palaeo-ecological / geomorphological history (. last 30 years e.g. English Channel, North Sea and Irish Sea coastlines). Global transfer of species (e.g. Lessepsian migrations via Suez canal) has gathered its importnance over the last 200 years. The evolution of the species pool is a dynamic and ongoing process. Biodiversity patterns on a broad-scale are a function of adaptation, extinctions and speciation. The species pool may also change following introduction of alien species, often through human activities (Stæhr et al., 2000). Broad scale biodiversity patterns are influenced by major physical factors such as climate, currents, upwelling, tidal elevation, wave climate, salinity, coastal topography and seabed composition, which can all vary with geographical location (e.g. greater waves on Atlantic coast of Ireland versus the more enclosed Irish Sea, salinity in Baltic versus North Sea, tides in Atlantic versus Mediterranean and Baltic).

2.1.2 Mesoscale – within coastline

The species assemblage found at a specific location is affected by the exchange with neighbouring populations through dispersal, mainly through suspended propagules (e.g. larvae and spores). The spatial distribution of source populations is largely governed by coastal geomorphology that determines the diversity of substrata and hence habitat types in a particular region. Morphodynamics of sediments further affect the coastal-scale distribution of sedimentary habitats. The presence of source populations, however, is not sufficient to ensure exchange between habitats.The dispersal between habitats depends on hydrodynamic transport, although interactions with behavioural responses (or gravitational sinking) may modify dispersal pathways. Hydrodynamic transport includes tidal, wind driven and baroclinic advection (currents) together with turbulent diffusion. Other coastal-scale factors that may influence species assemblages are point sources of nutrients, contaminants, suspended sediment and freshwater (e.g. from riverine discharge). Differences in geomorphology and bathymetry will also cause coastal-scale differences in wave climate that will in turn influence local species distribution.

Coastal geology, geomorphology and topography

The topography and geomorphology of the coastline are crucial to the distribution of species. The description of the large-scale distribution of species and assemblages therefore must take account of the characteristics of sediment, natural rock and artificial substrata. the underlying geology of an area can have significant effects on the distribution and abundance of species (Crisp, 1974; Holmes et al., 1997). For example, rock types of differing physical and chemical properties seem to affect the settlement of various barnacle species. Other features of the substratum are also important, such as the surface composition and orientation (Glasby, 2000, Glasby and Connell, 2001). For soft bottom communities this factor is coupled to hydrodynamics, d).

Localised nutrient supply due to small-scale upwelling, riverine run-off, seawage disposal
Local small-scale upwelling carries nutrients from deeper water to shallow water and changes the local nutrient concentrations. Fresh water run-off can carry nutrients from farmlands and forests via the catchment. Waste discharge may locally increase nutrient availability. Differences in the local concentration of available nutrients will have large impacts on the local species composition.

Hydrodynamic – sedimentary regimes affecting erosion / deposition, disturbance regime, turbidity and long-shore transport.
The coastline topography and geomorphology as well as the local bathymetry influence the hydrodynamics. The hydrodynamics is also responsible for the sedimentary regime affecting erosion and deposition of sediments, turbidity, disturbance regimes for the biota and long-shore transport. Soft-bottom assemblages are highly affected by changes in the sedimentary regimes (deposition, erosion) and modification of sediment characteristics such as organic matter and granulometry. Turbidity of waters also affects a variety of organisms, including seagrasses, invertebrates and algae by reducing light penetration through the water column. The factors and processes described above will in turn affect the connectivity of habitats and larval supply – sources and sinks of propagules, recruitment regimes, metapopulation dynamics. Connectivity of habitats and larval supply can be very important for the large-scale distribution of species and assemblages. In fragmented habitats the connectivity is low and the species composition may be affected by chance events. The connectivity and larval supply thus determines colonisation probabilities for species and populations. Low connectivity means low colonisation probability and high connectivity means high colonisation probability. The dynamics caused by extinctions and colonisations is often termed metapopulation dynamics. Post-recruitment events may also control the population survivorship rates and the persistence of recruits is often a more relevant factor in controlling population dynamics than the recruitment itself. (Jackson, 1986)Species composition in fragmented habitats is strongly dependent on residual currents. On the other hand residual currents will be less important for the dispersal of organisms existing in a commonly occurring habitat or where the habitat is narrow but well connected. Assuming a fragmented habitat the range expansion of species may depend largely on the extreme values of the actual hydrodynamics, and not the mean residual current.

2.1.3 Local scale - major abiotic factors and processes

Several abiotic factors affect the distribution of species on a local scale (Lewis, 1964, Stephenson and Stephenson, 1972, Raffaelli and Hawkins, 1996). These include vertical and horizontal patterns of distribution caused by tidal elevation, wave exposure, light penetration and, in sediments, physical and chemical gradients. In addition, local disturbance caused by extreme events such as wave-induced impact, depletion of oxygen and sediment burial can create a mosaic pattern of species occurrence. Some key gradients are summarised below:

Tidal elevation / depth.

On macrotidal shores, the time of emersion / submersion and consequently desiccation stresses experienced by intertidal organisms, as well as the time to take up nutrients (algae) and food (invertebrates), markedly depends on the tidal level (Lewis, 1964; Raffaelli and Hawkins, 1996). The distribution of species is affected by tidal level, as physiological tolerance to emersion and desiccation stresses varies between and within species but in general a higher number of species tend to better tolerate lower shore environmental conditions (Lewis, 1964; Newell, 1979; Raffaelli and Hawkins, 1996; Spicer and Gaston, 2000). This pattern is particularly evident on macrotidal shores, where epibiotic assemblages differ markedly between different tidal levels.On microtidal shores, the structure of benthic assemblages changes considerably with increasing depth, from an algal monopolized community to a community dominated by sessile invertebrates. This is mainly due to a decrease in light penetration, which can be further reduced by turbidity (Gaçia et al., 1996; Irving and Connell, 2002).

Wave exposure

Wave action plays a major role in the composition of rocky littoral and sub-littoral communities shores (Lewis, 1964; Hiscock, 1983; Raffaelli and Hawkins, 1996). On exposed shores benthic organisms experience greater wave-induced forces and consequently face a higher risk of breakage or dislodgement from the rock and consequently their persistence. Wave action, however, can increase wetting of upper shore species, nutrient supply for algae and suspended food for filter feeders. Foraging times can be both positively and negatively impacted. Conversely on more sheltered shores, reduced water movement is generally associated with greater sediment deposition and siltation on the rock substratum, which can be cause of disturbance. Species respond differently to this stress gradient (Denny et al., 1988; Denny, 1995); some organisms thrive better and are naturally more abundant in wave swept conditions (e.g. mussels and barnacles), whilst others are adapted to more sheltered conditions (e.g. the macroalga Ascophyllum nodosum and the gastropod Osilinus lineatus). .

Salinity

Salinity gradients occur in estuaries and coastal areas near riverine inputs. This factor affects particularly the species pool, as only few species can tolerate low or variable salinities. Salinity can affect the vertical distribution of species: in the sopralittoral zone salinity can increase considerably in crevices and rock pools (Raffaelli and Hawkins, 1996).

Physical disturbance

In rocky intertidal and subtidal assemblages, physical disturbances associated with partial or total loss of biomass have been recognised as primary mechanisms that generate mosaics of patches at different stages of recovery, and control abundance and diversity of species (Dayton 1971, Menge 1976, Sousa 1979, 2001; Paine and Levin 1981, Airoldi 2000 a, 2003). Waves, excessive heat, scour from sediment and other debris are examples of natural disturbances that cause mortality of organisms and open discrete patches of open space (Dayton 1971, Hawkins and Hartnoll 1983, Airoldi and Virgilio 1998).

2.1.4 Local scale - Biological interactions and behaviour

On rocky shores the following biological interactions and processes are extremely important in influencing species distribution at small spatial scales:

1. Grazing / predation
2. Competition for space
3. Biologically mediated disturbance (algal sweeping, bioturbation)
4. Facilitation (positive interactions, sheltering etc.)
5. Biodeposition and sediment trapping
6. Larval and adult behaviour

Local biodiversity reflects the direct and indirect interactions among and within species. Trophic interactions are particularly strong on hard substrata, for example limpet grazing on algae on rocky shores (Hawkins, 1981; Hawkins et al., 1992). Competition for space or resources often reduces the diversity of species assemblages but diversity can often be higher at intermediate levels of physical and biological disturbance (Caswell, 1978). Examples are biologically mediated disturbances like algal canopy sweeping on rocky shores and bioturbation in sediments (Rhoads, 1974). Certain species can also improve conditions for other species and so increase the local biodiversity. Such "facilitation" effect includes several mechanisms, e.g. sheltering from canopy-forming macro-algae or mussel beds promoting recruitment of polychaetes and small crustaceans. Some species build 3-dimensional structures that alter the physical conditions leading to changes in the species assemblage. Examples include reef-building polychates consolidating sand beds, encrusting algae creating complex secondary substrata, and meadow-forming seagrass attenuating wave energy. Organisms changing the hydrodynamic regime by wave attenuation or flow reduction will often promote sediment trapping offering new habitats for sediment-living organisms or exclude species sensitive to high sediment load. Finally, spatial heterogeneity of abiotic and biotic factors may interact with behaviour during all life stages. Gregarious responses during the settlement phase in barnacles are one example that leads to aggregated distribution patterns.

2.1.5 Micro scale - Complexity

On even smaller scales (<10 cm), factors such as heterogeneity in surface topography (roughness) affect the availability of refuge from hydrodynamics and grazing (Fretter and Manly, 1977; Underwood and Chapman, 1998). In sediments small-scale gradients in grain size and compaction (both horizontally and vertically in the sediment column) may lead to changes in porous flow and chemical composition with strong effects on infauna assemblages.

2.1.6 Human activities

Human activities alter the marine environment at various scales from global (e.g., climate change) to the local (point source pollution). Major factors likely to interact with natural processes in the coastal zone are outlined below. These factors need to be considered when predicting the impacts of LCS construction:

Global changes

Anthropogenic release of greenhouse gases is now widely accepted to be influencing the climate of the planet. Various predictive scenarios have been made. In short, air and sea temperatures will increase, as will sea level (IPCC, 2001a,b). The Atlantic Ocean and adjacent seas will become stormier in part due to greater frequency of NAO positive winter values. Thus in addition to rise in average temperature and wave height the incidence of extreme events will be more likely.. Southern species will migrate towards the poles. Increased likelihood of extreme events will lead to an increasing number of LCS being built along the coast. This in turn will have marked effects in the distribution of species. There is evidence from the Delos project and climate change programmes (e.g., the MarClim project coordinated by the MBA – www.marclim.mba.ac.uk) of species extending their ranges using artificial structures as stepping stones between areas of natural hard substrates or in their absence extending their distribution (Herbert et al., 2003). A good example is the southern snail, Gibbula umbilicalis, which has been found at Elmer 60 km east of its previous limit. Southern fish species such as anchovies (Engraulis sp.) and sardines (pilchards, Sardina pilchardus) have also been found around the breakwater at Elmer.

Spread of exotic species

The arrival of new species from different biogeographic provinces has increased in recent years. The main vectors are ships and aquaculture. Thus new highly competitive species in Europe such as seaweeds Undaria and Sargassum (from Japan) can arrive in an area and markedly change the ecology of an LCS (Floc'h et al., 1996; Staehr et al., 2000). Coupled with global environmental change, escapes of non-native species from aquaculture become more likely (e.g. Crassostrea, an oyster of far eastern origin).

c) Disturbance due to maintenance and food harvestingof LCS

Frequent maintenance of LCS, such as replacement or relocation of boulders within a structure, can cause severe disturbance to epibiotic assemblages... Maintenance of LCS reduces effectively species diversity by keeping the assemblages at an early successional stage, thus dominated by opportunistic species such as ephemeral algae (Ulva spp., Porphyra sp.). As a consequence, frequent maintenance, while increasing the availability of uncolonised space (bare rock), will have profound effects on the richness of species and on the biomass supported by LCS.

Broad-scale eutrophication

Eutrophication (anthropogenic nutrient enrichment) is a common phenomenon in enclosed bays and estuaries due to a combination of agricultural run-off and human and agricultural wastes (Correggiari et al., 1992). It can also scale up to larger areas such as the northern Adriatic, parts of the Baltic and the southern North Sea and possibly the irish Sea, resulting in eutrophic seas (Allen et al., 1998). On a large scale atmospheric input of nitrogen can also be important. Eutrophication causes several effects in the marine ecosystem. Higher concentration of nutrients will lead to an increase in the abundance of phytoplankton and consequently greater food resources for filter-feeders such as mussels. However, the likelihood of harmful algal blooms (e.g. red tides) will also increase causing anoxia and thus killing macroalgae and marine invertebrates (Southgate et al., 1984). Macroalgal growth for example ephemeral green algae will also be faster in eutrophic conditions, in many instances being able to outpace grazing activities.On LCS, eutrophic waters coupled with high levels of disturbance will create optimanl conditions for proliferation of slippery green algae.Sediments, in turn, will tend to become muddy and compact, leading to substantial changes in the chemical gradients in the sediment (e.g., anoxia) which will, in turn, modify the infaunal composition (i.e., reduction of diversity,and proliferation of opportunistic species). Impacts of eutrophication will be worse on the landward side of LCS, where water movement is significantly reduced, particularly if the structures if connected to the shore by groynes.

Localised acute and chronic pollution

Acute pollution incidents (e.g., oil spills) and chronic point source pollution (e.g., heavy metals, persistent organics including leachates from antifouling paints) will affect the species composition and successional processes of benthic assemblages. On rocky shores acute incidents such as oil spills (e.g. Torrey Canyon)generally lead to mass-mortality of organisms, in particular more sensitive species such as limpets (Southward and Southward, 1978). Following deaths of these grazers, early successional, opportunistic species such as ephemeral algae will flourish. Other macroalgae such as fucoids will follow but marine invertebrates such as barnacles and limpets will take longer to recolonise. Epibiotic assemblages on LCSs will be similarly affected by such polution incidents. Chronic pollution can severely affect the epibiotic species. For example, predatory whelks, which are commonly found on LCSs, have been shown to be particularly sensitive to TBT pollution from antifouling paints which can induce "imposex" (females become masculinised) leading to sterility (Gibbs and bryan, 1986; Bryan et al., 1986; Spence et al., 1990). This problem is still evident near marinas and commercial ports, despite the ban of TBT on small boats throughout Europe. Under certain conditions, however, the effects on benthic communities caused by both acute and chronic pollution generally tend to reverse once the pollution source is eliminated or reduced. For example, after the clean-up of the river Mersey (near Liverpool, UK) limpets (Patella vulgata) and dogwhelks (Nucella lapillus) have been found recolonising LCSs on Merseyside in recent years.

Overexploitation of natural living resources

Overfishing and the proliferation of coastal infrastructures such as marinas and sea defences have significantly reduced the fish stocks, particularly for species that tend to settle in shallow coastal waters. LCSs, however, seem to create suitable habitats (particularly the sheltered landward side) for settlement of juveniles of commercial fish such as sea bass, sole and plaice, and crustaceans, such as lobster and crabs. LCSs can therefore could represent new nursery grounds for fish, contributing to enhance the local fishery.

Effects of recreational use of LCS

Shellfish harvesting and recreational use of LCSs can lead to disturbance through collection of a range of organisms for food, bait, or aquaria, and trampling, particularly during summer (Duran and Castilla, 1989, Kingsford et al.1991, Dye, 1992, Keough and Quinn, 1998, Fraschetti et al., 2001, Moreno, 2001). These activities are likely to affect the persistence, growth and abundance of more vulnerable species, thus leading to changes in diversity and dynamics of the whole assemblage, as largely documented on rocky shores (reviewed in Thompson et al., 2002).,). For example, on LCSs along the North Adriatic Sea mussels are subject to intensive harvesting, creating patches of bare space and increasing the abundance of pioneer species such as ephemeral algae. Intensive fishing removes top level predators and may alter the food webs leading to an increase in lower trophic levels such as limpets and an associated reduction, in algal abundance– especially ephemerals (Bulleri et al., 2000). Similar effects could occur if predatory birds such as oystercatchers are scared away by human activities (Coleman et al. 2003). Scaring away birds will also reduce guano deposition that will reduce green algal bloom such as Prasiola, on the top of structure (Wootton, 1991).

2.2 Tools for assessment of impacts

2.2.1 Rapid field assessment protocol for evaluation of ecological conditions of the proposed LCS

As part of the scoping study (see §6.10) a rapid field assessment of local ecological features should be carried out to characterise the physical and biological features of the site and enable prediction of impacts of the planned LCS. Much of the information will also be gathered as part of site characterization s for engineering purposes and so it may be possible to make savings by combining these surveys. Below is a checklist of information to be collected in a preliminary site visit. This is based on the work that can be done by a team of experienced coastal ecologists. The time necessary to accomplish the field survey will vary depending on the site where the LCS will be built. In general, longer time is required for field surveys in the subtidal and microtidal shores due to technical difficulties in accessing the sites.

1. The site and at least two adjacent beaches 10 km to either side should be visited. In macrotidal shores, it is essential to carry out the field visit at low tide and also high tide, ideally on spring tides, whereas in microtidal habitats the visit should include a scuba diving survey. The area visited should also be defined by GPS coordinates.
2. At each site a sketch of the beach profile at low tide (or by diving, for subtidal systems), on 3-4 transects should be drawn.
3. Biotopes at various shore levels (e.g. HWS, MHWN, MTL, MLWN, MLWS on macrotidal shores or depth intervals on microtidal shores) should be described using standard classification schemes (e.g. Connor et al., 1995; Garrabou et al., 1998). Some digging and mesh sieving along with photographs of the area may be required to help identification of biotopes and characterisation of sediment characteristics (grain size, oxic layer).
4. Visits to adjacent rocky shores or any artificial structures (seaside piers, groynes, harbour walls, moles, jetties, existing sea walls etc.) should be made, carrying out a rapid assessment of rocky shore biotopes present (using BioMar classification). Particularly, evidence of scouring around any hard substrates should be noted. In the assessment, the presence of the following key species should be recorded: mussels, as they both play an important role in filtration (Wilkinson et al., 1996), but they can also interfere with performance of LCS if very abundant (by reducing porosity of structures); Sabellaria, a reef forming worms that can reduce porosity as for mussels; limpets, winkles & topshells, which are important for controlling algal growth (Jenkins et al., 1999, 2001; Thompson et al., 2000; Boaventura et al., 2002); green algae, that can represent a nuisance for recreational use of LCS and may indicate disturbance; fucoids, as they can provide an indication of wave exposure (e.g. for Atlantic: Ascophyllum is an indicator of sheltered shores whilst Fucus is an indicator of more exposed sites, Raffaelli and Hawkins, 1996); proportion of dead and live barnacles, as an index of scouring on the structures; presence of starfish and gastropod Nucella, which feed on mussels and can control their abundance (Minchin

and Dugan, 1989); Cystoseira species, as they could provide information on the environmental quality in the Mediterranean (Benedetti-Cecchi et al., 2001), as well as seagrasses that could also contribute too stabilize the coastline; Capitella and other indicators of organic enrichment in soft bottoms (Airas and Rapp, 2003). It also important to search for presence of alien species (Sargassum, Undaria, Caulerpa, Rapana, Occulina, non native oysters such as Crassotrea gigas in the UK). In the absence of hard structures navigation buoys can be a good indicator of the likelihood of local subtidal epifaunal assemblages.

5. Accumulation of algal and seagrass detritus on the beach should be quantified, as the presence of LCS is likely to increase the accumulation rate, which could have both negative and positive effects (Alongi and Tenore, 1985).

2.2.2 A biotope model for prediction of impacts on soft-bottoms

Introduction

Within the framework of the DELOS project, a methodology was developed that can be used to predict the environmental effects of adding an LCS to an area of coastline. The method is based on a combination of predictive modelling of physical changes in the environment and analysis of these changes from the viewpoint of effects on species habitats. This approach is particularly suitable for sandy beaches where the macrofauna communities are controlled almost entirely by physical processes (McArdle and McLachlan 1992) i.e. each population is structured by its response to the physical environment rather than by biological interactions (McLachlan et al. 1995).

Methodology

The methodology is based on a three-step approach, namely predictive modelling, selection of biotopes, collection of baseline data and analysis of impacts.

Predictive modelling

The DELFT3D package, developed by WL | Delft Hydraulics, amongst others, can be utilised to describe hydrodynamics, wave action and sediment transport of the midfield and farfield of a study area. Delft3D is a model system that consists of a number of integrated modules which together allow the simulation of hydrodynamic flow (under the shallow water assumption), computation of the transport of water-borne constituents such as salinity and heat, short wave generation and propagation, sediment transport and morphological changes, and the modelling of ecological processes and water quality parameters. For this methodology, the DELFT3D-FLOW module was used which performs the hydrodynamic computations and simultaneous (or "on-line") calculation of the transport of sediments, morphological changes, salinity and heat (Lesser et al. 2000, 2002).

Biotope selection

The second stage involves finding a way of linking the physical changes to effects on the ecology and this is done, for instance, by using the BioMar Classification developed for the UK and Ireland by Connor et al. (Connor et al. 1997). A biotope is defined as 'the habitat together with its recurring associated community of species, operating together at a particular scale' (Connor et al. 1997). The classification provides a link between the physical environment and its associated biological community, which is exploited in this methodology in order to predict changes in the latter as a result of changes in the former.

All the output produced by the physical model (such as current velocity, bed shear stress, height zone) are subsequently converted to classifications to match the BioMar physical parameters definitions. Other parameters used as part of the Biomar classification (salinity and substratum type) were input directly rather than produced as a result of the model.

Baseline data collection

In order to prepare an impact study, baseline data need to be collected for a study site.

a. Physical data

Bathymetric, tidal range and wave data measurements from the area are necessary as inputs for the model. For waves typical stormy weather conditions should be included as these conditions could be most structuring for local biotopes distributions. In addition a map of substratum types is needed. The substrate definitions given in the BIOMAR system are most suitable. On the basis of above data, the mathematical model can produce values for maximum bed shear stress and maximum current velocities for each cell based on combinations of weaves and currents and pre-design locations of LCS and/or other structures.

b. Biological data

Fieldwork should be carried out in order to produce an accurate map of biotopes for the real situation for comparison with the situation predicted by the model. Biotopes should be mapped using GPS to mark the boundaries. Infaunal cores should also be collected to confirm the biotope designations.

Results

The Delft3D–sediment-online software can be used to predict the environmental impact of any amount of cases; various breakwater layouts in combination with various environmental forcing conditions. The result for each case is a set of BioMar class values for physical parameters being designated for each of the 5500 cells. A procedure is then applied that selects the biotopes that can occur within the predicted set of parameter class values for each cell. Biotopes recorded in the field during baseline data survey can be compared with those predicted by the model. This enables calibration of the model to the present situation and allows evaluation of the type and magnitude of changes for each computed case in a straightforward fashion. For the field situation at the Elmer study site a total of six biotopes were mapped. The predictive accuracy of the model for the situation of a breakwater present with no waves, was 65%. For the situation of a breakwater present with waves, the model accurately predicted 69% of the biotopes that had been recorded. As expected, for the control situation without breakwater, with relatively few biotopes, the model achieved a high accuracy rate of 97% although this dropped to 76% if the situation with waves was modeled. The hierarchical nature of the BioMar classification means that the model can also be used to predict biotope complexes, the next level up in the hierarchy. These initial trials with the model are encouraging and the model is still being refined in order to develop a tool for more accurately predicting change in the identity and extent of biotopes as a result of the addition of breakwaters. More specific details as to the biotopes identified by the model as being affected by LCS structures can be found in Frost et al. (In Prep.)

ANNEX 1, BIOMAR littoral hard substrates biotope listing

Littoral rock (and other hard substrata)	LR	B - Habitat complex
Lichens or algal crusts	LR.L	C - Biotope complex
Yellow and grey lichens on supralittoral rock*	LR.L.YG	D - Biotope
Prasiola stipitata on nitrate-enriched supralittoral or littoral fringe rock*	LR.L.Pra	D - Biotope
Verrucaria maura on littoral fringe rock	LR.L.Ver	D - Biotope
Verrucaria maura and *Porphyra umbilicalis* on very exposed littoral fringe rock*	LR.L.Ver.Por	E - Sub-biotope
Verrucaria maura and sparse barnacles on exposed littoral fringe rock*	LR.L.Ver.B	E - Sub-biotope
Verrucaria maura on very exposed to very sheltered upper littoral fringe rock*	LR.L.Ver.Ver	E - Sub-biotope
Chrysophyceae on vertical upper littoral fringe soft rock*	LR.L.Chr	D - Biotope
Blidingia spp. on vertical littoral fringe soft rock	LR.L.Bli	D - Biotope
Ulothrix flacca and *Urospora* spp. on freshwater-influenced vertical littoral fringe soft rock	LR.L.UloUro	D - Biotope

Exposed littoral rock (mussel/barnacle shores)	ELR	B - Habitat complex
***Mytilus* (mussels) and barnacles**	ELR.MB	C - Biotope complex
Mytilus edulis and barnacles on very exposed eulittoral rock*	ELR.MB.MytB	D - Biotope
Barnacles and *Patella* spp. on exposed or moderately exposed, or vertical sheltered, eulittoral rock*	ELR.MB.BPat	D - Biotope
Chthamalus spp. on exposed upper eulittoral rock*	ELR.MB.BPat.Cht	E - Sub-biotope
Barnacles and *Lichina pygmaea* on steep exposed upper eulittoral rock	ELR.MB.BPat.Lic	E - Sub-biotope
Catenella caespitosa on overhanging, or shaded vertical, upper eulittoral rock*	ELR.MB.BPat.Cat	E - Sub-biotope
Barnacles, *Patella* spp. and *Fucus vesiculosus* f. *linearis* on exposed eulittoral rock	ELR.MB.BPat.Fvesl	E - Sub-biotope
Semibalanus balanoides on exposed or moderately exposed, or vertical sheltered, eulittoral rock*	ELR.MB.BPat.Sem	E - Sub-biotope
Robust fucoids or red seaweeds	ELR.FR	C - Biotope complex
Fucus distichus and *Fucus spiralis* f. *nana* on extremely exposed upper shore rock*	ELR.FR.Fdis	D - Biotope
Corallina officinalis on very exposed lower eulittoral rock*	ELR.FR.Coff	D - Biotope

Himanthalia elongata and red seaweeds on exposed lower eulittoral rock*	ELR.FR.Him	D - Biotope

Moderately exposed littoral rock (barnacle/fucoid shores)	MLR	B - Habitat complex
Barnacles and fucoids (moderately exposed shores)	MLR.BF	C - Biotope complex
Pelvetia canaliculata and barnacles on moderately exposed littoral fringe rock*	MLR.BF.PelB	D - Biotope
Fucus vesiculosus and barnacle mosaics on moderately exposed mid eulittoral rock*	MLR.BF.FvesB	D - Biotope
Fucus serratus on moderately exposed lower eulittoral rock	MLR.BF.Fser	D - Biotope
Fucus serratus and red seaweeds on moderately exposed lower eulittoral rock*	MLR.BF.Fser.R	E - Sub-biotope
Dense *Fucus serratus* on moderately exposed to very sheltered lower eulittoral rock*	MLR.BF.Fser.Fser	E - Sub-biotope
Fucus serratus and under-boulder fauna on lower eulittoral boulders*	MLR.BF.Fser.Fser.Bo	E - Sub-biotope
Fucus serratus and piddocks on lower eulittoral soft rock	MLR.BF.Fser.Pid	E - Sub-biotope
Red seaweeds (moderately exposed shores)*	MLR.R	C - Biotope complex
Mixed red seaweeds on moderately exposed lower eulittoral rock	MLR.R.XR	D - Biotope
Palmaria palmata on very to moderately exposed lower eulittoral rock*	MLR.R.Pal	D - Biotope
Mastocarpus stellatus and *Chondrus crispus* on very to moderately exposed lower eulittoral rock*	MLR.R.Mas	D - Biotope
Osmundea (*Laurencia*) *pinnatifida* and *Gelidium pusillum* on moderately exposed mid eulittoral rock*	MLR.R.Osm	D - Biotope
Ceramium sp. and piddocks on eulittoral fossilised peat*	MLR.R.RPid	D - Biotope
Ephemeral green or red seaweeds (freshwater or sand-influenced)	MLR.Eph	C - Biotope complex
Enteromorpha spp. on freshwater-influenced or unstable upper eulittoral rock*	MLR.Eph.Ent	D - Biotope
Porphyra purpurea or *Enteromorpha* spp. on sand-scoured mid or lower eulittoral rock*	MLR.Eph.EntPor	D - Biotope
Rhodothamniella floridula on sand-scoured lower eulittoral rock*	MLR.Eph.Rho	D - Biotope
Mytilus* (mussels) and fucoids (moderately exposed shores)	MLR.MF	C - Biotope complex
Mytilus edulis and *Fucus vesiculosus* on moderately	MLR.MF.MytFves	D - Biotope

exposed mid eulittoral rock		
Mytilus edulis, *Fucus serratus* and red seaweeds on moderately exposed lower eulittoral rock	MLR.MF.MytFR	D - Biotope
Mytilus edulis and piddocks on eulittoral firm clay	MLR.MF.MytPid	D - Biotope
Littoral *Sabellaria* (honeycomb worm) reefs*	MLR.Sab	C - Biotope complex
Sabellaria alveolata reefs on sand-abraded eulittoral rock	MLR.Sab.Salv	D - Biotope

Sheltered littoral rock (fucoid shores)	SLR	B - Habitat complex
Dense fucoids (stable rock)	SLR.F	C - Biotope complex
Pelvetia canaliculata on sheltered littoral fringe rock*	SLR.F.Pel	D - Biotope
Fucus spiralis on moderately exposed to very sheltered upper eulittoral rock*	SLR.F.Fspi	D - Biotope
Fucus vesiculosus on sheltered mid eulittoral rock*	SLR.F.Fves	D - Biotope
Ascophyllum nodosum on very sheltered mid eulittoral rock	SLR.F.Asc	D - Biotope
Ascophyllum nodosum on full salinity mid eulittoral rock*	SLR.F.Asc.Asc	E - Sub-biotope
Ascophyllum nodosum, sponges and ascidians on tide-swept mid eulittoral rock*	SLR.F.Asc.T	D - Biotope
Ascophyllum nodosum and *Fucus vesiculosus* on variable salinity mid eulittoral rock*	SLR.F.Asc.VS	E - Sub-biotope
Fucus serratus on sheltered lower eulittoral rock	SLR.F.Fserr	D - Biotope
Fucus serratus, sponges and ascidians on tide-swept lower eulittoral rock	SLR.F.Fserr.T	E - Sub-biotope
Fucus serratus and large *Mytilus edulis* on variable salinity lower eulittoral rock	SLR.F.Fserr.VS	E - Sub-biotope
Fucus ceranoides on reduced salinity eulittoral rock*	SLR.F.Fcer	D - Biotope
Fucoids, barnacles or ephemeral seaweeds (mixed substrata)	SLR.FX	C - Biotope complex
Barnacles and *Littorina littorea* on unstable eulittoral mixed substrata	SLR.FX.BLlit	D - Biotope
Fucus vesiculosus on mid eulittoral mixed substrata*	SLR.FX.FvesX	D - Biotope
Ascophyllum nodosum on mid eulittoral mixed substrata*	SLR.FX.AscX	D - Biotope
Ascophyllum nodosum ecad *mackaii* beds on extremely sheltered mid eulittoral mixed substrata*	SLR.FX.AscX.mac	E - Sub-biotope
Fucus serratus on lower eulittoral mixed substrata	SLR.FX.FserX	D - Biotope
Fucus serratus with sponges, ascidians and red seaweeds on tide-swept lower eulittoral mixed substrata*	SLR.FX.FserX.T	E - Sub-biotope
Ephemeral green and red seaweeds on variable salinity or disturbed eulittoral mixed substrata	SLR.FX.EphX	D - Biotope
Fucus ceranoides on reduced salinity eulittoral mixed	SLR.FX.FcerX	D - Biotope

substrata*		
Mytilus **(mussel) beds (mixed substrata)**	SLR.MX	C - Biotope complex
Mytilus edulis beds on eulittoral mixed substrata*	SLR.MX.MytX	D - Biotope
Rockpools	LR.Rkp	C - Biotope complex
Green seaweeds (*Enteromorpha* spp. and *Cladophora* spp.) in upper shore rockpools*	LR.Rkp.G	D - Biotope
Corallina officinalis and coralline crusts in shallow eulittoral rockpools*	LR.Rkp.Cor	D - Biotope
Coralline crusts and *Paracentrotus lividus* in shallow eulittoral rockpools	LR.Rkp.Cor.Par	D - Biotope
Bifurcaria bifurcata in shallow eulittoral rockpools*	LR.Rkp.Cor.Bif	E - Sub-biotope
Cystoseira spp. in shallow eulittoral rockpools*	LR.Rkp.Cor.Cys	E - Sub-biotope
Fucoids and kelps in deep eulittoral rockpools*	LR.Rkp.FK	D - Biotope
Sargassum muticum in eulittoral rockpools*	LR.Rkp.FK.Sar	E - Sub-biotope
Seaweeds in sediment (sand or gravel)-floored eulittoral rockpools	LR.Rkp.SwSed	D - Biotope
Hydroids, ephemeral seaweeds and *Littorina littorea* in shallow eulittoral mixed substrata pools*	LR.Rkp.H	D - Biotope
Overhangs and caves	LR.Ov	C - Biotope complex
Rhodothamniella floridula in upper littoral fringe soft rock caves	LR.Ov.RhoCv	D - Biotope
Sponges and shade-tolerant red seaweeds on overhanging lower eulittoral bedrock	LR.Ov.SR	D - Biotope
Sponges, bryozoans and ascidians on deeply overhanging lower shore bedrock	LR.Ov.SByAs	D - Biotope

ANNEX 2, BIOMAR biotope sheet, example

Barnacles and fucoids (moderately exposed shores)		
Code: MLR.BF	Habitat complex: Moderately exposed littoral rock	C - Biotope complex

Habitat classification

Salinity: Full
Wave exposure: Moderately exposed
Substratum: Bedrock; boulders
Zone: Eulittoral
Height band: Upper shore, Mid shore, Lower shore

Biotope description

On moderately exposed rocky shores the extent of fucoid cover is typically less than that found on sheltered shores (SLR.F). The fucoids form a mosaic with barnacles on bedrock and boulders, rather than the blanket cover associated with sheltered shores, except for on the lower shore where there may be dense *Fucus serratus* (MLR.Fser). Beneath the band of lichens at the top of the shore (LR.YG and LR.Ver) the channel wrack *Pelvetia canaliculata* typically occurs overgrowing the black lichen *Verrucaria* spp. with sparse barnacles (MLR.PelB). Below, barnacles and limpets *Patella* may cover extensive areas of rock (ELR.BPat), particularly on steep or vertical rock. In the absence of ELR.BPat, the spiral wrack *Fucus spiralis* may occur (SLR.Fspi). On the mid shore the bladder wrack *Fucus vesiculosus* generally forms a mosaic with barnacles (MLR.FvesB). Finally, the serrated wrack *Fucus serratus*, dominates the lower shore (MLR.Fser); a number of sub-biotopes have been described: lower shore bedrock and boulders may be characterised by mosaics of *F. serratus* and turf-forming red algae (MLR.Fser.R); where the density of *F. serratus* is greater (typically common - superabundant) and the abundance of red algae less MLR.Fser.Fser should be recorded. The presence of boulders and cobbles on the shore can increase the micro-habitat diversity which often results in a greater species richness. Although the upper surface of the boulders may bear very similar communities to MLR.Fser.Fser there is often an increase in fauna (crabs, tube-worms, sponges and bryozoans) and MLR.Fser.Fser.Bo should be recorded.

Characterising species

Species	Typical abundance	Frequency	Faithfulness
Chthamalus montagui	Frequent	*	***
Semibalanus balanoides	Frequent	****	*
Patella vulgata	Frequent	****	*

Gibbula umbilicalis	Occasional	*	**
Melarhaphe neritoides	Frequent	*	**
Flustrellidra hispida	Occasional	*	**
Lomentaria articulata	Occasional	**	**
Membranoptera alata	Occasional	*	**
Leathesia difformis	Occasional	*	**
Ascophyllum nodosum	Occasional	*	**
Fucus serratus	Abundant	****	*
Pelvetia canaliculata	Frequent	*	**
Verrucaria maura	Common	*	**

ANNEX 3, Example model application

Using Delft3D-FLOW and D3d-WAVE, after calibration to measurements, calculations were executed for two lay-outs (with and without breakwaters), representing limiting conditions for the local species communities. In essence these runs aim for an extreme in wave exposure and currents that still occurs yearly and therefore could limit the suitability of local habitats for species with lifespans of more then a year. Therefore Hs has been increased from Hs=1.0m to Hs=2.5m in these runs. The parameters velocity, bed shear stress, depth and substrate are linked to occurrence of species communities through statistical analysis of many samples. The results of these runs are then used for biotope analysis.

Table 1 Tidal components estimated for the study site

	M2		S2		K1		O1	
	phase	amp	phase	amp	phase	amp	phase	amp
western bnd	323	1.8950	13.000	0.5700	131.000	0.0550	12.500	0.0550
eastern bnd	323	1.9035	13.342	0.5666	128.382	0.0556	12.215	0.0522

Velocity

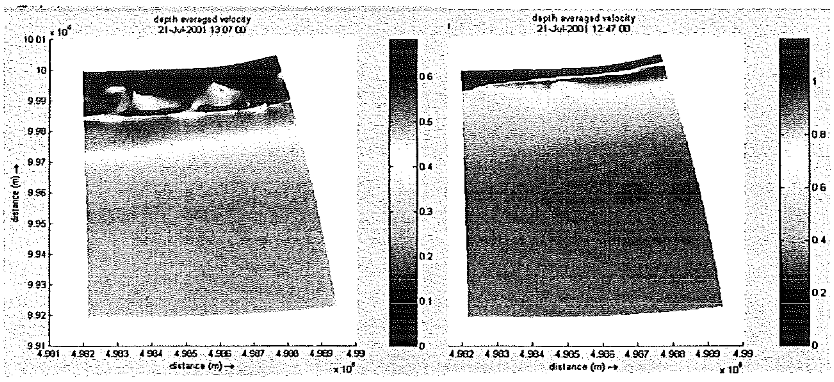


Figure 1 Velocity distributions near high water with and without breakwaters (Hs =2.5 m, mean Hs= 1.0 m). Wave effect on velocity are significant.

Bed shear stress

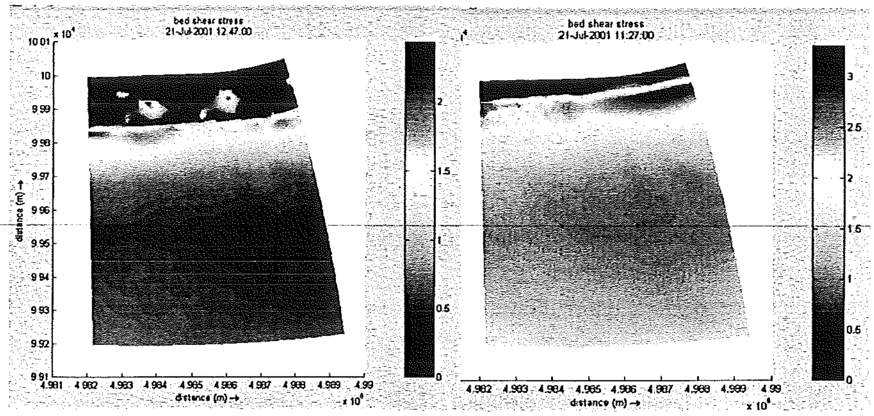


Figure 2 Total bed shear stress distributions near high water with and without breakwaters (Hs =2.5 m). Note the different scaling. Bed shear stress is calculated as combination of shear stress caused by tidal currents and waves.

Height and Depth

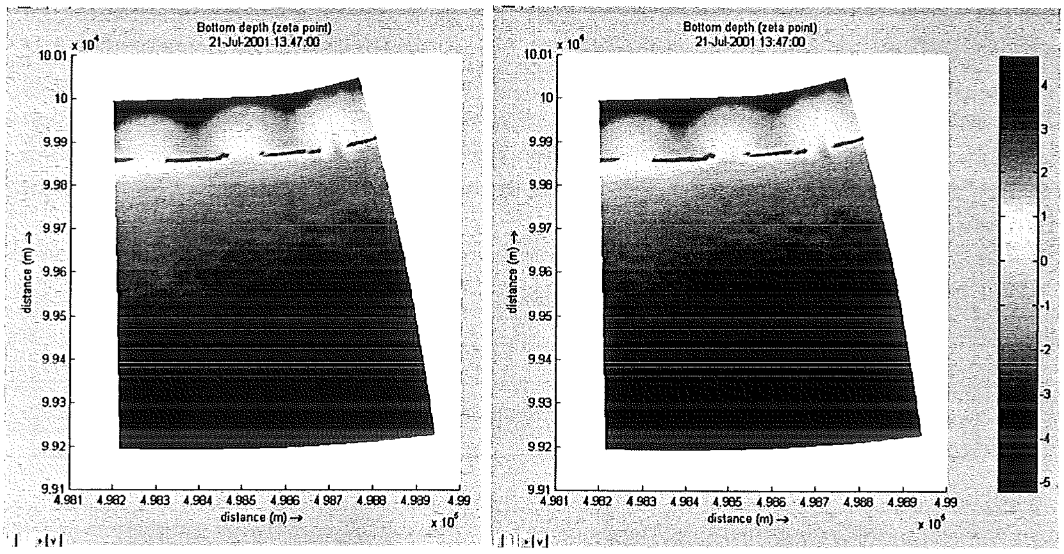


Figure 3 Bottom depth distributions relative to MSL with and without breakwaters (negative values are above MSL).

Substrate

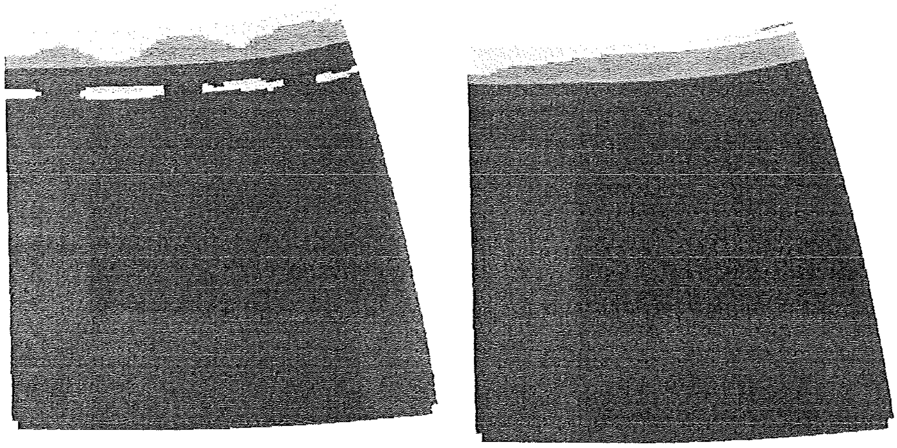


Figure 4 Substrate map of the study area (orange = fine sand, yellow = boulders and pebbles)

This map has been composed on the basis of some sediment samples that are available for the area. Sediment composition is assumed to become coarser (from fine sand to pebbles) going from subtidal to strandline depths-zones.

In order to test the performance of the BioMar biotope classification system, runs have been performed in order to calculate the distribution of both the more global biotope-complexes and the biotopes for an existing field situation. The results are based on two runs of the Delft3D morpho-hydrodynamic model, (1) storm weather situation (Hs=2.5m) with breakwater present and (2) storm weather present situation without a breakwater. Figure 5 shows both biotope and biotope-complex distributions for hard and soft substrates for the situation with the breakwater. The summation of areas gives a total biotope area per run in the tables 2 and 3 below. It can be seen that not all gridcells have been included in the classification (bright yellow colour). Especially the more detailed biotope system leaves areas unclassified.

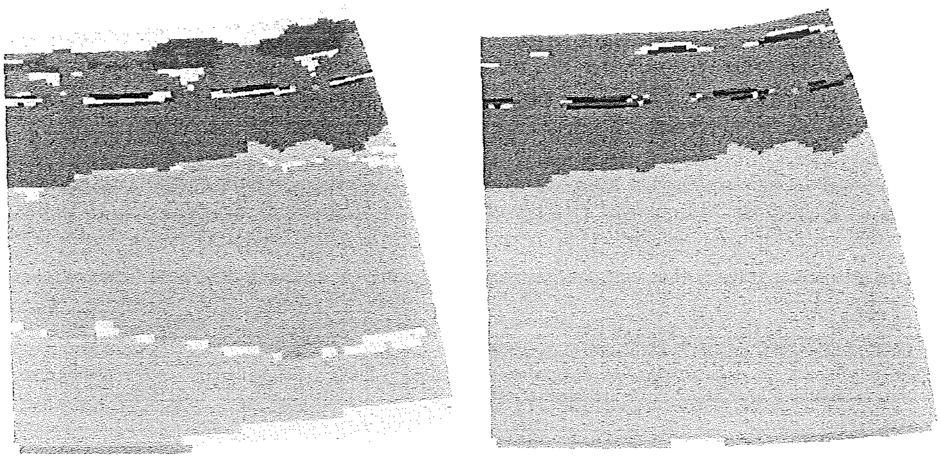
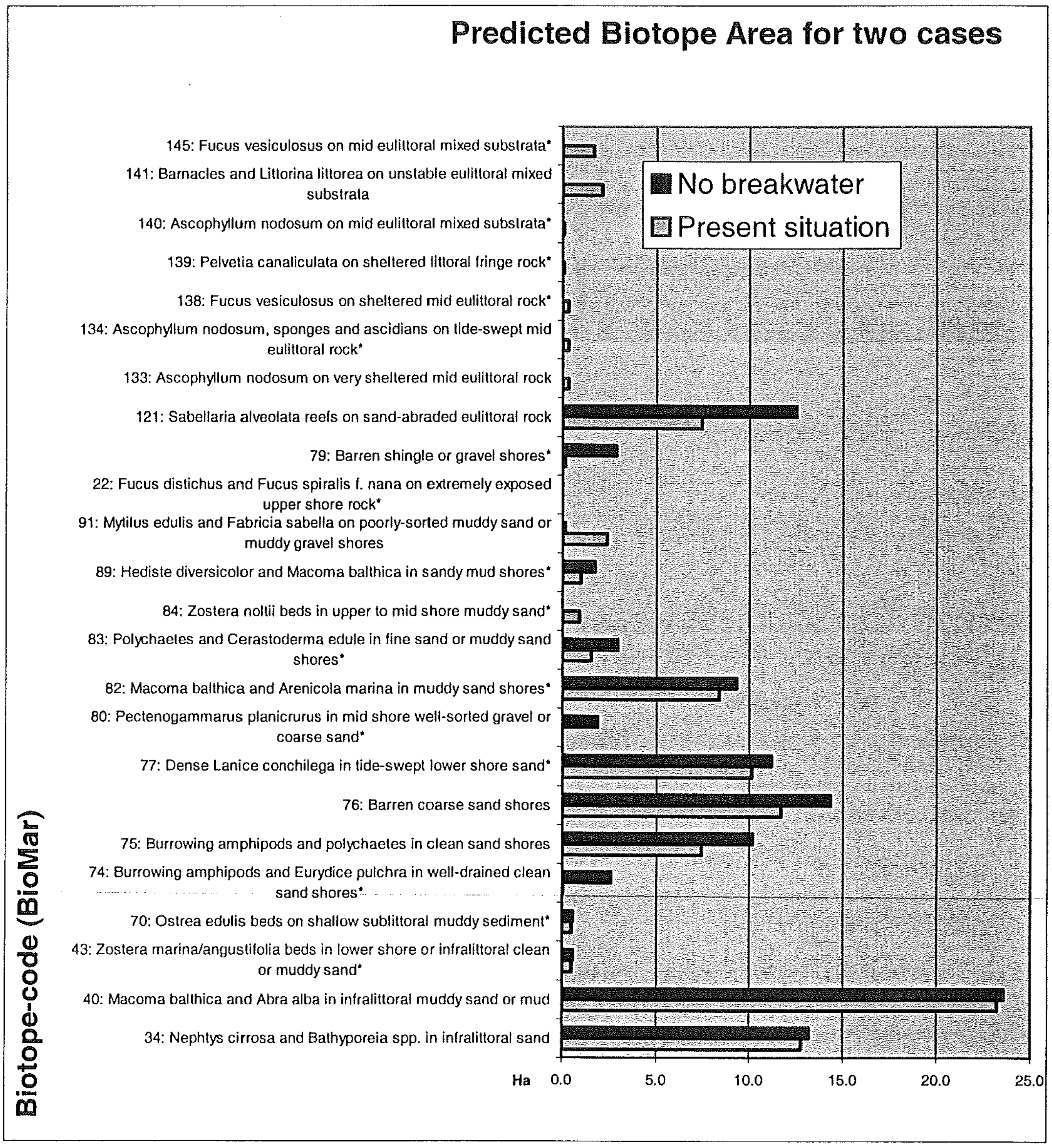


Figure 5 Biotope and biotope-complex results for present situation (Hs=2.5m, with breakwater)

Table 2 Biotope distributions for a run with and without breakwater.



The biotope description gives information on species to be expected in the environment of the study area. Figure 6 shows a detail with number of biotopes that link to numbers in Table 2.

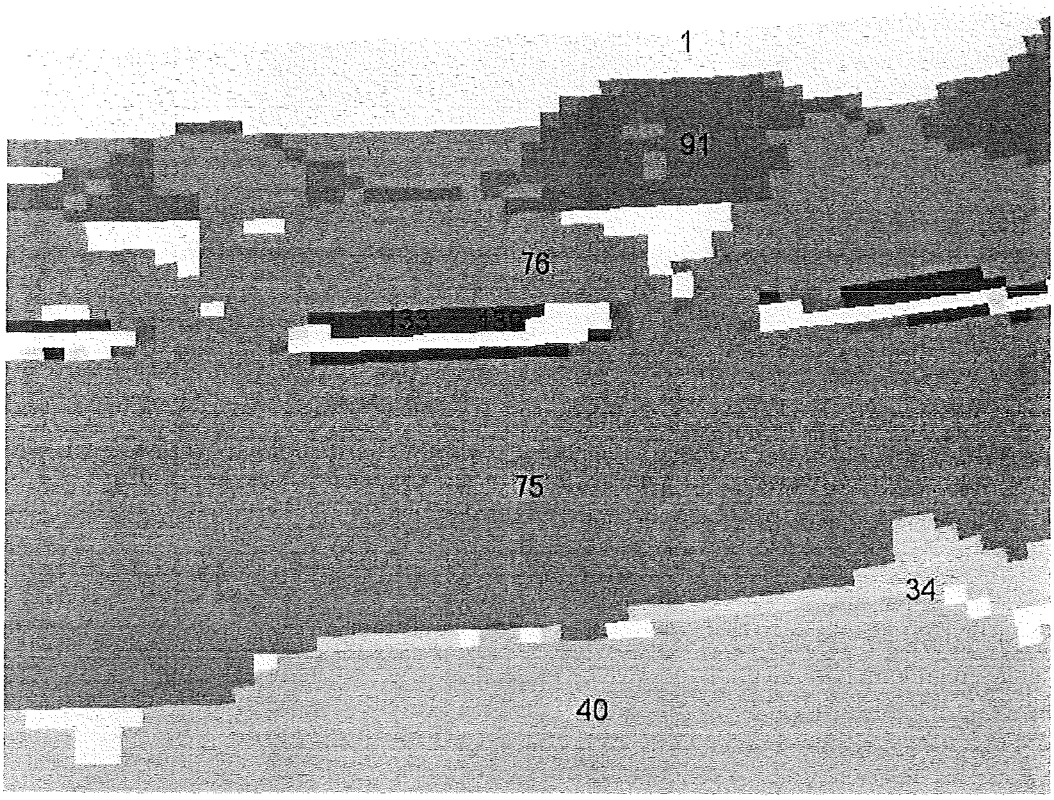
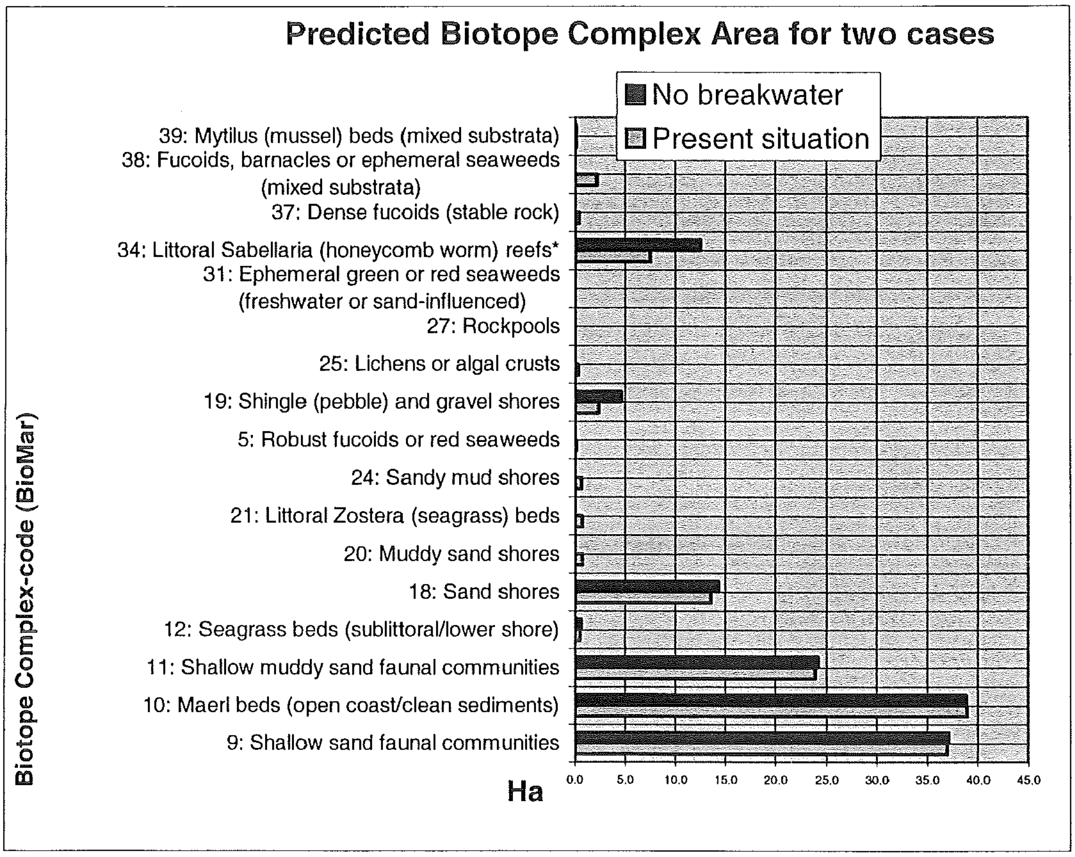


Figure 6 A detail of the study area showing biotope distributions (value 1 indicates missing data).

Table 3 Biotope-complex distributions for a run with and without breakwater



The biotope-complex description gives global information on species to be expected in the environment of the study area. Figure 7 shows a detail with number of biotopes that link to numbers in Table 3.

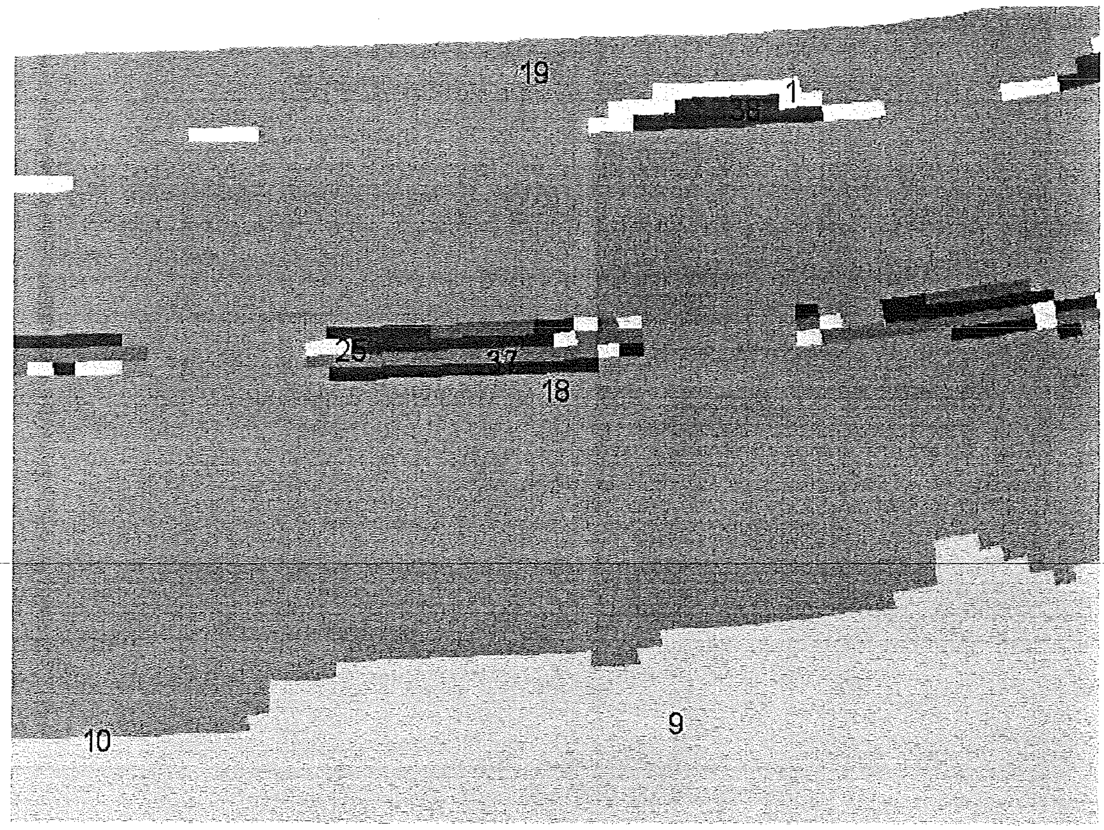


Figure 7 A detail of the study area showing biotope-complex distributions (value 1 indicates missing data).

Species list for most abundant biotopes

The BioMar dataset includes for most of the biotopes a list of species that are occurring in that biotope. In the table 4, this list is presented for 5 most abundant biotopes in the study area. In grey entries, species are listed that were also found by Southampton Oceanography Centre (P. Moschella et al.) in soft sediment sample site near (but not in) the study area.

Table 4 List of species potentially occurring in characteristic biotopes

IGS.FaS.NcirBat	**IMS.FaMS.MacAbr**	**LGS.S.AP**	**LMS.MS.MacAre**	**MLR.Sab.Salv**
34	**40**	**75**	**82**	**121**
Nephtys	Nephtys cirrosa	Nephtys cirrosa	Nephtys hombergii	Sabellaria alveolata
Nephtys cirrosa	Nephtys hombergii	Scolelepis squamata	Scoloplos armiger	Semibalanus balanoides
Scolelepis squamata	Scalibregma inflatum	Pontocrates	Pygospio elegans	Balanus crenatus
Spio filicornis	Lagis koreni	Bathyporeia	Arenicola marina	Balanus perforatus
Capitella capitata	Ampelisca spinipes	Eurydice pulchra	Ventrosia ventrosa	Elminius modestus
Pontocrates arenarius	Crangon crangon		Cerastoderma edule	Patella vulgata
Bathyporeia	Nucula nitidosa		Macoma balthica	Littorina littorea
Haustorius arenarius	Fabulina fabula			Nucella lapillus
Eurydice pulchra	Macoma balthica			Mytilus edulis
	Abra alba			Porphyra
	Echinocardium cordatum			Palmaria palmata
				Mastocarpus stellatus
				Ceramium
Ammodytes tobianus	Mya			Cladostephus spongiosus
	Corbula gibba			Fucus serratus
				Fucus vesiculosus
				Enteromorpha
				Ulva

3. Applying ecological structures

A new topic of many studies is bioengineering, with applications that are related to application of ecological components for enhancing coastal defence.

3.1 Classic: Use of saltmarshes for reclamation and protection

In the Netherlands salt marshes have been used for centuries to reclaim land *and* to protect the hinterland from the sea. The interaction between biology and physical processes needed to obtain this effect is subject of more and more studies. The value of saltmarshes for coastal protection is considerable and influences the design of seadefenses in many locations. Below text has been extracted from NWO-STW proposal for detailed study into saltmarsh dynamics.

3.1.1 The coastal defense value of salt marshes

Ongoing climate change is expected to result in accelerated sea level rise and increasing frequencies of extreme storm events during the next decades (IPCC, 2001). Global sea level has risen by about 10 to 20 cm during the 20th century and is expected to rise by 20 to 80 cm during the next century. Accelerated sea-level rise and increased storm frequency are a significant threat to low-lying coastal areas. The Netherlands forms an extreme example: over 50% of the total area of the Netherlands is currently below sea level and would have been flooded already today if no coastal protection by dikes and dunes would be present. The construction and maintenance of dikes are an enormous cost. Ongoing soil compaction and subsidence of polder areas further increases the catastrophic impact that future flooding would have in The Netherlands. Considering this combination of accelerated sea-level rise, increased storm frequency and soil subsidence, it is clear that the traditional engineering approach of continuous raising, widening and maintenance of dikes is not sustainable in the long term. For this reason, there is a need to explore new, alternative methods for coastal protection.

Salt marshes may offer opportunities for *alternative, cost-effective coastal defense* (e.g. Brampton, 1992; Turner and Dagley, 1993; Leggett and Dixon, 1994; Möller et al., 2001). Salt marsh vegetation is very effective in attenuating waves and currents during storm conditions (Möller et al. 1999). Furthermore, salt marsh vegetation is very effective in sediment trapping: as a result of this sediment accumulation, salt marshes quickly raise to a high elevation in the tidal frame and they have a high capacity to maintain this elevation relative to the rising sea level (Temmerman et al. 2004). In this respect, the presence of a salt marsh buffer between the sea and the land may strongly reduce the costs of constructing and maintaining dikes (e.g., Brampton, 1992; Turner and Dagley): dikes that have a salt marsh in front of them will be exposed to relatively weak currents and relatively low wave heights (because of attenuation of hydrodynamic forces by the salt marsh vegetation) and only during short time periods (because of the high salt marshes in front of the dikes). As a consequence, dikes that have a salt marsh in front of them do not need to be as high and wide, and do not require as much maintenance, as dikes that are directly bordering the sea and, consequently, are subject to direct wave impact and strong tidal currents. It has been suggested, for example, that as little as six meter of marsh vegetation can have such strong wave attenuation, that the presence of a marsh may reduce the cost of building the dike by one third (Turner and Dagley, 1993).

Salt marshes may be created either in seaward direction, e.g. by stimulating salt-marsh formation in between breakwaters, or in landward direction, e.g. by complete or partial removal of dikes and rebuilding of new dikes or restoration of old dikes more inland. The latter concept of "de-poldering" or "managed realignment", in order to create salt marsh buffer zones for coastal defense, is increasingly being implemented, e.g. in the U.K. (e.g., Pethick 2002). In The Netherlands, the use of salt marshes for coastal protection is most feasible in the Scheldt-Rhine-Meuse delta and in the Wadden Sea and Dollard. Some areas of "de-poldering" have already been realized in the Wadden Sea area, but with emphasis on nature restoration and conservation.

3.1.2 The ecological value of salt marshes

Salt marshes are widely recognized as valuable ecosystems. Costanza et al. (1997) estimated the economic value of the ecosystem services provided by salt marshes at about 10000 US \$ ha^{-1} yr^{-1}, which is very high compared to terrestrial ecosystems. The economically most important ecosystem service of salt marshes is their role in biogeochemical cycling in coastal zones: salt marshes act as an important natural filter (sink) for suspended sediments, nutrients and contaminants that otherwise would drain from the land into estuarine and coastal waters (e.g. Jickells, 1998). Furthermore, salt marshes play an important role as feeding and breeding areas for dense populations of e.g. fish, shellfish, crustaceans and birds (e.g., Allen and Pye 1992). The recognition of this ecosystem value of salt marshes has lead to several international protective regulations (e.g., Ramsar Convention (http://www.ramsar.org); European Community Habitats Directives (http://www.ecnc.nl)).

The *dynamic nature of salt marshes* is determinant for their ecological value. Salt marshes are characterized by a clear zonation in species, such as plant species: pioneer vegetation is found at the seaward side (i.e., low marsh) and climax vegetation near to the landward side (i.e., high marsh). This zonation is however dynamic, as sediment deposition during tidal flooding will cause low marsh to evolve into high marsh. In areas where hydrodynamic conditions are relatively mild and elevation permits, new pioneer vegetation may establish at the seaward front of this accreting marsh, so that over time the marsh will gradually extent in seaward direction. In areas where hydrodynamics are relatively strong, the establishment of new pioneer vegetation in seaward direction may be prevented and the salt marsh may erode in landward direction. Ongoing vertical sediment accretion on this landward eroding marsh will result into monotone climax vegetation on the marsh. After significant salt marsh retreat due to erosion, hydrodynamic conditions in front of the eroding marsh *may* be suitable again for re-establishment of new pioneer vegetation. In ideal situations, the above mentioned sequence of processes will result in cyclic behavior of salt marshes, where periods of seaward salt marsh extension alternate with periods of landward salt marsh erosion. This dynamic nature of salt marshes is crucial for the cyclic rejuvenation of monotone climax vegetation into new pioneer vegetation and is determinant for the preservation of habitat and species diversity.

3.1.3 Problem: sustainable management of salt marshes

Despite the coastal defense value and ecological value of salt marshes, their surface area has strongly declined in The Netherlands during the past centuries. Human activities such as land reclamation for agricultural, industrial and urban development (e.g., Dutch Delta, Wadden Sea) and dredging for navigation (e.g., Westerschelde) have strongly reduced the total area of salt marshes in The Netherlands. The majority of the remaining salt marshes are restricted in their size, enclosed by a dike on the landward border and a narrow mudflat on the seaward border. The combination of:

(1) the increasing recognition of the coastal defense value of salt marshes,
(2) international protective regulations that call for maintaining salt marsh areas as valuable ecosystems,
(3) ongoing intensive human use of estuaries and coastal zones,

emphasizes the *need for a sustainable management strategy* for salt marshes, which guarantees the protection of existing and newly created salt marsh areas, with preservation of their dynamic nature and in harmony with ongoing human use of estuaries and coastal zones. However, due to the inherent dynamic nature of salt marshes, the best strategy to manage these systems is not yet fully understood.

With respect to management, we may distinguish three different levels:

(1) At the _(inter)national scale_, the most urgent management questions deal with long-term coastal protection in view of accelerated sea-level rise, increased storm frequency and soil subsidence. This may require exploration of new, alternative and/or additive methods for coastal protection, as it can not be expected that the traditional approach of continuous raising, widening and maintenance of dikes will be sustainable in the long term. If possible, coastal protection should combine added nature values. The use of salt marsh buffer zones may be a suitable additive measure to dike building, that combines coastal protection and nature conservation. Up to now, however, it is unclear whether salt marsh buffer zones are sustainable in the long term or whether salt marsh shores will erode when sea-level rises and storm frequency increases. As a consequence, it is still a high risk for managers to choose for the creation of salt marsh buffer zones. Deeper insights and predictive models of salt marsh evolution are needed to support such management decisions at an (inter)national level.

(2) At the _estuary scale_, management questions deal with how to combine nature conservation with continuously growing anthropogenic demands on the estuarine system, such as dredging for navigation and land reclamation. The complications in this respect may be well illustrated e.g by the management study made by ProSes (Jeuken et al., 2004). In this study, the impact of further deepening of the navigation channel and de-poldering on the morphology of the Westerschelde was studied based on model simulations. However, although the ProSes study underlined the high importance of the salt marshes for the nature value of this estuarine ecosystem, the present models were not able to make predictions of the effects on the salt marshes. This underlines that suitable models, including salt marsh processes, are needed for the management of estuaries and coastal zones.

(3) At the _marsh scale_, questions deal with how to manage a salt marsh so that the long-term preservation of its coastal defense value and ecological value is sustained. In this respect, we propose that two contrasting management strategies may be distinguished: dynamic vs. preservative management. _Dynamic management_ is based on the idea of cyclic salt marsh behavior by allowing the occurrence of natural dynamic processes of landward erosion and seaward re-establishment. With respect to coastal protection, dynamic salt marsh management is very cost-effective: no structures, such as dams, brick stones, etc., are needed, since the idea is that landward erosion will alternate with seaward salt marsh re-establishment. With respect to the ecological value of salt marshes, cyclic erosion and re-establishment causes a natural rejuvenation of the vegetation, which increases biodiversity and reduces the risk of development into monotone climax vegetation. However, dynamic management includes the risk that landward erosion may continue up to the dike, *without* re-establishment, so that the dike becomes subject to direct wave impact and so that salt marsh habitat is permanently lost. The latter may be prevented by so called _preservative management_. Preservative management aims at maintaining the position of the salt marsh shoreline by fortifying the shoreline with e.g. dams, brick stones, etc. However, this fixation of the seaward marsh edge requires extra costs, while ongoing sediment accretion on the marsh will change the salt marsh habitat into monotone climax vegetation that lack natural dynamics. Thus, where possible, dynamic management is preferred, since it is most cost-effective and it preserves the natural dynamics of salt marshes. However, choosing between dynamic and preservative management is hampered by the _limited scientific knowledge on the cyclic behavior of salt marshes_. Up to now, it is unclear *where* dynamic management is possible (how much space for cyclic salt marsh behavior is needed?) and *when* salt marsh managers should take the decision to shift from dynamic to preservative management and vice versa (to what extent does environmental changes, such as sea level rise and increased storm frequency, lead to a higher vulnerability for permanent erosion of salt marsh sites?). In order to support such management decisions, novel scientific insights and models of cyclic salt marsh behavior are needed.

In order to resolve erosion problems along the coast of Louisiana, innovative solutions have been explored that may improve effectiveness and cost efficiency (Campbell, 2004).

3.2 Innovation: Bioengineered submerged breakwater

This research involves a technology, termed an "oysterbreak", which is a bioengineered submerged breakwater. This structure promotes oysters to form a dense structure that dissipates wave energy. Since the structure is biologically dominated, initial material use is modest.
The oysterbreak was evaluated through a series of experiments. Settlement patterns were analyzed by quantifying the biological fouling on the structure during its deployment in Grand Isle, Louisiana for one year. Secondly, settlement preference on materials was analyzed in a tank under various flows. To investigate further, the wave interactions with various scaled designs were also analyzed in a wave tank. The transmission, reflection, and dissipation characteristics were determined as growth occurred. Lastly, a predictive model was developed from the results.
Experiments suggest that a uniform distribution pattern could be expected in the absence of predation. Also, it was shown that mortar coating was superior for oyster settlement to PVC pipe and commercially available oyster tubes. The wave tank experiments concluded that wave transmission through the structure decreased as growth occurred. It was also shown that a structure with 2 vertical slats/metre, could be used to effectively dissipate waves.
The predictive model developed suggests that the oysterbreak can be used in field conditions. The model showed that after one year of growth, an oysterbreak 20 meters wide has the capacity to reduce wave energy by 70%. This prediction is consistent with other submerged breakwater designs. The results of these experiments will be used to design, deploy and monitor full scale oysterbreaks.

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Kustverdediging van de koppen van de Waddeneilanden

De dynamiek van de kust nabij buitendelta's en passende maatregelen voor het kustbeheer

Syllabus voor de PAO cursus Innovatief denken en doen in kustverdediging
Een uitsnede uit het rapport RIKZ/2004.017

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Samenvatting

De buitendelta's zijn zandrijke ondieptes zeewaarts van de zeegaten die tussen de Waddeneilanden liggen. De buitendelta's bestaan uit platen, die merendeels onder water liggen, en getijdegeulen. De geulen en platen op de buitendelta's zijn dynamisch, zowel de locatie als de hoogte verandert in de loop van de tijd.

De veranderingen op de buitendelta's beïnvloeden de aangrenzende kust, doordat geulen in de richting van de kust verplaatsen en doordat zandplaten met de kust verhelen. Ten opzichte van de 'normale' doorgaande kust zijn de kustprofielen van eilandkoppen zowel extreem steil, wanneer een getijdegeul onder de kust ligt, als extreem flauw, bij verheelde zandplaten. Bij de koppen van de Waddeneilanden én de kop van Noord-Holland is de achteruitgang van de kustlijn meestal gekoppeld aan de landwaartse verplaatsing van een getijdegeul.

De buitendelta's vormen een schakel in het zanddelende systeem, samen met de Waddenzee en de eilandkusten. Onder invloed van de stijgende zeespiegel 'vraagt' de Waddenzee zand, dat voor een belangrijke deel wordt geleverd door de eilandkusten, met kustlijnachteruitgang als gevolg.

Het huidige Nederlandse kustbeleid heeft als strategische doel het duurzaam handhaven van de veiligheid en duurzaam behoud van de functies en waarden in het duingebied. Beheermaatregelen worden genomen wanneer de achteruitgang van de kust waarden en functies van het achterliggende gebied bedreigd. Bij de koppen van de eilanden zijn zowel ontwikkelingen op de buitendelta, als de zandvraag van de Waddenzee oorzaak van achteruitgang van de kust. De beheermaatregelen moeten zó worden ingericht dat functies en waarden kunnen worden behouden. Voortkomend uit het strategische doel van het kustbeleid zijn twee operationele doelstellingen voor beheermaatregelen geformuleerd:
1. Het tegengaan van structurele kustachteruitgang.
2. Het aanvullen van de zandvoorraad van het gehele zanddelende systeem.
Samen met de drietrapsstrategie van zand vrij laten bewegen (niet ingrijpen) – zandbuffers aanleggen (suppleren)– zand vasthouden (harde constructies) vormen de twee doelstellingen het uitgangspunt voor het beoordelen van verschillende beheermaatregelen voor eilandkoppen.

Strandsuppleties
Strandsuppleties voeden de ondiepe kustzone. Wanneer de kustachteruitgang wordt veroorzaakt door de landwaartse verplaatsing van een geul compenseren strandsuppleties dit slechts voor een deel. Voor het tegengaan van de landwaartse geulverplaatsing moeten andere methoden dan strandsuppleties in stelling worden gebracht. Wanneer Noordzeezand (zand van buiten het zanddelend systeem) gebruikt wordt voor suppleties van de kustzone, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem.

Harde verdedigingen (bestorting, strandhoofden, korte en lange dammen)
Wanneer ontwerp en uitvoering van de harde kustverdedigingen goed passen bij het erosieprobleem kunnen deze lokaal zeer effectief zijn. Harde kustverdedigingswerken leveren echter geen aanvulling van de zandvoorraad

van de Nederlandse kust en belemmeren in sommige gevallen het doorgaande zandtransport langs de kust.

Zandbuffers (geulwandsuppleties, zanddammen)
Indien voldoende zand wordt aangebracht, langs de geulwand of dwars op de geul, zal een getijdegeul tijdelijk uit de kust worden gehouden. Ervaringen met grote ingrepen met zand rond eilandkoppen zijn beperkt, zodat niet op voorhand is vast te stellen hoe lang een dergelijke ingreep meegaat. Zandbuffers van Noordzeezand dragen bij aan de tweede doelstelling voor het kustbeheer, het op peil houden van het zandbudget van de Nederlandse kust.

Zandsluitingen
In situaties met structurele kusterosie als gevolg van de aanwezigheid van een relatief kleine getijdegeul onder kust kan een zandsluiting van de getijdegeul overwogen worden. De oorzaak van de kustachteruitgang wordt hiermee weggenomen en ter plaatse ontstaat een zandbuffer voor de kust. Zandsluitingen voldoen aan beide operationele doelstellingen voor het kustbeheer, mits het zand op de Noordzee wordt gewonnen. Ervaringen met zandsluitingen ten bate van het kustbeheer ontbreken vooralsnog.

Aanbevolen wordt om operationele kennis op te doen met het aanbrengen van zandbuffers en het uitvoeren van zandsluitingen bij eilandkoppen. Deze ingrepen kunnen gebruikt worden om tijdelijke fasen van erosie, die kenmerkend zijn voor de ontwikkelingen bij eilandkoppen, te overbruggen.

Zandwinning
Voor zandwinning zijn op de Noordzee, buiten de –20 m NAP contour zandwingebieden aangewezen. Zandwinning is niet toegestaan op buitendelta's, in de kustzone en in de Waddenzee. Het zandwinbeleid voor de Waddenzee en aangrenzende gebieden is onder andere ingegeven door de kennis van het zanddelende systeem. De tweede doelstelling van het kustbeheer, aanvullen van de zandvoorraad van het fundament van de kust, niet wordt bereikt als het suppletiezand gewonnen wordt op de buitendelta's. Zandwinning op de buitendelta's is daarom een tijdelijke maatregel,die niet wordt aanbevolen.

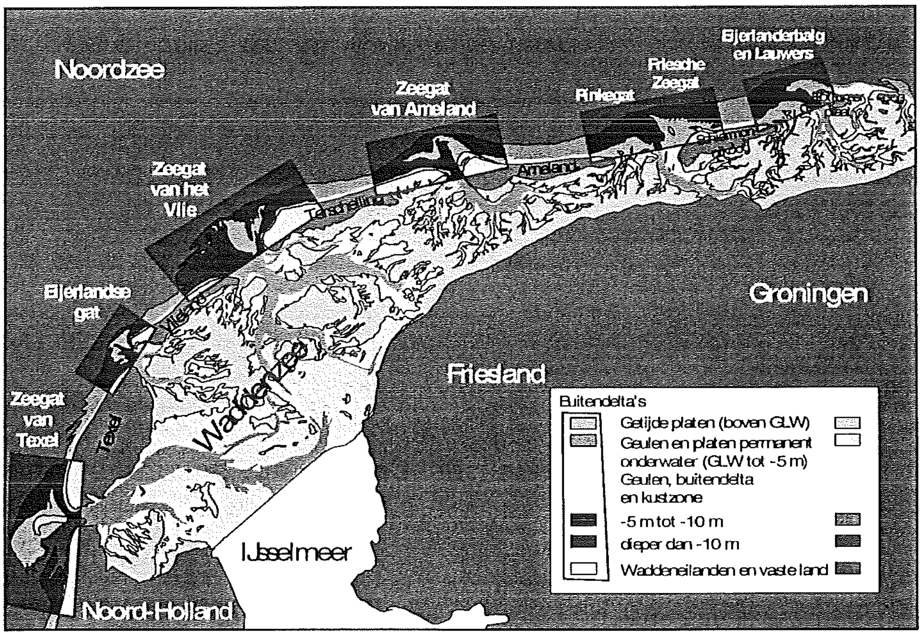
Bovenstaande overzicht is een algemene schets van de voor- en nadelen van beheermaatregelen voor eilandkoppen. Lokale factoren bepalen deels ontwerp, morfologische ontwikkelingen en de impact van de verschillende beheermaatregelen. Per locatie dient daarom een afweging gemaakt te worden over de haalbaarheid van de verschillende beheermaatregelen. Naast de bovenstaande operationele doelstellingen van het kustbeheer moeten de beheermaatregelen gericht zijn op het duurzaam behoud van de functies en waarden in het kustgebied. Hiervoor dienen ook de korte en lange termijn baten en lasten, risico's en effecten op ecologie en waterkwaliteit te worden vergeleken.

Inhoudsopgave

1 Inleiding

1.1 De Koppen van de Waddeneilanden

De koppen van de Waddeneilanden én de kop van Noord-Holland wijken in vorm en dynamiek af van de Hollandse kust en de centrale delen van de Waddeneilanden. Belangrijkste oorzaak voor de andere vorm en dynamiek is de invloed van de buitendelta's op de kust (figuur 2.1). Voor een antwoord op de beheervragen wordt daarom dieper ingegaan op de buitendelta's en hun invloed op de kust en het zanddelende systeem.



Figuur 2.1:
De buitendelta's van de Waddenzee, de koppen van de Waddeneilanden en de Kop van Noord-Holland.

1.2 Het kustbeheer van de koppen van de Waddeneilanden

In sommige gevallen nopen ontwikkelingen op de buitendelta tot kustbeheermaatregelen aan de eilandkop. Dit is bijvoorbeeld het geval wanneer een getijdegeul steeds dichter bij de kust komt te liggen en té dicht nadert, of wanneer snelle kustachteruitgang plaatsvindt bij een strandvlakte, die uitsteekt ten opzichte van de doorgaande kustlijn. Beheermaatregelen worden genomen wanneer de achteruitgang van de kust functies in het kustgebied bedreigt. Bij alle Nederlandse Waddeneilanden heeft overigens tenminste één van de twee eilandkoppen de ruimte voor natuurlijke dynamiek. Dit rapport is niet gericht op deze dynamische eilandkoppen, maar op de andere helft, de verdedigde eilandkoppen (tabel 2.1).

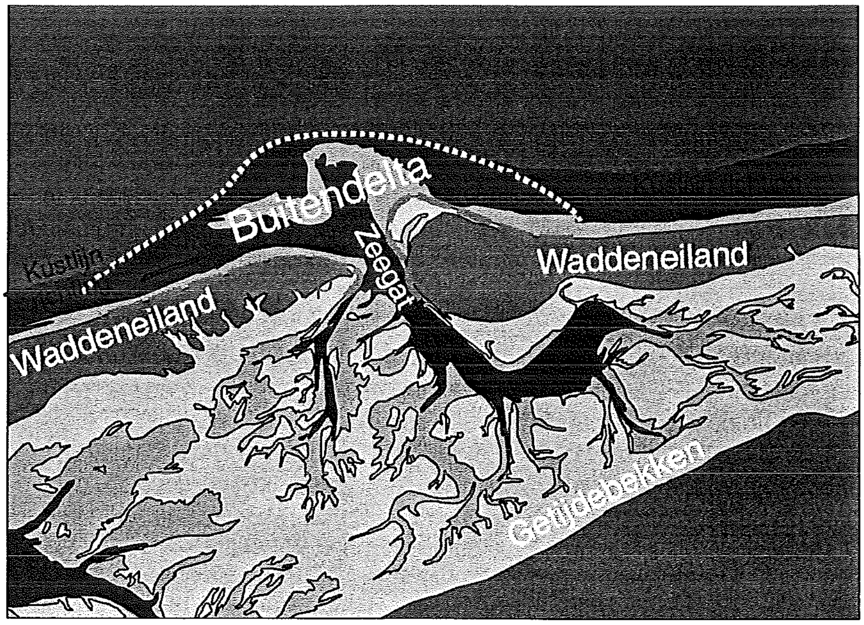
Tabel 2.1:
De eilandkoppen Waddeneilanden en de Kop van Noord-Holland, met de bijbehorende buitendelta's en de wijze waarop het kustbeheer is ingericht.

Buitendelta	Eilandkop	Beheer	Bestaande beheermaatregelen
Zeegat van Texel (Marsdiep)	Kop van Noord-Holland (Den Helder-Julianadorp)	Handhaven kustlijn	Harde kustverdediging (zinkwerk, bestorting & instandhouden hoofden Helderse Zeewering- tot RSP 1)
			Strandsuppleties & instandhouden strandhoofden (ten zuiden van RSP 1,5)
	Zuidwest-Texel	Geen maatregelen	Ruimte voor dynamiek strandvlakte (De Hors tot RSP 9)
		Handhaven kustlijn	Instandhouden strandhoofden, strandsuppleties en geulwandsuppletie(ten noorden RSP 9)
Het Eijerlandse Gat	Noordkop Texel (Eijerland)	Handhaven kustlijn	Strandsuppleties (voorheen tot RSP 31 , tegenwoordig tot RSP 28)
			Harde kustverdediging Eijerlandse Dam en bolwerken Eijerland en Robbengat (zinkwerk en bestorting)
	Zuidwest Vlieland (De Vliehors)	Geen maatregelen	Ruimte voor dynamiek strandvlakte (De Vliehors tot RSP 40)
Het Vlie	Oostkop Vlieland	Handhaven kustlijn	Strandhoofden, onderwatersuppletie, strandsuppleties
			Havenstrand en oostpunt: korte dammen met bestorting & duinvoet verdediging
	Noordwest Terschelling	Geen maatregelen (wél BKL vastgesteld)	Ruimte voor dynamiek strandvlakte, strand en duinen (Noordvaarder, tot RSP 8)
Zeegat van Ameland	Oost Terschelling (De Boschplaat)	Geen maatregelen (wél BKL vastgesteld)	Ruimte voor dynamiek strandvlakte, strand en duinen (Boschplaat, RSP 20 - 26)
		Geen maatregelen	Ruimte voor dynamiek strandvlakte (Boschplaat RSP 26 tot oostpunt)
	West Ameland	Handhaven kustlijn	Lokaal harde kustverdediging: geulwandverdediging (RSP 47.5-49.5) en palenrijen en dammen(RSP 49.5-2.4)
			Strandsuppleties (RSP 2 – 4)
		Geen maatregelen (wél BKL vastgesteld)	Ruimte voor dynamiek strandvlakte (RSP 4 -10)
		Handhaven kustlijn	Onderwatersuppletie, strandsuppleties (RSP 10 en verder)
Pinkegat & Friesche Zeegat	Oost Ameland (De Hon)	Geen maatregelen	Ruimte voor dynamiek strandvlakte (De Hon, vanaf RSP 23)
	West Schiermonnikoog	Handhaven kustlijn	Door landwaartse ligging BKL ruimte voor dynamiek strandvlakte (RSP 1 – 7)
Lauwers	Oost Schiermonnikoog	Geen maatregelen	Ruimte voor dynamiek strandvlakte (De Balg, vanaf RSP 10.4)

1.3 Wat zijn buitendelta's?

De bijna onzichtbare buitendelta's zijn onderdeel van de kuststrook en buitendelta's vormen de verbindende schakel tussen de overige onderdelen van het kustgebied: de Noordzee, de eilandkusten, de Hollandse kust en de achterliggende getijdebekkens en estuaria.
Buitendelta's zijn de zandige ondieptes zeewaarts van de zeegaten die tussen de Waddeneilanden liggen (zie bijvoorbeeld van Veen, 1936, Sha, 1990 en Steijn, 1991). Ten opzichte van de eilandkusten en de onderwateroevers steken de buitendelta's zeewaarts uit. De buitendelta's bestaan uit platen, die al dan niet boven water uitsteken en getijdegeulen (figuur 3.1). Vaak lopen de getijdegeulen uit in ondieptes of drempels. De ligging van de platen en geulen is niet statisch, zowel de plaats als de hoogte variëren sterk in de tijd (Van Veen, 1936, Joustra, 1971, Steijn 1991 en het kader op pagina 24). De grootte van de buitendelta is gerelateerd aan de grootte van het getijdegebied en het getijdevolume dat bij het zeegat hoort (Walton en Adams, 1976, Steijn, 1991 en Louters en Gerritsen, 1994). Hoe groter het getijdeprisma, des te meer zand er in de buitendelta aanwezig is. Bij de Zeeuwse en Zuid-Hollandse zeearmen kan de Voordelta beschouwd worden als een aaneenschakeling van buitendelta's (Sha en Van den Berg, 1993).

Figuur 3.1:
Schematische voorstelling van een buitendelta en de aangrenzende eilandkoppen, met de kenmerkende morfologische elementen



1. Grenzen in de wet en de morfologie.
Naast de veranderlijke fysieke grenzen (paragraaf 3.1 en figuur 3.1), worden de buitendelta's ook begrensd vanuit verschillende wet- en regelgeving. De wet- en regelgeving betreft niet alleen de buitendelta's, maar ook andere gebieden. Hieronder volgt een niet-uitputtend overzicht grenzen die voor de buitendelta's belangrijk zijn.

De PKB-grenzen: De planologische kernbeslissing Waddenzee is geldig voor het gebied dat op de kaart in de pkb is weergegeven. Een deel van de zeegaten valt binnen het pkb-gebied. In de PKB staan beleidsregels voor activiteiten in het Waddengebied, die voor de overheid bindend zijn.
De basiskustlijn: De basiskustlijn is de kustlijn die conform het beleid 'dynamisch handhaven' gehandhaafd moet worden. Hij komt overeen met de ligging van de gemiddelde kustlijn in 1990. [derde kustnota]
De kustlijn: De ligging van de kustlijn komt ongeveer overeen met de laagwaterlijn en vormt de scheiding tussen water en land.
De 1-kilometergrens: Grens van de provinciale en gemeentelijke indeling. Daarmee ook de grens van de invloedssfeer van de streek- en bestemmingsplannen en de daarin verankerde uitwerking van het ruimtelijke ordeningsbeleid. Deze lijn in veelal in coördinaten weergegeven.
Grenzen van de Speciale Beschermingszone i.k.v. de Habitatrichtlijn: De Waddenzee is gedeeltelijk aangemeld als Speciale Beschermingszone i.k.v. de Habitatrichtlijn, deze grens varieert in de ligging t.o.v. de kust.
Grenzen van de Speciale Beschermingszone i.k.v. de Vogelrichtlijn: De waddenzee is aangewezen als speciale beschermingszone in het kader van de vogelrichtlijn. Deze grens komt overeen met de 3-mijlsgrens.
De –5 meter dieptelijn: De grens waaronder schelpenwinning (als grondstof) voor bepaalde gebieden is toegestaan. Daarboven is het winnen van schelpen uitgesloten.
De 1-mijlsgrens: Tot de 1-mijlszone gelden zowel de ecologische als chemische doelstellingen van de Kaderrichtlijn Water. Tussen de 1-mijlszone en de 12-mijlszone alleen de chemische.
De 3-mijlsgrens: Dit is de zone de zeewaartse begrenzing van het Waddengebied, zoals overeengekomen in het trilaterale waddenzeeverdrag (tussen Nederland, Duitsland en Denemarken).
De –20meter dieptelijn: De grens waarvan landwaarts zandwinning niet is toegestaan, met uitzondering van de vaargeulen.
De 12-mijlsgrens: De 12-mijlsgrens bakent het verschil tussen de territoriale zee (het stuk zee waarover de Nederlandse staat evenveel zeggingskracht heeft als over haar land) en de exclusieve economische zone af. In de exclusieve economische zone is de rechtsmacht van Nederland minder verreikend.

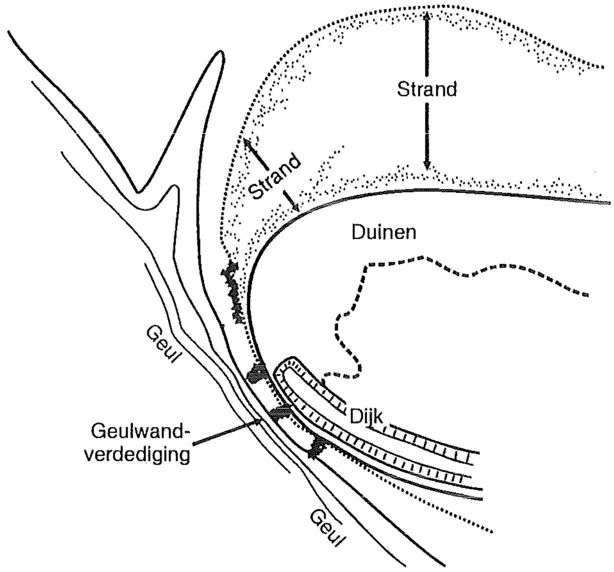
De gebiedsgrenzen uit de verschillende wettelijke kaders lopen soms dwars door elkaar heen. Sommige grenzen hebben wel een duidelijke koppeling met de morfologie, anderen in het geheel niet. De lappendeken van weten en regels die wel of niet geldig zijn op de buitendelta's kan integraal beheer in de weg staan.

2 Kustvorm en kustdynamiek nabij buitendelta's

2.1 De vorm van eilandkoppen

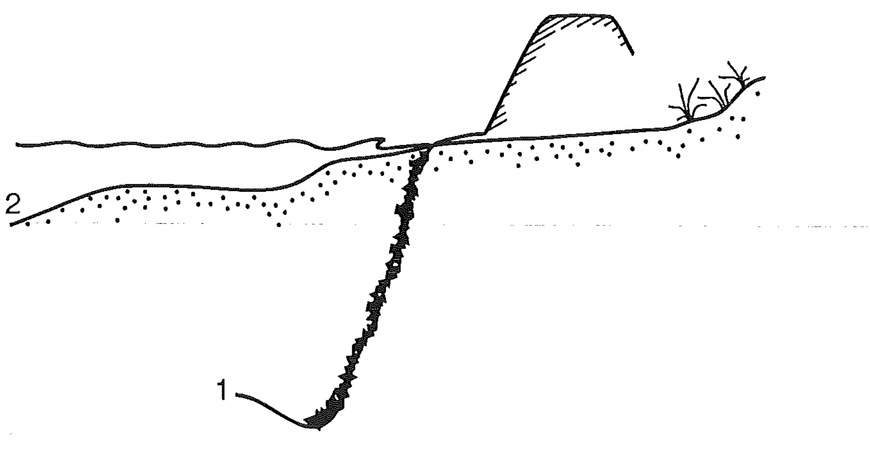
De kust bij de eilandkoppen van de Waddenzee heeft een duidelijk andere vorm dan de kust van het centrale deel van de Waddeneilanden of de Hollandse kust. In een kaartbeeld vallen de sterke kromming van de kustlijn en de variatie in strandbreedte op (figuur 4.1). De strandbreedte varieert van enkele kilometers (er wordt dan meestal van strandvlakte gesproken) tot nul bij dijken met oeververdediging.

Figuur 4.1:
De vorm van de kustlijn bij een eilandkop, met veel variatie in de breedte van het strand.



Bij de kustprofielen nabij buitendelta's is ook de variatie opvallend (figuur 4.2). In sommige profielen gaat de bodem steil naar beneden tot grote diepte, terwijl in andere gevallen het profiel zeer flauw is. Voor de strandbreedte en de steilheid geldt ruwweg: hoe steiler het profiel, des te smaller het strand. De steile en diepe profielen liggen dáár waar getijdegeulen dicht onder de kust lopen. Juist bij de eilandkoppen bereiken de getijdegeulen hun grootste diepte. De variatie van de kustvorm bij een enkele eilandkop kan enorm zijn: gebieden met steile en diepe onderwateroevers liggen vaak niet meer dan enkele kilometers van de gebieden met zeer flauwe onderwateroevers.

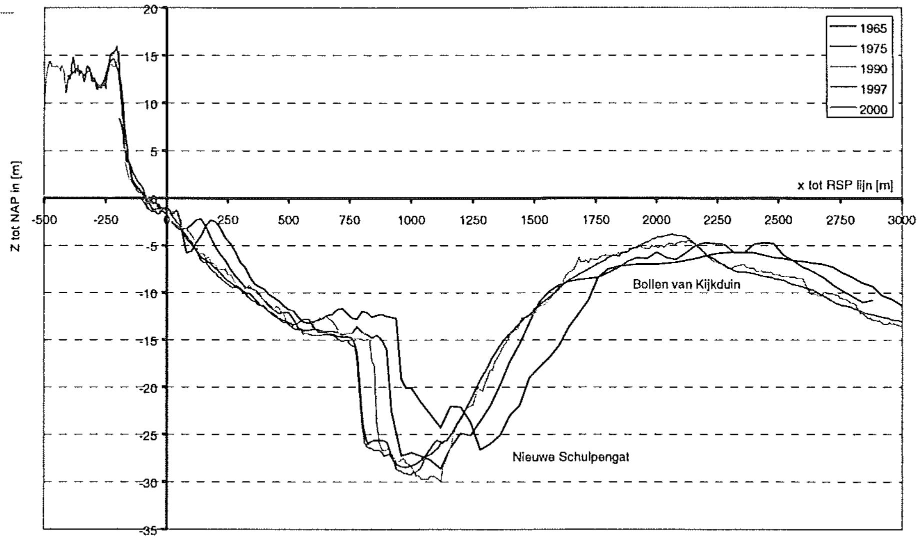
Figuur 4.2:
Verschillende kustprofielen bij een eilandkop, met veel variatie in de steilheid en maximale diepte. 1: Steil profiel en smal strand, kenmerkend voor een getijdegeul de dicht langs een eilandkop loopt. 2: Flauw profiel met breed strand, dat aanwezig is wanneer een grote zandplaat met de eilandkop is verheeld.



2.2 Kustdynamiek bij buitendelta's

De kustvakken bij eilandkoppen hebben een eigen dynamiek, die afwijkt van dynamiek van de Hollandse kust en de centrale delen van de Waddeneilanden (zie bijvoorbeeld IJnsen en van den Boogert, 1993 en Wijnberg, 1995). De tijdsduur waarop uitbouw en achteruitgang afwisselen is bij eilandkoppen meestal langer en de mate van uitbouw en achteruitgang groter.
De kustontwikkeling bij eilandkoppen wordt met name bepaald door de ontwikkelingen van de geulen en platen op de buitendelta's. De verplaatsing van een getijdegeul naar de kust leidt tot grote zandverliezen van de kustzone en (tijdelijke) kustachteruitgang. De verplaatsing van zandplaten van de buitendelta naar de kust en, uiteindelijk, de verheling van die zandplaten met de kust leidt tot zeer grote (tijdelijke) uitbouw. Die uitbouw kan overigens lokaal gepaard gaan met problemen, zoals op Ameland is voorgekomen (Israël en Oost, 2001). De uitbouw of achteruitgang kan permanent zijn, als gevolg van de lange termijn ontwikkeling van zeegat, buitendelta en eiland, maar kan ook tijdelijk zijn, wanneer de ontwikkeling onderdeel is van een zich herhalende morfologische cyclus (zie het kader op pagina 24).

Figuur 4.3:
Kustprofielen dwars op de Kop van Noord-Holland (Jarkus raai 409) in verschillende jaren uit de periode 1965 tot 2000 (Elias en Cleveringa, 2003). In de profielen is steeds het kenmerkende plaat-geul-kust profiel zichtbaar. De drie profielen laten zien dat de geul en plaat steeds dichter tegen de kust 'drukken'.



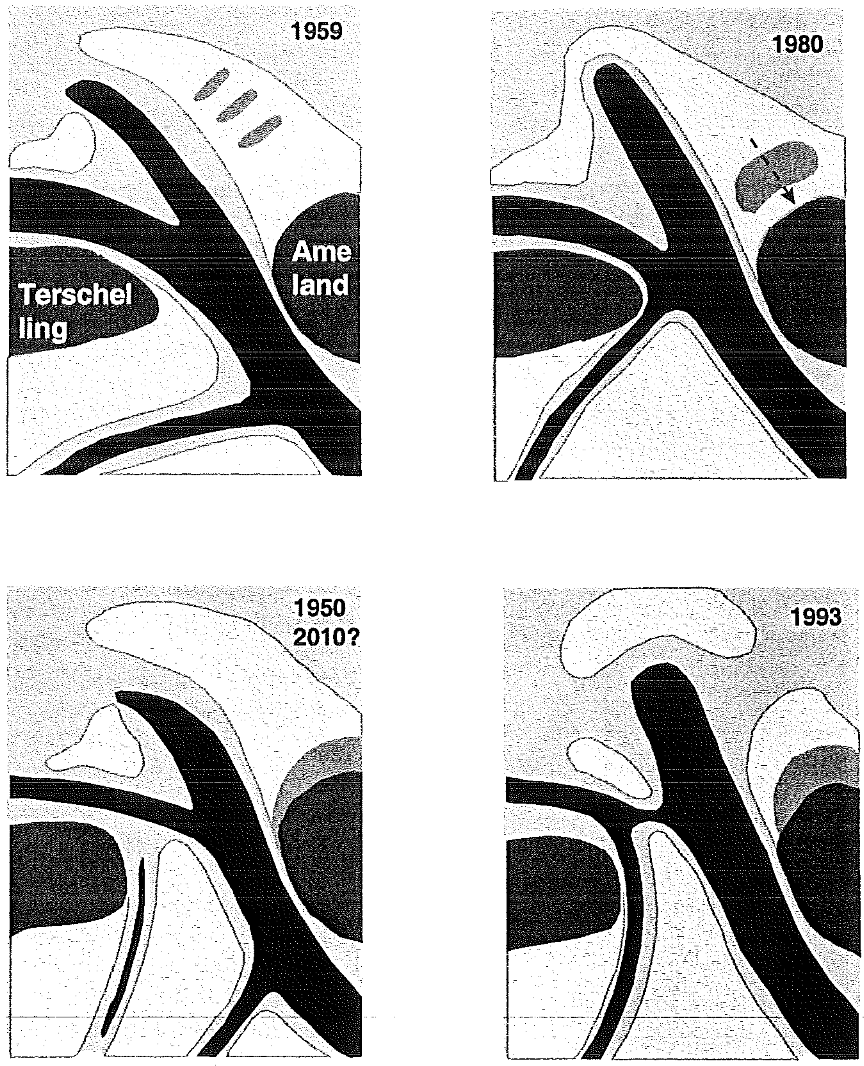
2.3 Plaat – geul – kust

In bijna alle gevallen waarbij op de eilandkop de kust terugschrijdt, gebeurt dit terwijl een getijdegeul landwaarts verplaats. De getijdegeul "duwt" daarbij als het ware de kust naar achteren. In de meeste gevallen bevindt zich zeewaarts van de getijdegeul in een zandplaat, die ook landwaarts verplaatst. De karakteristieke plaat-geul-kust configuratie geeft een kenmerkende kustprofiel, met een steil kustprofiel van strand naar de as van de geul (figuur 4.3).
Vaak wordt het landwaarts verplaatsen van de getijdegeul, al dan of niet onder invloed van de landwaartse verplaatsing van de zandplaat, gezien als de onderliggende oorzaak van de kustachteruitgang. Deze oorzaak-gevolg relatie ligt niet altijd zo eenvoudig, de kustachteruitgang kan ook het gevolg zijn van (een combinatie van) andere factoren, bijvoorbeeld een verandering in de golfgedreven langstransporten (bijvoorbeeld bij de kust van Zuidwest Texel, Cleveringa, 2001).

2.4 De noodzaak tot kustverdediging bij eilandkoppen

Bij alle grote zeegaten van de Waddenzee is in het verleden (van de 18e eeuw) op een of andere wijze een poging gedaan om de kustachteruitgang van een van de eilandkoppen tegen te gaan. De noodzaak hiervoor kwam altijd voort uit directe bedreigingen van de veiligheid, omdat de kustachteruitgang bewoners en bebouwing bedreigde of al verzwolgen had. In alle gevallen was de kustachteruitgang het gevolg van de landwaartse verplaatsing van een getijdegeul (figuur 4.3). In de tweede helft van de 20e eeuw zijn alle eilandkoppen waar sprake was van bedreiging van de veiligheid effectief vastgelegd.
Vanaf 1990 wordt de kustlijn gehandhaafd op toenmalige positie door het uitvoeren van zandsuppleties. Alleen wanneer het niet ander kan, worden harde kustverdedigingsmaatregelen uitgevoerd. Dit betrof in de periode sinds 1990 altijd een kop van een Waddeneiland, namelijk de noordkop van Texel, de oostkop van Vlieland en de westkop van Ameland. Overigens wordt alleen voor de bewoonde Waddeneilanden de kustlijn gehandhaafd en hebben alle Waddeneilanden een eilandkop waar de natuurlijke ontwikkelingen de vrije hand hebben (bij Terschelling betreft het zelfs beide eilandkoppen; tabel 2.1).

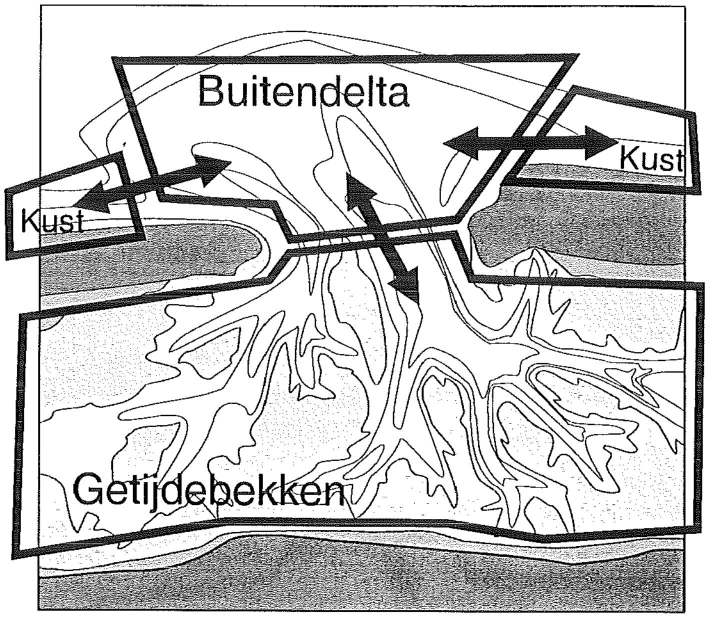
Figuur 4.4:
Morfologische cyclus van de buitendelta van Het Zeegat van Ameland (Israël, 1998).



3 Het zanddelend systeem: buitendelta's, eilandkusten en de Waddenzee

3.1 Wat is het zanddelend systeem?

De kusten van de waddeneilanden en de buitendelta's vormen samen met de Waddenzee een zanddelend systeem (zie voor een toegankelijke inleiding 'Het mysterie van de Wadden; Hoe een getijdesysteem inspeelt op zeespiegelstijging'Louters en Gerritsen, 1994). Het begrip 'zanddelend systeem' is gebaseerd op de kennis over de ontwikkelingen van het gehele kustsysteem op een langere termijn (tien tot duizenden jaren) en de onderliggende processen (Oost e.a., 2000).



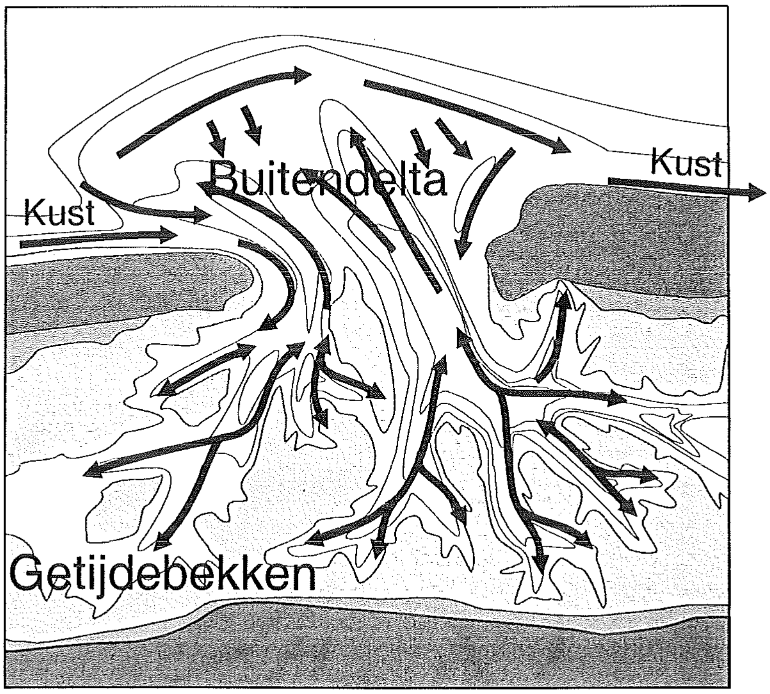
Figuur 5.1:
De belangrijkste componenten van het zanddelend systeem: de buitendelta's, de aangrenzende kusten en de geulen en platen van de Waddenzee.

Voor een schematische omschrijving van het zanddelende systeem beperken we ons tot één zeegat (figuur 5.1). Op grote schaal is het systeem in vier componenten op te delen: het getijdebekken (de Waddenzee zelf), de eilandkusten links en rechts van het zeegat en de buitendelta met de keel van het zeegat. Langs de eilandkusten vindt golfgedreven transport van zand plaats (figuur 5.2). Dit golfgedreven transport bereikt de buitendelta, waar een deel van het zand naar het getijdebekken wordt getransporteerd en een deel langs het zeegat wordt getransporteerd naar de volgende eilandkust ('sand by-passing'). De getijdestroming transporteert het zand door de keel van het zeegat naar het getijdebekken, waarbinnen het transport verder gaat. In het getijdebekken wordt het zand verder getransporteerd door de getijdestroming en, wanneer het zand de platen bereikt heeft, ook onder invloed van golven.

3.2 Het zanddelend systeem in evenwicht

Wanneer er geen veranderingen optreden in het zanddelend systeem is er sprake van een dynamische evenwichtssituatie: er gaat dan net zoveel zand het getijdebekken in, als er weer uit gaat (Louters en Gerritsen, 1994). De evenwichtssituatie manifesteert zich ook in een evenwicht tussen de waterbeweging en de morfologie van het systeem. De 'natte doorsnede' van de geulen in het getijdebekken correspondeert dan met hoeveelheid water die er doorheen stroomt (Eysink, 1979, Gerritsen en de Jong, 1985, Sha, 1990, van der Spek, 1994, Louters en Gerritsen, 1994). Het volume zand in de buitendelta correspondeert met de hoeveelheid water die het zeegat (gemiddeld) in- en uitstroomt (Louters en Gerritsen, 1994). Deze empirische relaties worden gevonden voor alle zeegaten en buitendelta's in de wereld, waarbij lokale bijzonderheden, zoals de verhouding tussen de invloed van golven en getij en de korrelgrootte van het sediment, voor variatie zorgen (Jarret, 1976, Walton en Adams, 1976).

Figuur 5.2:
Een schematische voorstellingen van de zandtransport-processen in het zanddelende systeem van buitendelta's, eilandkusten en Waddenzee.

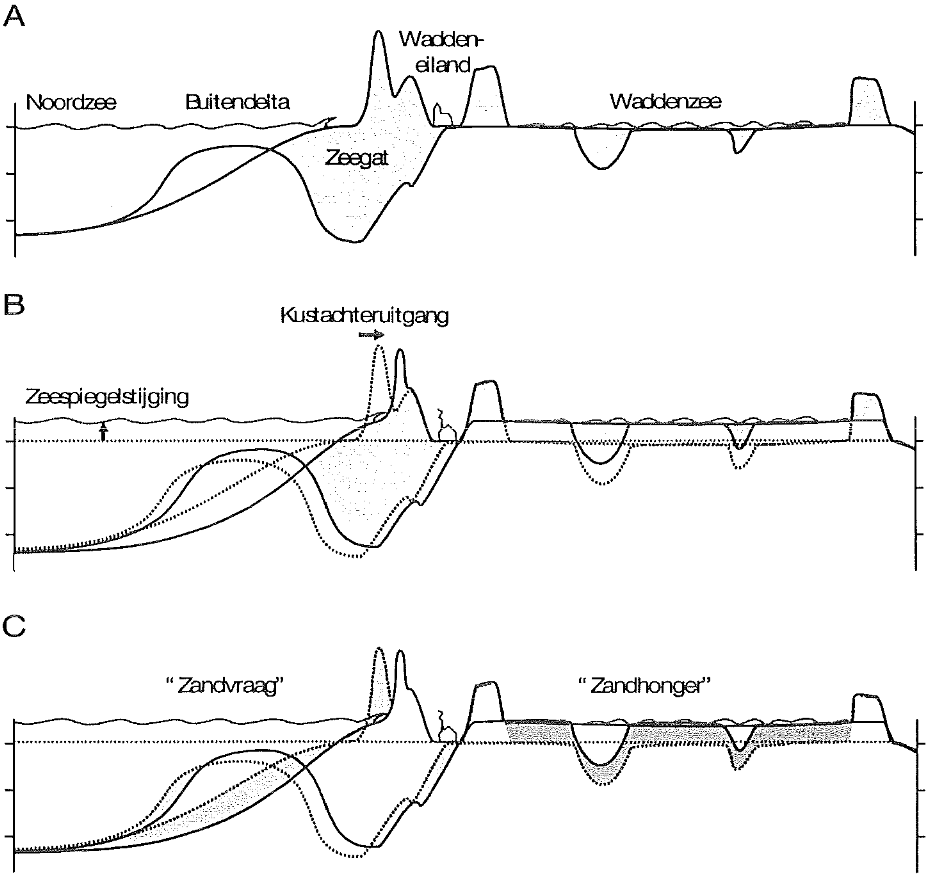


3.3 'Zandhonger' en 'zandvraag': De effecten van zeespiegelstijging op het zanddelend systeem

Terugkerende begrippen bij het ter sprake komen van de Waddenzee en zeespiegelstijging zijn 'zandhonger' en 'zandvraag ' en vragen die daarbij horen zijn 'waarom heeft de Waddenzee zandhonger?' en 'aan wie vraagt de Waddenzee zand?'.

'Zandhonger'
Een stijging van de zeespiegel (tegenwoordig met een snelheid van ongeveer 20 cm per eeuw, in de toekomst mogelijk sneller onder invloed van verschillende broeikaseffecten) resulteert in een kleine verdieping van het getijdebekken. Een iets dieper bekken is beter in staat om sediment, dat steeds in grote hoeveelheden in beweging is binnen het zanddelende systeem, vast te houden. Ook zijn er andere effecten die zorgen voor zandhonger, zoals verlanding van de Waddenzee en bodemdaling door zand-, schelpen-, zout- of gaswinning. Elk proces dat de evenwichten in de Waddenzee verstoort, zodat zand van buiten de Waddenzee nodig is om de evenwichten te herstellen, draagt bij tot de zandhonger van de Waddenzee. De bruto zandtransporten binnen het bekken en van en naar het bekken zijn enkele tientallen malen hoger dan het netto zandtransport (Oost e.a., 2000).

De relatieve geringe toename in het netto transport kan dan ook makkelijke tot stand komen. Het toegenomen netto zandtransport is de 'zandhonger'. Het is een combinatie van verschillende fysische effecten die de 'zandhonger' veroorzaakt, een inleiding hierin kan bijvoorbeeld gevonden worden in de 'Integrale Bodemdalings Studie' (Oost e.a., 2000).



Figuur 5.3:
'Zandvraag' en 'zandhonger' onder invloed van de stijgende zeespiegel. A: Doorsnede van de kust, Waddeneilanden en Waddenzee, inclusief buitendelta en zeegat. B: De Waddenzee groeit mee met de stijgende zeespiegel. Het zand voor de Waddenzee wordt ontrokken aan de kustzone en de buitendelta, waardoor deze terugschrijden. C: De 'zandhonger' van de Waddenzee en de 'zandvraag' aan de kustzone en buitendelta.

'Zandvraag'

Het zand om de 'zandhonger te stillen' komt uit de andere onderdelen van het zanddelend systeem en dit is de 'zandvraag' van het getijdebekken. De onderdelen die het zand leveren zijn de buitendelta en de kusten die aan het zeegat grenzen (figuur 5.3)[1]. De buitendelta treedt waarschijnlijk op als zandbron, bij de algehele achteruitgang van de kust, én als doorgeefluik voor zand van de kust naar het getijdebekken.

Samenvattend: De 'Waddenzee heeft zandhonger' door de stijging van de zeespiegel en andere, deels antropogene, oorzaken.
De 'Waddenzee vraagt zand aan de andere onderdelen van het zanddelende systeem', dit zand komt uit de Noordzeekustzone.

[1] De dimensies van de buitendelta blijken in evenwicht te zijn met de grootte van het getijdevolume van een zeegat. Dit evenwicht verandert niet persé bij een stijging van de zeespiegel (alleen bij een zeer snelle stijging zal de grootte van het getijdevolume en daarmee het zandvolume van de buitendelta toenemen). Bij een niet-veranderend getijdevolume kan de buitendelta zand leveren en toch haar dimensies behouden, als zij samen met de aangrenzende kust landwaarts terugtrekt.

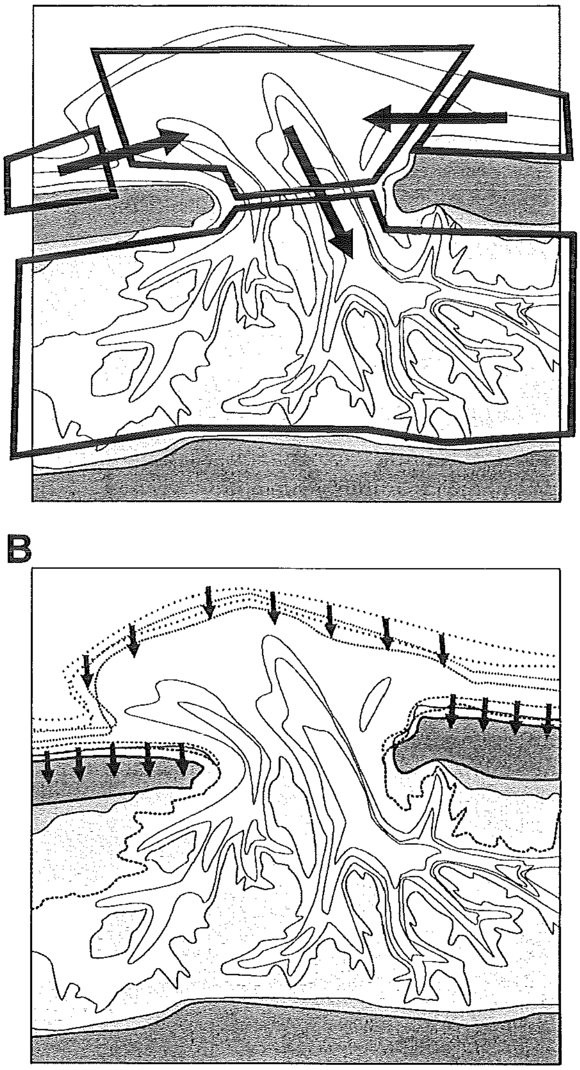
3.4 De rol van buitendelta's in het zanddelend systeem

Over de rol van de buitendelta in het zanddelende systeem is het laatste woord nog niet gesproken. Door Louters en Gerritsen (1994) worden de buitendelta's omschreven als tijdelijke zandbuffers voor de andere delen van het zanddelende systeem. De buitendelta's vangen ook de grote klappen op (Oost e.a., 2000). Op grond van nieuwe morfologische analyses (Walburg, 2001) en modelberekeningen (Kragtwijk, 2001) is geconstateerd dat veranderingen in het getijdebekken direct doorwerken in de kustvakken die aan buitendelta en het zeegat grenzen én in de buitendelta (figuur 5.4). De buitendelta lijkt daarmee vooral een doorgeefluik van zand tussen kust en Waddenzee.

Figuur 5.4:
Het resultaat van veranderingen in het zanddelende systeem, bijvoorbeeld door het stijgen van de zeespiegel.
A: de buitendelta als doorgeefluik tussen Waddenzee en kust.
B: De resulterende algehele achteruitgang van de kust en de buitendelta.

A

B



3.5 Kustbeheer en het zanddelende systeem

Het effect van de doorgaande stijging van de zeespiegelstijging is een continue zandvraag aan de Noordzeekustzone, die resulteert in een achteruitgang van de Waddenkust en de Kop van Noord-Holland. Ook de buitendelta's van de Waddenzee Schrijden langzaam landwaarts onder invloed van de stijgende zeespiegel (figuur 5.4 b). De kustlijn wordt vanaf 1990 op zijn plaats gehouden door het uitvoeren van zandsuppleties (op het strand en onderwater). Met deze suppleties wordt slecht een deel van de zandvraag gevoed, omdat ook zandverlies plaatsvindt waar de kustlijn niet wordt gehandhaafd (omdat geen Basisksuslijn is vastgesteld) en bij de buitendelta's. Om aan de zandvraag als gevolg van de zeespiegelstijging te voldoen moet derhalve meer zand worden gesuppleerd, dan nodig is voor het handhaven van de kustlijn (Mulder, 2000).

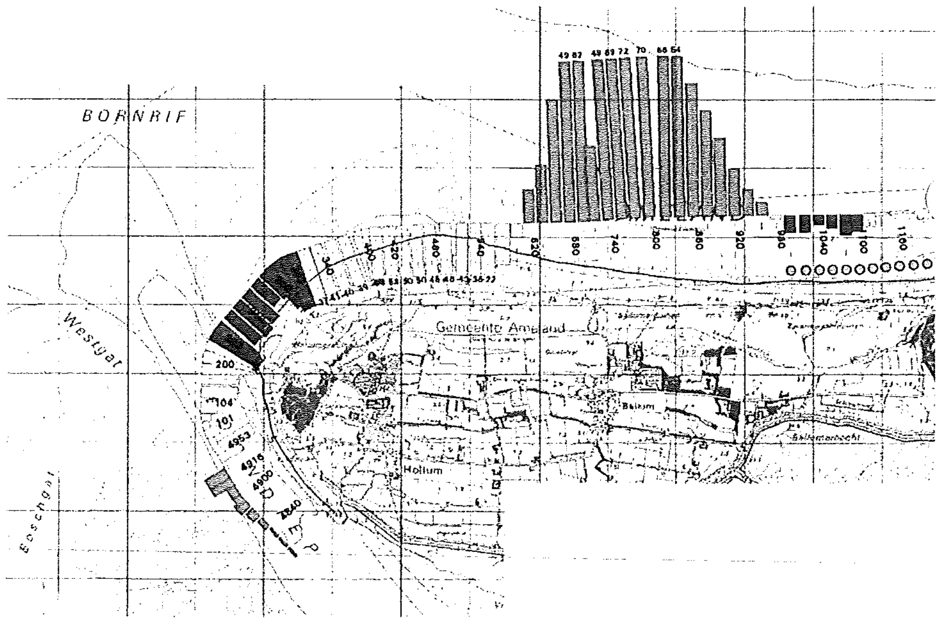
4 De twee doelstellingen van het kustbeleid

4.1 Het kustbeleid: Duurzaam handhaven van veiligheid én functies en waarden

Het Nederlandse kustbeleid is gericht op het duurzaam handhaven van de veiligheid en duurzaam behoud van de functies en waarden in het duingebied (1^e, 2^e en 3^e kustnota, Ministerie van Verkeer en Waterstaat, 1990,1996 en 2000). Duurzaam betekent in dit geval met behoud van de karakteristieken van de zandige kust. De beheermaatregelen moeten zó worden ingericht dat functies en waarden kunnen worden behouden. Voortkomend uit deze wens zijn de twee operationele doelstellingen voor beheermaatregelen geformuleerd. De operationele doelstellingen sluiten direct aan op de dynamiek van de kust én op het zanddelende systeem van de buitendelta's, eilandkusten en de Waddenzee.

4.2 De korte termijn doelstelling: Een stabiele kustlijn

In de Wet op de Waterkering (Ministerie van Verkeer en Waterstaat, 1996) is vastgelegd dat de kustlijn wordt gehandhaafd op zijn positie. In de praktijk wordt uitgegaan van de kustlijnligging van 1990, zoals bepaald met de MKL/BKL methode (Hillen e.a.,1991). De *korte termijn* operationele doelstelling van het beheer van de kust is het handhaven van de kustlijn. Het begrip 'korte termijn' betreft de reikwijdte van de beheermaatregelen en niet de reikwijdte van het beleidsdoel. De kustlijnligging is direct gekoppeld aan de dynamiek, die bij de koppen van de eilanden met name wordt bepaald door de buitendelta's (Hoofdstuk 4). Bij de keuze voor het type beheermaatregel om deze operationele doelstelling te bereiken speelt de vorm van de kust een belangrijke rol, hierop wordt in het volgende hoofdstuk dieper ingegaan



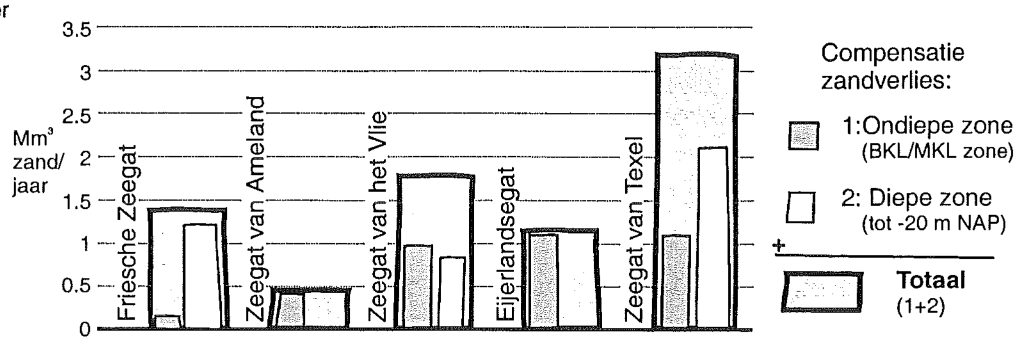
Figuur 6.1:
De kustlijnkaart van Ameland van het jaar 2003 met momentane kustlijn (MKL) en basiskustlijn (BKL), het instrument om de stabiliteit van de kustlijn vast te stellen (Snijders en Uit den Bogaard, 2003).

Bij het vaststellen van de kustlijnligging wordt uitgegaan van de trend over een periode van tien jaar in het zandvolume van de ondiepe kustzone. De impliciete tijdschaal van de methode ligt daarmee ook rond de tien jaar (Cleveringa e.a., 2003). Jaarlijks wordt beoordeeld of maatregelen noodzakelijk zijn voor het handhaven van de kustlijn (figuur 6.1). Voor alle duidelijkheid, ook over 50 jaar moet de kustlijn nog op zijn huidige positie liggen (uitgaande van het huidige beleid).

4.3 De lange termijn doelstelling: De zandvoorraad op peil

In de 3e kustnota (Ministerie van Verkeer en Waterstaat, 2000) is vastgesteld dat de zandverliezen van de gehele Nederlandse kust worden gecompenseerd. De gehele Nederlandse kust begint bij de dieptelijn van –20 m NAP op de Noordzee. De zandverliezen als gevolg van de 'zandhonger' van de Waddenzee en de Westerschelde zijn hierbij inbegrepen (figuur 6.2: zie Mulder, 2000 voor een uitwerking van de getallen). Vooralsnog wordt uitgegaan van een scenario met de huidige snelheid van zeespiegelstijging, van circa 20 cm/eeuw, bij een versnelde stijging van de zeespiegel nemen de zandverliezen toe. De *lange termijn* doelstelling voor het beheer van de kustlijn is daarmee het compenseren van de zandverliezen van de gehele kust, oftewel het handhaven van de zandvoorraad van het kustfundament.
De zandverliezen van de kust van de kop van Noord-Holland tot de Monding van de Eems worden voor een belangrijk deel bepaald door de 'zandhonger' van de Waddenzee (Hoofdstuk 5). Door de zandhoger te compenseren kan de kustachteruitgang, die het gevolg is van de zandvraag, worden beperkt. Hiervoor is meer zand nodig dan gemiddeld sinds de start (in 1990) van het beleid van 'dynamisch handhaven' met zandsuppleties, in dit gebied wordt aangebracht.

Figuur 6.2:
Schatting van de suppletiebehoefte per deelsystemen van de Waddenkust, waarbij onderscheid wordt gemaakt in de suppletiebehoefte gericht op de kustlijnhandhaving (ondiep) en de resterende suppletie behoefte ter compensatie van de negatieve zandbalans (diep). Samen (diep + ondiep) wordt met deze suppletiehoeveelheden de zandvoorraad op peil gehouden (naar Mulder, 2000).



Bij het vaststellen van de zandverliezen is gekeken naar een periode van 30 jaar (1965-1995), met een vooruitblik van 50 jaar. De impliciete tijdschaal waarop naar de zandverliezen van de gehele kust wordt gekeken is 50 jaar. De bijbehorende ruimteschaal beslaat de gehele kust van de –20 m tot en met Westerschelde en Waddenzee. De zandvoorraad van het kustfundament hoeft niet vandaag of morgen op peil te zijn, hiervoor is een periode van 50 jaar beschikbaar. De natuurlijke zandtransportprocessen (figuur 5.2) hebben alle tijd om het zand te verdelen, zodat het terechtkomt dáár waar het nodig is.

4.4 Van doelstellingen naar beheermaatregel volgens de drietrapsstrategie

Het zand van het zanddelende systeem is constant in beweging en deze dynamiek is een kenmerkend onderdeel van de Nederlandse kust. Uitgaande van deze dynamiek én van de twee operationele doelstellingen is een drietrapsstrategie benoemd voor kustbeheermaatregelen (uit de Beleidsagenda voor de kust, Ministerie van Verkeer en Waterstaat, 2002):

1: Zand zoveel mogelijk laten bewegen: Ruimte voor dynamiek.

2: Zandtekorten aanvullen waar functies bedreigd worden, door zandsuppleties
en het aanleggen van zandbuffers.

3: Zand vasthouden door het aanleggen van harde constructies.

Wanneer moet worden ingegrepen zijn zandsuppleties en zandbuffers de aangewezen maatregelen. Harde ingrepen zijn een laatste middel, vanwege de hoge aanlegkosten, negatieve omgevingseffecten en het feit dat harde kustverdedigingen geen bijdrage leveren aan de zandvoorraad van de Nederlandse kust. Alleen wanneer zandsuppleties geen oplossing bieden voor het erosieprobleem worden harde maatregelen overwogen.

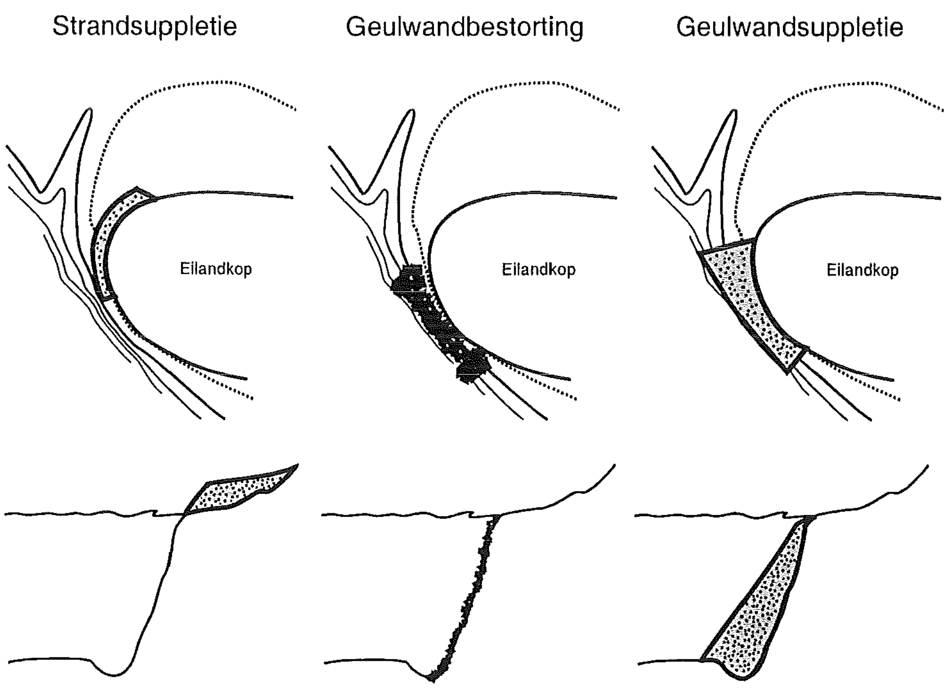
5 Beheermaatregelen voor eilandkoppen op rij

5.1 Kustbeheer: Strandsuppleties, zandbuffers, of harde kustverdedigingsmaatregelen?

Kustbeheermaatregelen worden uitgevoerd met twee operationele doelen:
1. Het tegengaan van lokale kustachteruitgang.
2. Het aanvullen van de zandvoorraad van het gehele zanddelende systeem.

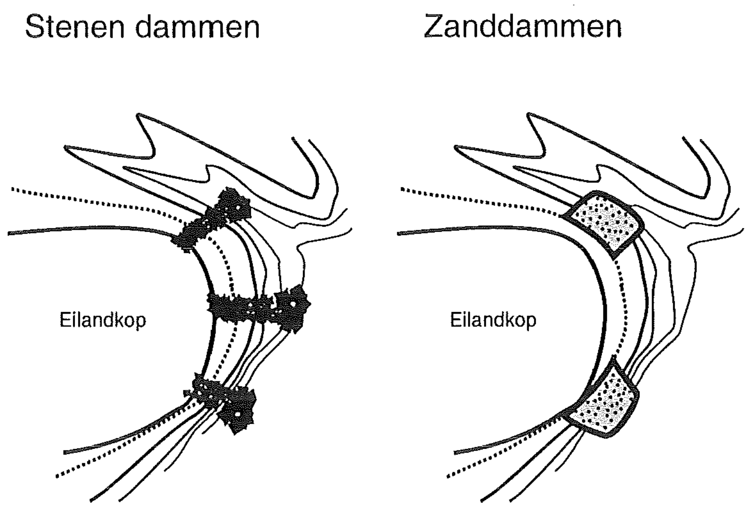
Uitgangspunt voor het inrichten van de beheermaatregelen is daarbij de drietrapsstrategie van - vrij laten bewegen - zandbuffers aanleggen - zand vasthouden.
In eerste instantie komt de noodzaak tot het nemen van beheermaatregelen voort uit de achteruitgang van de kustlijn. De beheermaatregelen zijn dan ook gericht op de eerste doelstelling, het stabiliseren van de kustlijn. Om ook aan de tweede doelstelling te voldoen moet meer zand in het kustsysteem worden gebracht dan in de periode 1990-2000. Het zand dat nodig is om de zandvoorraad op peil te houden biedt tal van nieuwe mogelijkheden voor beheermaatregelen bij eilandkoppen. Ook voor de eilandkoppen ontstaat de mogelijkheid om van statische handhaving, met strandsuppleties en harde kustverdedigingswerken, over te gaan naar dynamische handhaving, met ruimte voor de natuurlijke verplaatsingen van geulen en platen.

Figuur: 7.1:
Drie typen kustlangse beheermaatregelen voor de kustverdediging van eilandkoppen varianten: strandsuppleties harde ingrepen en zandbuffers.



Alternatieve maatregelen (figuur 7.1 en 7.2) worden in dit hoofdstuk vergeleken in het licht van de operationele doelen en de drietrapsstrategie. Het strategische doel van het kustbeleid is het behoud van de functies en waarden van het kustgebied. Om dit doel te bereiken moeten ook de effecten op de verschillende functies en waarden vergeleken worden. De basis voor deze vergelijking is een afwegingskader, waarvoor in Hoofdstuk 9 een voorzet wordt gegeven.

Figuur: 7.2:
Een harde kustverdediging en een zandbuffer dwars op de kust.



5.2 Strandsuppleties

Zandsuppleties zijn sinds 1990 dé aangewezen methode voor het beheer van de kust. Waar mogelijk worden zandsuppleties tegenwoordig op de vooroever aangebracht, zowel vanwege het kostenaspect (goedkoper) als vanwege de minimale hinder op het strand. Op de koppen van de Waddeneilanden en bij de kop van Noord-Holland worden nabij getijdegeulen vrijwel altijd strandsuppleties uitgevoerd (figuur 7.3).

Figuur 7.3:
Suppletie op het strand van Texel.

Strandsuppleties zijn een effectieve manier om het zandvolume in de ondiepe kustzone (de BKL/MKL zone) op peil te houden, maar juist bij situaties met een zeer steile vooroever (door de aanwezigheid van een getijdegeul) kan erosiebestrijding met strandsuppleties niet toereikend zijn (Roelse, 2002). Suppleties zijn niet eenmalig, herhaling blijft op termijn noodzakelijk, omdat (vrijwel altijd) sprake is van doorgaande kustachteruitgang en de uitgevoerde suppleties niets veranderen aan de erosieprocessen die leidden tot de achteruitgang.

Effectiviteit in termen van stabiliteit van de kustlijn
Strandsuppleties voeden de ondiepe kustzone en de invloed op het diepere deel van het kustprofiel is beperkt. Wanneer (een deel van) de kustachteruitgang wordt veroorzaakt door de landwaartse verplaatsing van een geul compenseren de strandsuppleties slechts een deel van deze ontwikkeling. Terwijl de geul verder landwaarts opschuift, blijft het strand netjes op peil. Het Oostgat bij de zuidwestkust van Walcheren is een duidelijk voorbeeld van deze ontwikkeling (Israël, 2001). Dit kan een tijdlang goed gaan, maar op een gegeven ogenblik moet ook de geulverplaatsing tot stilstand worden gebracht. Voor het tegengaan van de landwaartse geulverplaatsing moeten andere methoden dan strandsuppleties in stelling worden gebracht.

Effectiviteit in termen van de zandvoorraad van de Nederlandse kust
Wanneer Noordzeezand (zand van buiten het zanddelend systeem) gebruikt wordt voor suppleties van de kustzone, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem. De zandvolumes per suppletie zijn relatief laag, maar wanneer de suppleties vaak herhaald worden kan toch sprake zijn van een substantiële aanvulling van de sedimentvoorraad van de Nederlandse kust.

5.3 Harde verdedigingen (bestorting, strandhoofden, korte en lange dammen)

Harde verdedigingen zijn de laatste trede van de drietrapsstrategie en moeten zoveel mogelijk worden vermeden. Alleen wanneer zandsuppleties geen oplossing bieden voor het erosieprobleem worden harde maatregelen overwogen.
Bij eilandkoppen met een landwaarts verplaatsende getijdegeul dicht onder de kust worden van oudsher harde (stenen of betonnen) constructies toegepast, in de vorm steenbestortingen met of zonder stroomhoofden. De Helderse Zeewering bij de Kop van Noord-Holland is het oudste voorbeeld uit het Wadddengebied van harde verdediging, maar ook de noordpunt van Texel (Bolwerken Eijerland en Robbegat), de noordoostpunt van Vlieland en de westkop van Ameland zijn voorzien van steenbestorting. De Eijerlandse Dam op de noordpunt van Texel is een ander type harde verdediging, de beoogde werking van deze lange dam is het veranderen van de evenwichtsligging van de noordwestkust van Texel.

Effectiviteit in termen van stabiliteit van de kustlijn
De effectiviteit van de harde kustverdedigingen voor het tegengaan van kusterosie is afhankelijk van het ontwerp in relatie tot de oorzaak van de erosie. Alleen wanneer de maatregel goed aansluit op het erosieprobleem kan een harde kustverdediging effectief zijn (Technische Advies Commissie voor de Waterkeringen,1995). Wanneer de kusterosie op lange termijn daadwerkelijk afneemt, hoeven ter plaatse minder zandsuppleties uitgevoerd te worden. Wanneer de maatregel niet goed aansluit op de erosieproblemen en daarom deels of niet effectief is, zijn aanvullende (harde of zachte) kustbeheermaatregelen noodzakelijk.

Effectiviteit in termen van de zandvoorraad van de Nederlandse kust
Harde kustverdedigingswerken leveren geen aanvulling van de zandvoorraad van de Nederlandse kust. De ingrepen kunnen een -tijdelijke- onderbreking opleveren van de zandtransporten tussen de onderdelen van het zanddelende systeem.

Figuur 7.4:
De Eijerlandse Dam aan de noordkop van Texel. Op de foto is ook het Bolwerk Eijerland te zien: een kustverdedigingswerk dat in 1955 is aangelegd om het terugschrijden van de noordkop van Texel tegen te gaan.

Harde kustverdedigingswerken: De kustverdediging van de noordkust van Texel.

De noordpunt van Texel is ingepakt met harde kustverdedigingswerken. Naast de bolwerken Eijerland en Robbegat ligt sinds 1995 de Eijerlandse dam (figuur 7.4). Minder zichtbaar is het zinkwerk dat de tenen van bolwerken en dam beschermt tegen uitschuring door getijdestromingen. Overigens dienen de bolwerken en de dam verschillende doelen: de bolwerken moeten het eiland beschermen tegen het naar het zuiden uitbreidende zeegat (Rijkswaterstaat, 1955), terwijl de dam de kusterosie langs het strand van Texel moet tegengaan (Rakhorst en Pwa, 1993).
De noordpunt van Texel grenst aan het Eijerlandse Gat, het zeegat tussen Texel en Vlieland. Sinds de tweede helft van de 18e eeuw wordt de noordkust van Texel langzaam geërodeerd, waarbij de ontwikkeling van de geulen op de buitendelta, in het zeegat en in het getijdebekken een belangrijke rol spelen. De vuurtoren van Eijerland stond rond 1850 nog comfortabel in een duincomplex, maar tegenwoordig staat hij op een steenworpafstand van de zee. In de jaren '50 van de 20ste eeuw zijn de bolwerken aangelegd, om verdere erosie en verlies van de vuurtoren en andere bebouwing tegen te gaan.
De Eijerlandse dam is aangelegd om de erosie langs de Texelse kust te verminderen (door het beperken van de kustboog van Texel neemt de gradiënt in het langstransport af). De grote hoeveelheid zand die aan de noordzijde van de dam ligt is een bijkomend (onverwacht) voordeel die een extra bescherming oplevert voor het Bolwerk Eijerland.

5.4 Zandbuffers (geulwandsuppleties, zanddammen)

Oplossingen met zand om de landwaartse verplaatsing van getijdegeulen af te remmen, of ruimte te creëren tussen geul en kust zijn slechts zeer beperkt uitgevoerd. Oplossingen met zand kunnen de vorm krijgen van suppleties op de wand van de getijdegeul of zanddammen dwars op de kust (figuur 7.1 en 7.2, zie ook Roelse, 2002, voor andere typen ingrepen). Oplossingen met zand beperken de natuurlijke ontwikkelingen van de kust veel minder dan harde ingrepen, de uitwisseling van zand met andere delen van de kust blijft mogelijk. Verder levert het zand een bijdrage aan het zandbudget van het kustfundament.

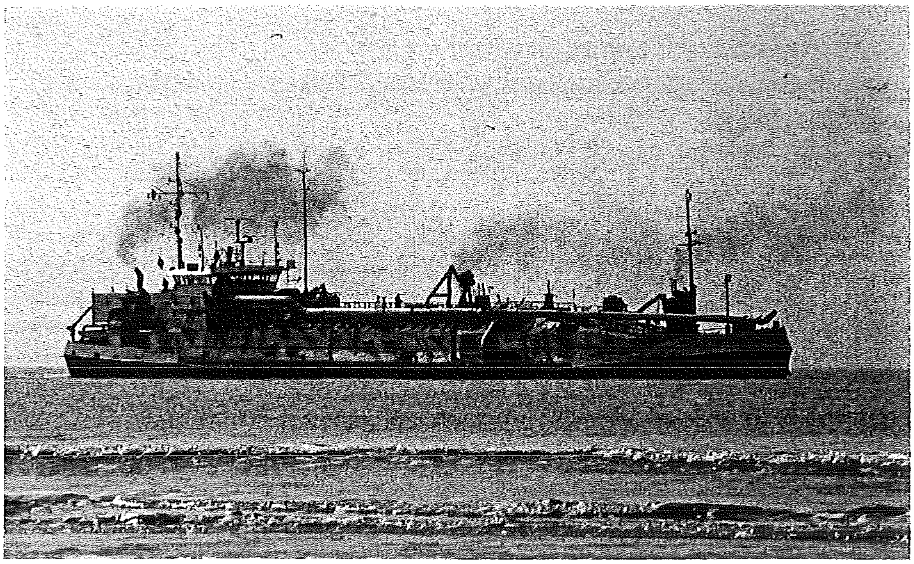
Kustverdedigingsmaatregelen met zand, anders dan de reguliere strand- en onderwatersuppleties, worden niet met name genoemd in de kustnota's en de beleidsagenda voor de kust (Ministerie voor Verkeer en Waterstaat, 1991, 1996, 2000 en 2001). Uitgaande van de drietrapsstrategie en de doelstelling om de zandvoorraad van de gehele Nederlandse kust te handhaven kan gesteld worden dat zandbuffers bij uitstek passen in het huidige kustbeleid. Ingrepen met zand hebben, net als de gebruikelijke strand- en vooroeversuppleties, een beperkte levensduur. Herhaling van de ingreep blijft derhalve noodzakelijk, omdat (vrijwel altijd) sprake is van doorgaande kustachteruitgang. Wanneer de erosieprocessen cyclisch zijn (zie kader op pagina 24), kunnen de ingrepen met zand worden gebruikt om de periode van erosie te overbruggen. Herhaling van de suppleties is dan alleen gekoppeld aan de periode van erosie.

Effectiviteit in termen van stabiliteit van de kustlijn
Indien voldoende zand wordt aangebracht in de vorm van een geulwandsuppletie zal een getijdegeul tijdelijk uit de kust worden gehouden. Voor het schatten van de benodigde hoeveelheid zand voor een geulwandsuppletie volstaat het vergelijken van het bestaande profiel met het gewenste profiel, per raai over de lengte van het gebied. Het benodigde volume zal in alle gevallen (veel) groter zijn dan een strandsuppletie. De ervaring met geulwandsuppleties is beperkt (zie kader op pagina 41), maar bieden voldoende aanknopingspunten om een schatting te maken van de levensduur.

Zanddammen dwars op de kust dienen om een getijdegeul dicht onder de kust uit de kust te drukken. De zanddammen dwingen de getijdenstromen tot het volgen van een andere route en beperken daarmee de sturende kracht achter de bedreigende ontwikkelingen van de geul. Deze strategie lijkt bruikbaar op locaties waar de kustachteruitgang is gekoppeld aan een van de kleine (vloed)geulen. Zanddammen zijn niet toepasbaar waar de ontwikkelingen van de hoofdgeul in het zeegat bepalend zijn voor de kust. De voorspelbaarheid van de levensduur is vooralsnog beperkt.

Effectiviteit in termen van de zandvoorraad van de Nederlandse kust
Wanneer Noordzeezand (zand van buiten het zanddelend systeem) gebruikt wordt voor de aanleg van zandbuffers in de kustzone, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem. De prijs per m^3 is laag en de aangebrachte volumes zijn groot, waarmee de aanleg van zandbuffers een zeer effectieve wijze is om het kustfundament op peil te houden.

Figuur 7.5:
Sleephopper 'Hein' lost zijn lading zand dicht onder de kust van zuidwest Texel, op de geulwand van het Molengat



De geulwand-suppletie in het Molengat aan de zuidwestkust van Texel.

In gebieden met een sterke (getijde)stroming onder de kust, zoals bij veel eilandkoppen het geval is, worden normaal gesproken geen onderwatersuppleties uitgevoerd. Verondersteld wordt dat het zand snel verdwijnt (Nourtec, 1997) en onderwatersuppleties weinig of niet effectief zijn (2e Kustnota, Ministerie van Verkeer en Waterstaat, 1995). Deze veronderstellingen zijn echter niet op praktijkvoorbeelden gebaseerd.

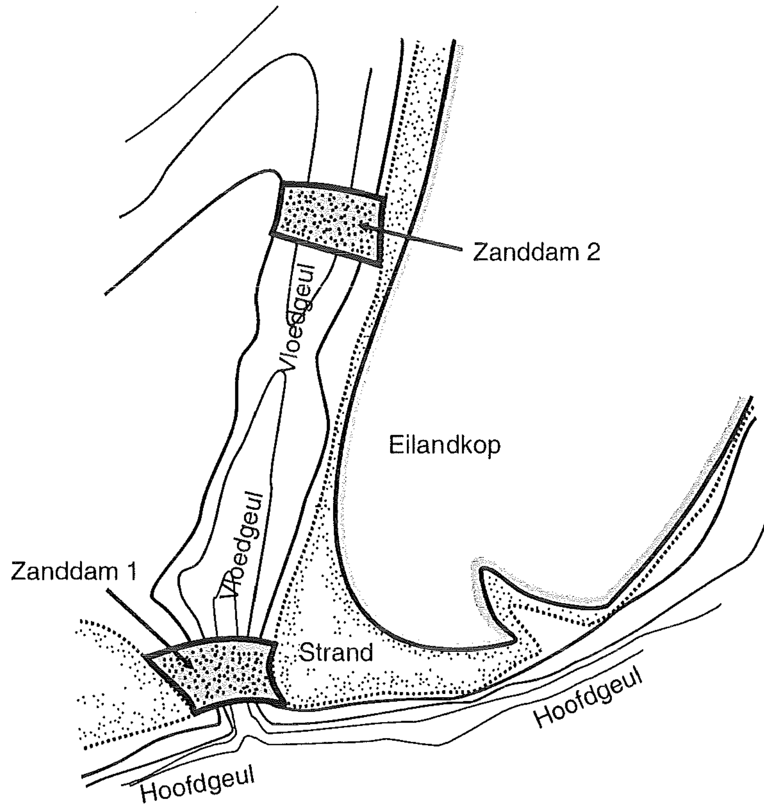
In 2001 is een kleine onderwatersuppletie aangebracht op de geulwand van het Oostgat van de Westerschelde (Koomans en Oosterhoff, 2002 en referenties in dit rapport). Het gestorte zand bleef goed liggen op de steile geulwand (met een helling van 1:7) en verassend genoeg bleek ook na 1 jaar nog veel van het zand op de geulwand aanwezig.

In 2003 is in het Molengat bij Zuidwest Texel een onderwatersuppletie aangebracht op de overgang van vooroever naar de getijdegeul Molengat (figuur 7.5). Deze suppletie moet de achteruitgang van de kustlijn tegengaan. Vragen die bij deze onderwatersuppletie spelen zijn: 1) komt het gesuppleerde zand ten goede aan de kustlijn (meer specifiek: aan de BKL/MKL zone)? en 2) hoe snel verdwijnt het gesuppleerde zand? Om deze vragen te beantwoorden wordt de ontwikkeling van de geulwandsuppletie Molengat en het aangrenzende strand regelmatig gemeten en geëvalueerd.
De ervaringen bij het Molengat en het Oostgat kunnen aanleiding zijn om in de toekomst ook geulwandsuppleties op andere locaties in te zetten.

5.5 Zandsluitingen

Zand kan gebruikt worden om getijdegeulen volledig af te sluiten (figuur 7.6), om de structurele erosie door een geul dicht onder de kust te stoppen. Het getijdewater zal zijn weg vinden door een andere bestaande of nieuwe geul op de buitendelta. Deze oplossing kan alleen soelaas bieden in gebieden waar de kustlijnontwikkeling achteruitgang wordt bepaald door een relatief kleine kustnabije (vloed)geul. Met nadruk wordt vermeld dat dit type zandsluitingen niet is bedoeld om het volledige zeegat af te sluiten.

Figuur 7.6
Zandsluiting, in de vorm van twee zanddammen, die de stroming door de getijdegeul volledig beëindigen.



Zandsluitingen van geulen onder de kust zijn omvangrijke varianten op de eerder genoemde zandbuffers. De zandsluiting vormt met de aanliggende zandplaat een grote zandbult voor de kust. In veel gevallen betekent een zandsluiting een versnelling van de natuurlijke trend, waarbij zandplaten met de kust verhelen. Ook bij zandsluitingen is ruimte voor de natuurlijke ontwikkelingen van de kust, de zandbult zal onder invloed van de zandtransporten van vorm veranderen en in omvang afnemen. Het zand levert hiermee een bijdrage aan het zandbudget van het kustfundament.

Effectiviteit in termen van stabiliteit van de kustlijn
Zandsluitingen dwingen de getijdenstromen tot het volgen van een andere route en stoppen daarmee de sturende kracht achter de bedreigende ontwikkelingen van de geul. Daarbij wordt door de zandsluiting, samen met de ondiepte zeewaarts van de getijdegeul, een grote zandbult voor de kust gecreëerd. Hiermee wordt een situatie geschapen die vergelijkbaar is met natuurlijke verhelingen van zandplaten met de kust, zoals bijvoorbeeld bij Ameland en Schiermonnikoog zijn opgetreden (Israel en Oost, 2001). Zandsluitingen lijken uitstekend bruikbaar op locaties waar de kustachteruitgang is gekoppeld aan een van de kleine (vloed)geulen. Voorbeelden van dergelijke geulen zijn het Molengat bij zuidwest Texel, de

Zuiderstortemelk bij Vlieland, het Boomkensdiep bij Noordwest Terschelling en de anonieme geul ('Oostgat') bij de noordwestkust van Ameland.

In de Zeeuwse en Zuid-Hollandse Delta is veel ervaring opgedaan met zandsluitingen van (delen van) getijdebekkens (Struik e.a., 1992). Deze zandsluitingen waren overigens van een veel grotere omvang dan de sluiting van de relatief kleine geulen waarvan hier sprake is. Op basis van relatief eenvoudige berekeningen kan vastgesteld worden hoeveel en hoe snel zand moet worden aangebracht om een volledige sluiting te bereiken (Struik e.a., 1992). Voor het aanbrengen van zanddammen dwars op de kust is relatief veel zand nodig ten opzichte van reguliere zandsuppleties.

Effectiviteit in termen van de zandvoorraad van de Nederlandse kust
Wanneer Noordzeezand (zand van buiten het zanddelend systeem) gebruikt wordt voor de aanleg van zandsluitingen, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem. De prijs per m^3 is laag en de aangebrachte volumes zijn groot, waarmee zandsluitingen een zeer effectieve methode vormen om het kustfundament op peil te houden.

6 Aanbevelingen voor het kustbeheer van eilandkoppen

In dit rapport wordt een beknopt overzicht gegeven van de mogelijke kustbeheermaatregelen bij de eilandkoppen van de Wadden en de Kop van Noord-Holland. Deze kustgebieden hebben een vorm en dynamiek die afwijkt van de Hollandse kust en de kust van de centrale delen van de Waddeneilanden, omdat de ontwikkelingen voor een belangrijk deel worden bepaald door de buitendelta's van de Waddenzee.

6.1 Twee operationele doelstellingen en de drietrapsstrategie

Het vigerende Nederlandse kustbeleid is gericht op het duurzaam handhaven van de veiligheid en duurzaam behoud van de functies en waarden in het duingebied. Op basis van de kennis van de dynamiek van de kust en van het zanddelende systeem is het kustbeleid vertaald in twee operationele doelstellingen voor beheermaatregelen:

1. Het handhaven van de kustlijn, door lokale kustachteruitgang tegen te gaan.
2. Het handhaven van de totale zandvoorraad in de kustzone, door de zandvoorraad van de kust aan te vullen.

En bij de operationele doelstelling is de drietrapsstrategie van - vrij laten bewegen - zandbuffers aanleggen - zand vasthouden uitgangspunt voor het inrichten van de beheermaatregelen.

6.2 Vier kustbeheermaatregelen

Vier kustbeheermaatregelen voor eilandkoppen, namelijk zandsuppleties op het strand, harde kustverdedigingsmaatregelen, zandbuffers (geulwandsuppleties en zanddammen) en zandsluitingen zijn bekeken in het licht van bovenstaande operationele doelstellingen.

Strandsuppleties

1e Operationele doelstelling: +/-

Wanneer de kustachteruitgang wordt veroorzaakt door de landwaartse verplaatsing van een geul compenseren strandsuppleties dit slechts voor een deel.

2e Operationele doelstelling: +

Wanneer Noordzeezand wordt gebruikt, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem.

Aanbevolen wordt om strandsuppleties in te blijven zetten, zolang het alternatief geulwandsuppletie of zandbuffer nog niet operationeel beproefd is. Zandsuppleties zijn de aangewezen maatregelen om strandverlaging en strandachteruitgang nabij eilandkoppen te compenseren.

Harde verdedigingen (bestortingen, strandhoofden, korte en lange dammen)

1e Operationele doelstelling: +

Wanneer ontwerp en uitvoering van de harde kustverdedigingen goed passen bij het erosieprobleem kunnen deze lokaal zeer effectief zijn.

2e Operationele doelstelling: -

Harde kustverdedigingswerken leveren geen aanvulling van de zandvoorraad van de Nederlandse kust en belemmeren het zandtransport langs de kust

Harde kustverdedigingsmaatregelen zijn de laatste keuze in de drieptrapsstrategie voor kusdtbeheer. Derhalve wordt, in lijn met huidige kustbeleid, aanbevolen tot terughoudend bij het inzetten van harde kustverdedigingsmaatregelen.

Zandbuffers (geulwandsuppleties, zanddammen)

1e Operationele doelstelling: + ?

Indien voldoende zand wordt aangebracht, langs de geulwand of dwars op de geul, zal een getijdegeul tijdelijk uit de kust worden gehouden. De samenhang van kust, buitendelta en Waddenzee blijft in stand en daarmee de zanduitwisseling in het zanddelende systeem.

2e Operationele doelstelling: +

Wanneer Noordzeezand wordt gebruikt wordt de zandbuffers, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem.

Ervaringen met grote ingrepen met zand rond de eilandkoppen zijn zeer beperkt, zodat zandbuffers vooralsnog papieren oplossingen zijn. Aanbevolen wordt om kennis op te bouwen over de effectiviteit (levensduur, kosten, omgevingseffecten) van zandbuffers op basis van praktijkervaring, omdat zandbuffers een alternatief zijn voor strandsuppleties en harde kustverdedigingswerken.

Zandsluitingen

1e Operationele doelstelling: + ?

In situaties met een relatief kleine getijdegeul onder de kust neemt een zandsluiting de oorzaak van de kustachteruitgang weg en ontstaat ter plaatse een zandbuffer voor de kust.

2e Operationele doelstelling: +

Wanneer Noordzeezand wordt gebruikt wordt de zandsluitingen, zal het zand direct ten goede komen aan het zandbudget van het zanddelend systeem.

Ook zandsluitingen van kustnabije (vloedgeulen) voldoen op papier aan beide doelstellingen van het kustbeheer. Met een zandsluiting wordt op een grootschalige wijze ingegrepen in de natuurlijke dynamiek, maar blijft ruimte voor de natuurlijke ontwikkelingen. Praktijkervaring met zandsluitingen ten bate van het kustbeheer ontbreekt. Aanbevolen wordt om deze beheersoptie nader te verkennen.

6.3 Tijdelijke overbrugging bij morfologische cyclus

Veel van beheerproblemen bij eilandkoppen zijn gekoppeld aan cyclische ontwikkelingen van geulen en platen op de buitendelta's. Het is belangrijk om bij het inzetten van maatregelen vanwege een tijdelijke negatieve trend, ook rekening te houden met toekomstige positieve ontwikkelingen. De positie van een geul fixeren met harde kustverdedigingswerken is op korte termijn een uitstekende oplossing, maar op deze manier worden de natuurlijke positieve ontwikkelingen op langere termijn vrijwel uitgesloten. *Met zachte kustverdedigingswerken, in de vorm van zandsuppleties om een erosieve periode te overbruggen of (partiele) zandsluitingen om de periode van erosie versneld te beëindigen, is er nog alle ruimte voor toekomstige positieve natuurlijke ontwikkelingen.*

6.4 'No regret': beheermaatregelen met zand

Ingrepen met zand, ook in de vorm van zandbuffers of zandsluitingen, kunnen beschouwd worden als 'no regret' maatregelen. Na enige tijd heeft de doorgaande erosie de aangebrachte zandvoorraad opgeruimd en is uitgangssituatie hersteld. Dit geldt niet voor harde kustverdedigingswerken, waarvan het opruimen (kostbare) inspanning vereist. Van zachte ingrepen die niet de beoogde effectiviteit of levensduur hebben, hoeft men dan ook minder spijt te hebben dan van harde ingrepen die niet voldoen.

6.5 De integrale afweging en lokale haalbaarheid zijn doorslaggevend

De drie verschillende kustbeheermaatregelen zijn vergeleken ten aanzien van de twee operationele doelstellingen die voortkomen uit het vigerende kustbeleid. Bij deze vergelijking zijn andere doelen en randvoorwaarden, zoals kosten, risico's, ecologische, en juridische overwegingen buiten beschouwing gebleven. In de praktijk moet voor iedere locatie een integrale afweging gemaakt worden over de haalbaarheid van verschillende beheermaatregelen

6.6 Een afwegingskader voor het kustbeheer van eilandkoppen

De voor- en nadelen van verschillende kustbeheermaatregelen zijn op rij gezet in het licht van de twee operationele doelstellingen van het kustbeheer. Naast de twee operationele doelstellingen moeten de verschillende beheermaatregelen worden vergeleken in termen van financiële haalbaarheid, ecologische effecten, juridische aspecten en risico's en effecten op andere doelen. In ieder geval dient getoetst te worden of de kustbeheermaatregel voldoen aan het strategische doel van het kustbeleid, namelijk het duurzaam behoud van functies en waarden.

Tabel 10.1
Overzicht van de verschillende aspecten van een afwegingskader voor de haalbaarheid van kustverdedigingsmaatregelen (zie bijlage 2). + :positief; ± : neutraal; - : mogelijk negatieve gevolgen; -- : zeker negatieve gevolgen ; ? onvoldoende kennis

	Baten & Kustbeleid				Lasten		Risico's		
Kustbeheermaatregel	Drietrapsstrategie	Handhaven kustlijn 1e operationele doelstelling	Grootschalige zandbalans 2e operationele doelstelling	Ecologisch en Waterkwaliteit	Korte termijn kosten: Aanleg	Lange termijn kosten: Onderhoud	Voorbereidingstraject	Uitvoeringstegenvallers	Effectiviteit en omgevingseffecten
Zandsuppleties op het strand	1	+	+	?	±	-	+	±	+
Zandbuffers (geulwandsuppleties & zanddammen) & Zandsluitingen	2	?	+	?	-	?	±	±	±
Harde kustverdedigings-maatregelen	3	+	--	?	--	-	-	-	-

Een afwegingskader kan bij het bepalen van de haalbaarheid een nuttig instrument zijn. In het afwegingskader worden de verschillende functies van kust én buitendelta uitgewerkt. Bestaande afwegingskaders, zoals in de PKB Waddenzee (Ministerie van Volkshuisvesting, Ruimtelijke Ordening en Milieu, 2001), voor de lange termijn ontwikkeling van de Westerschelde (Graveland e.a., 2002), ingrepen ten bate van de kustveiligheid (Maasbommel, 2003) en zandwinning op de Noordzee (Willems, 2002) kunnen hiervoor als basis dienen.

7 Aanbevelingen voor zandwinning ten bate van het kustbeheer

7.1 Zandwinning voor het kustbeheer

Kustbeheermaatregelen met zand, in de vorm van zandsuppleties op het strand, geulwandsuppleties of zanddammen maken zandwinning noodzakelijk. De buitendelta's van de Waddenzee liggen dichtbij de suppletielocaties en bevatten grote volumes zand, waarmee de buitendelta's een voor de hand liggende bron van (relatief goedkoop) zand vormen. Zandwinning is echter niet toegestaan op buitendelta's, in de kustzone en in de Waddenzee. Voor zandwinning zijn op de Noordzee, buiten de –20 m NAP contour zandwingebieden aangewezen. Het zandwinbeleid voor de Waddenzee en aangrenzende gebieden is onder andere ingegeven door de kennis van het zanddelende systeem (Werkgroep 1, 1981).

7.2 Zandwinning op de buitendelta's en de tweede operationele doelstelling van het kustbeleid

De buitendelta's van de Waddenzee zijn een onlosmakelijk onderdeel van het zanddelende systeem, samen met de aangrenzende kust en de Waddenzee (Hoofdstuk 5). Zandwinnen op de buitendelta's om er zandsuppleties mee uit te voeren betekent dus: zand herverdelen binnen het zanddelende systeem. De tweede doelstelling van het kustbeheer, het op peil houden van de zandvoorraad (Hoofdstuk 6), wordt niet bereikt met suppletiezand uit de buitendelta's. Zandwinnen op de buitendelta is daarom in het beste geval een tijdelijke maatregel, die hoe dan ook moet worden gevolgd door compensatie van de zandverliezen met zand uit de Noordzee.

7.3 Een bredere kijk op de effecten van zandwinning

Kustbeheermaatregelen met zand sorteren wél effect op de tweede doelstelling van het kustbeheer wanneer hiervoor zand van de Noordzee en géén effect wanneer zand uit de buitendelta's wordt gebruikt. Vanuit het kustbeleid heeft zandwinning op de Noordzee daarom de voorkeur boven zandwinning op de buitendelta's. Daarnaast zijn er ook fysische en ecologische argumenten om zandwinning op de Noordzee te prefereren. In Bijlage 2 is een beschrijving gegeven van verschillende aspecten die bij zandwinning een rol spelen.

Aanbevolen wordt om zand te winnen op de Noordzee (buiten de –20 m dieptelijn), omdat de tweede doelstelling van het kustbeheer, aanvullen van de zandvoorraad van het fundament van de kust, niet wordt bereikt als het suppletiezand gewonnen wordt op de buitendelta's. Deze aanbeveling stemt overeen met het vigerende zandwinbeleid, dat zandwinning alleen op de Noordzee toestaat.

7.4 Morfologisch baggeren

In het verleden zijn zandwinputten met meer en minder succes gebruikt om de ontwikkelingen op de buitendelta (van het Eijerlandse Gat: Rakhorst, 1999), op de Voordelta (Krabbegat: in Rijkswaterstaat, 1988 ,Onrustpolder: De Groot, 2002) en in getijdebekkens (Robbengat: Rakhorst, 1981) te sturen. Dit zogeheten 'morfologische baggeren' kan een positief effect hebben op de kust, zodat de eerste doelstelling van het kustbeheer, het handhaven van de kustlijn, makkelijker wordt behaald. Bij morfologisch baggeren moeten de voordelen voor het behalen van de eerste doelstelling worden afgewogen tegen het niet behalen van de tweede doelstelling. Daarnaast spelen de effecten op andere doelen een rol. Het afwegingskader dat in het volgende hoofdstuk wordt gepresenteerd biedt hier aanknopingspunten voor.

8 Conclusies

De eilandkoppen van de Waddenzee en de Kop van Noord-Holland hebben een vorm en dynamiek die afwijkt van de 'normale' Hollandse kust en de centrale delen van de waddeneilanden. De vorm en dynamiek van deze kustvakken is het direct resultaat van de buitendelta's van de Waddenzee. De kustachteruitgang in het zanddelende systeem van buitendelta's, eilandkusten en Waddenzee is gerelateerd aan de 'zandvraag' van Waddenzee, als gevolg van de stijgende zeespiegel.

Wanneer functies en waarden van de achterliggende gebieden in het geding zijn, noopt de kustachteruitgang van de koppen van eilanden tot het nemen van beheermaatregelen. Uitgangspunt voor het inrichten van de beheermaatregelen is de drietrapsstrategie van vrij laten bewegen - zandbuffers aanleggen - zand vasthouden. De beheermaatregelen worden beoordeeld op twee operationele doelstellingen:
1. Het tegengaan van structurele kustachteruitgang.
2. Het aanvullen van de zandvoorraad van het gehele zanddelende systeem.

Beheermaatregelen in de vorm van zandsuppleties, op het strand of onderwater, maken zandwinning noodzakelijk. Om aan de tweede operationele doelstelling van het kustbeleid te voldoen, moet het zand worden gewonnen buiten het zanddelende systeem van buitendelta's, eilandkusten en Waddenzee. Zandwinning op de buitendelta's is een tijdelijke maatregel,die niet wordt aanbevolen.

Strandsuppleties zijn bij situaties met een getijdegeul dicht onder de kust niet altijd toereikend voor het tegengaan van structurele kustachteruitgang en voldoen dan niet aan de eerste operationele doelstelling. In dergelijke gevallen moet worden gekeken naar andere beheermaatregelen.

Zandbuffers en zandsluitingen zijn een alternatief voor strandsuppleties bij eilandkoppen met een getijdegeul dicht onder de kust. Zandbuffers en zandsluitingen leveren een bijdrage aan de zandvoorraad van het zanddelende systeem en daarmee aan de tweede operationele doelstelling. Aanbevolen wordt om de effectiviteit van zandbuffers op de achteruitgang van de kust te bepalen door op verschillende locaties zandbuffers aan te brengen en de morfologische ontwikkelingen te monitoren en te evalueren.

Harde kustverdedigingsmaatregelen in de vorm van geulwandbestortingen, korte of lange dammen zijn de laatste en minst wenselijke optie in de drietrapsstrategie voor het kustbeheer. Harde ingrepen kunnen de kustlijnachteruitgang effectief tegengaan, maar leveren geen bijdrage aan de zandvoorraad van de kust en kunnen de uitwisseling van zand hinderen. Aanbevolen wordt om zeer terughoudend met harde kustverdedigingsmaatregelen te blijven omgaan.

De drie typen beheermaatregelen zijn niet vergeleken ten aanzien van kosten, risico's, en ecologische effecten. In de praktijk moet voor iedere locatie een integrale afweging gemaakt worden over de haalbaarheid van de beheermaatregelen, waarbij verschillende alternatieven vergeleken worden. Een van de hindernissen bij het vergelijken van beheermaatregelen is het ontbreken van voldoende ecologische kennis van eilandkoppen en buitendelta's.

Colofon

Deze syllabus
Deze syllabus is een uitsnede uit het Rijkswaterstaat RIKZ rapport 'Kustverdediging van de koppen van de Waddeneilanden; de dynamiek van de kust nabij buitendelta's en passende maatregelen voor het kustbeheer' (rapportnummer RIKZ/2004.017)
Het rapport is geschreven door
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Gerard Poot

Een pdf van dit rappor kunt u downloaden van de RIKZ website:
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Literatuurverwijzingen
Deze vindt u in het complete rapport, zie bovenstaande website.

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Functioneel en technisch ontwerp van "beach connected structures"

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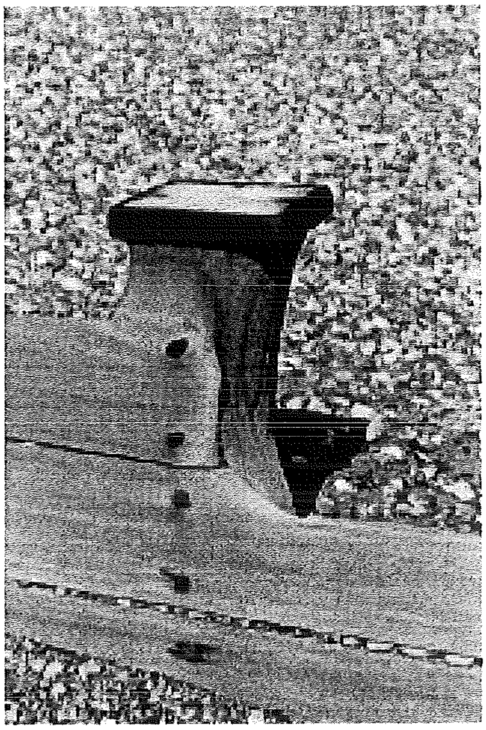
Inleiding
In deze bijdrage wordt ingegaan op "beach connected structures", dat wil zeggen kustbeschermingsconstructies die op de één of andere wijze verbonden zijn met de kustlijn. Dit zijn onder andere:

- strandhoofden, paalhoofden en paalschermen
- langsverdedigingen, duinvoetverdedigingen
- headlands

Strandhoofden
Het is belangrijk om bij strandhoofden en aanverwante constructies een onderscheid te maken tussen brandingsstroomremmers en getijstroomremmers. Brandingsstroomremmers zijn hoofden die het door scheef invallende golven gegenereerde zandtransport verminderen en tot een zaagtandachtige kust leiden. Dit is uitgebreid besproken door Van de Graaff (IK5). Een getijstroomremmer is vergelijkbaar met een rivierkrib en heeft vooral tot taak om de getijstroom van de kust weg te houden. Getij is een stroming die ontstaat door waterstandsverschil. Zo'n stroming kan verminderd worden door het plaatsen van weerstandselementen in de stroombaan. Een brandingsstroom wordt intern aangedreven, deze kan dus niet verminderd worden door weerstandselementen, maar wel tegen gehouden worden door "schotten".

Traditioneel worden strandhoofden uitgevoerd in breuksteen. In Nederland is de traditie echter om strandhoofden uit te voeren in setsteen, dit vanwege de traditioneel hoge steenprijs. Het valt te betwijfelen of, gegeven de hoge arbeidskosten op dit moment, een gezet hoofd te prevaleren valt boven een hoofd in reuksteen. Breuksteen constructies kunnen berekend worden met de standaardmethoden (van der Meer formule, bij voorkeur aangepast voor ondiep water, zie hiervoor de bijdrage van Van Gent IK17). Rekenregels voor het ontweren van zetsteenconstructies zijn ontwikkeld voor dijktaluds en min of meer ook toepasbaar voor hoofden. De ontwerpregels zijn niet zonder meer toe te passen, omdat de belasting op een andere manier aangrijpt en daardoor bestaat er een onzekerheid. Recent onderzoek naar de stabiliteit van zetsteen op een havendam die geheel onder water staat geeft een indruk van de sterkte, maar ook in dit geval is het niet mogelijk om tot een eenduidige, simpele rekenregel specifiek voor strandhoofden te komen. Dit houdt dus in dat er bij zetsteen strandhoofden een bepaalde overdimensionering nodig is.

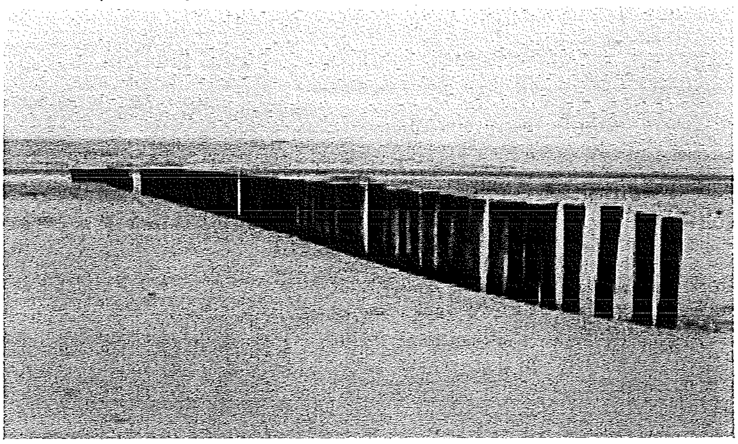
Met name in Engeland is er veel ervaring met houten strandhoofden. Deze hoofden worden daar traditioneel gebruikt op relatief smalle stranden (met een grote korrel, zeer grof zand of grind). Om de hoofden goed te laten functioneren wil men de hoofden niet te hoog boven het strand laten uitsteken. Hoge hoofden zijn kostbaar, visueel niet aantrekkelijk en geven een sterke verstoring van het stroombeeld (en dus veel negatieve effecten van additionele turbulentie). Een hoog hoofd is ook niet nodig omdat (zeker bij grof bodemmateriaal) het sedimenttransport dicht bij de bodem plaats vindt. Stranden zijn nogal dynamisch en daarom moet de hoogte van het hoofd regelmatig aangepast worden aan de hoogte van het strand.


Erosie van een onderdeel van een houten strandhoofd

Bij houten strandhoofden is dat eenvoudig te realiseren. Er is de afgelopen eeuw veel geëxperimenteerd met allerlei vormen van houten strandhoofden. Recent is door HR Wallingford in opdracht van Defra [Crossman & Simm, 2004] een rapport gemaakt met een samenvatting van die ervaringen. In dit rapport wordt ook ingegaan op het onderhoud van houten hoofden. Hout is natuurlijk erg gevoelig voor erosie door met wind en water meegevoerde zandkorrels. De houtconstructie moet dus zo uitgevoerd worden dat versleten onderdelen makkelijk vervangen kunnen worden.

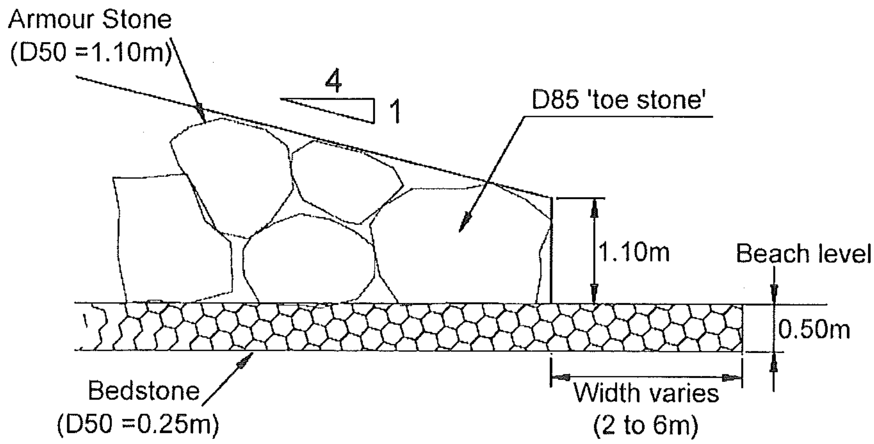
De conclusie van deze studie is ook dat effectieve houden hoofden een schuttingvorm moeten hebben. Bijzondere vormen (zoals een zigzag hoofd) zien er wel indrukwekkend uit, maar de meerwaarde weegt vaak niet op tegen de extra kosten. Als meerwaarde wordt genoemd dat door de zigzagvorm de muistroom niet direct langs het hoofd loopt, maar op enige afstand blijft en het hoofd dus minder diep gefundeerd hoeft te worden.

Ook in Nederland is er wat ervaring met houd en strandhoofden, maar het betreft hier dan paalhoofden en paalschermen. Paalhoofden zijn stenen hoofden met daarop één of twee rijen houten palen, paalschermen zijn rijen palen zonder een stenen berm eronder.


Paalscherm op Ameland

Het doel van deze palen is het vergroten van de weerstand tegen doorstroming; ze zijn dus eigenlijk alleen effectief als getijstroomremmer, en niet als brandingsstroomremmer. Ervaringen met paalschermen die het door brekende golven aangedreven langstransport zouden moeten verminderen zijn dan ook vrij negatief.

Paalhoofden zijn daarentegen vaak wel effectief. De palen verminderen de getijstroom dicht bij de kust, en het brandingsstroomtransport bij de bodem kan door de stenen bermen effectief tegengehouden worden. Door het plaatsen van palen op de stenen bermen wordt ook de stroomgradiënt (in de getijstroom) bij de kop van het strandhoofd minder, en daardoor is er soms een wat lichtere kopbestorting mogelijk.



Teen van een "slecht" uitgevoerd strandhoofd (Bradbury *et al*, 2003)

In Engeland is recentelijk onderzoek gedaan naar bewust 'slecht' uitgevoerde breuksteen strandhoofden. Men heeft de filterconstructie geminimaliseerd, iedere vorm van laagopbouw weggelaten en te lichte stenen gebruikt. De ervaringen hiermee zijn eigenlijk heel goed. Zoals te verwachten is er meer schade dan bij een "goed" ontworpen constructie, maar herstel is eenvoudig en uit te voeren met materieel zonder hoge mobilisatiekosten. De gekapitaliseerde kosten van deze constructies blijken lager uit te vallen dan bij onderhoudsarme constructies.

Kunststoffen
Sinds een aantal jaren wordt er ook steeds meer gekeken naar kunststofconstructies, zoals Geocontainers, Geotubes en allerlei vormen van grote zandzakken. Het voordeel van deze constructies is dat ze zonder steen toegepast kunnen worden, en dat is voor veel plaatsen in de wereld een belangrijk voordeel. Een heel groot bezwaar van kunststofconstructies op en boven de waterlijn is hun kwetsbaarheid voor vandalisme. Met een eenvoudig stanleymes is een strandhoofd gebouwd met Geotubes in een paar minuten volledig te vernielen. Er bestaan tegenwoordig wel methoden om gaten te dichten met kunststofplaten, maar dat is toch vrij arbeidsintensief, en deze platen moeten onmiddellijk na het ontstaan van het gat aangebracht worden. Los daarvan worden kunststofconstructies vaak visueel niet positief ervaren.

Bij de uitvoering van kunststofconstructies is de vullingsgraad heel belangrijk. Als de constructie niet voldoende gevuld is, kan er door golfwerking beweging in de constructie ontstaan, waardoor deze uiteindelijk faalt. Bij een overvulling bestaat een grote kans op scheuren van het doek door te grote spanningen.

Schade aan een Geotube in Dubai (Weerakoon, 2003)

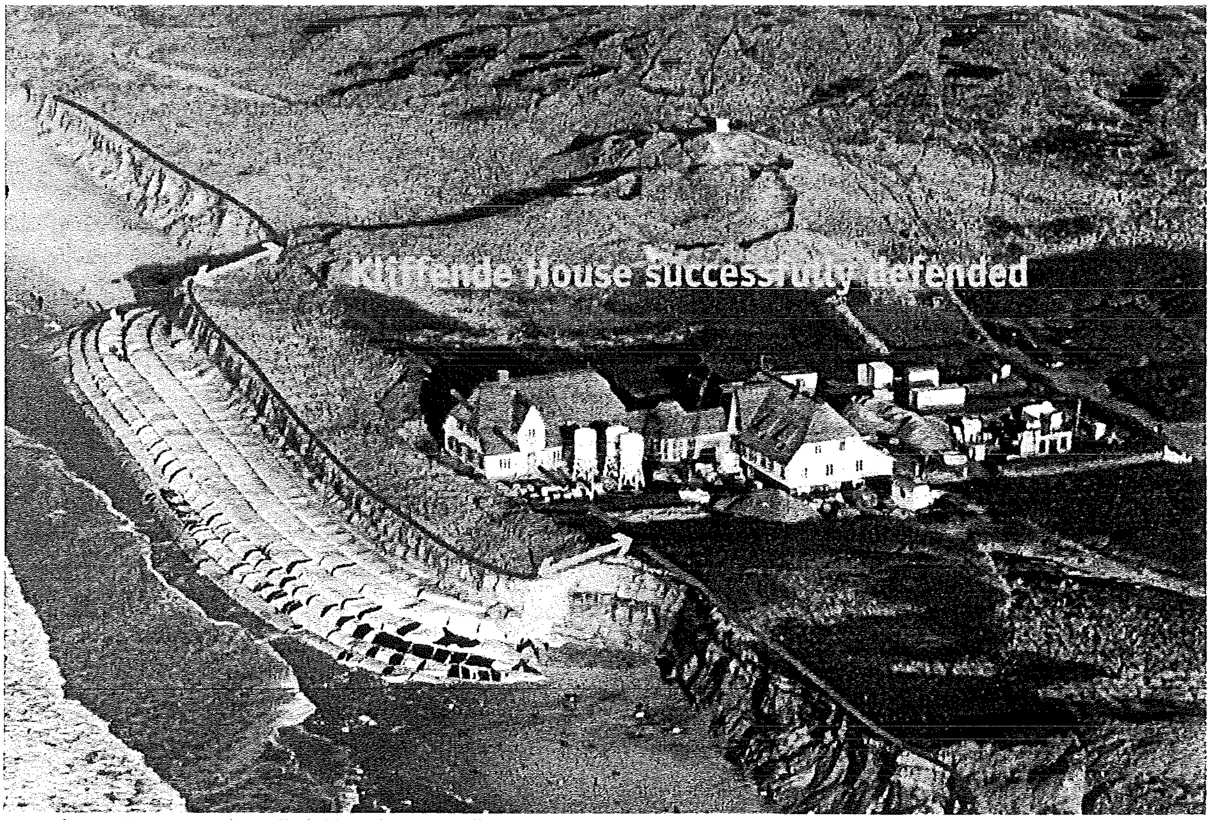
Een belangrijk aandachtspunt bij Geotubes is ook dat zij een bepaalde stijfheid hebben, en dus niet zonder meer alle bodemveranderingen kunnen volgen. Dat betekent dat als er een erosiekuil ontstaat die onder de bodem van de Geotube uitkomt, er kans is op onderloopsheid. Dit leidt dan weer tot grotere erosie en uiteindelijk wordt het gat zo graat dat de Geotube kan gaan verzakken. Bij een aantal kunststofconstructies is dit een wezenlijk probleem gebleken.

Duinvoetverdedigingen en andere vormen van langsverdediging

Zoals al aangegeven in de bijdrage van Van de Graaff is een langsverdediging zelden een oplossing tegen doorgaande kusterosie. Of, zoals een engelse geomorfoloog het eens uitdrukte: "Het effect van de klifvoetverdediging is dat de kliferosie is teruggebracht van 1 m/jaar naar 10 m per 10 jaar".

Langsverdedigingen hebben alleen zin bij een kust die langjarig stabiel is (of stabiel gehouden wordt door zandsuppletie), maar die zo nu en dan bij een storm afslaat. Het natuurlijk proces zorgt dan dat het afgeslagen zand na verloop van tijd weer terugkomt. Het is natuurlijk heel vervelend als dit precies ter plekke van bebouwing gebeurt. Het effect van een langsverdediging is dat tijdens een storm er geen zand aangevoerd wordt uit het achterland. De consequentie daarvan is dat het strand verlaagd. Bij een langjarig stabiele kust zal dit zand op natuurlijke wijze weer terugkomen. Bij een langzaam eroderende kust zal dit niet gebeuren en is een suppletie van het strand noodzakelijk.

Bij het ontwerp van een langsverdediging moet er dus rekening mee gehouden worden dat er voor de constructie een ontgrondingskuil ontstaat. Deze kuil mag niet leiden tot instabiliteit van de constructie. Dat kan door de verdediging voldoende diep door te trekken of door een horizontale bescherming aan te brengen. Bij een horizontale bescherming wordt deze zodanig uitgevoerd dat het zeewaartse einde mee kan vervormen met de verdieping van het strand. Dit wordt dus uitgevoerd als falling apron of hanging apron.

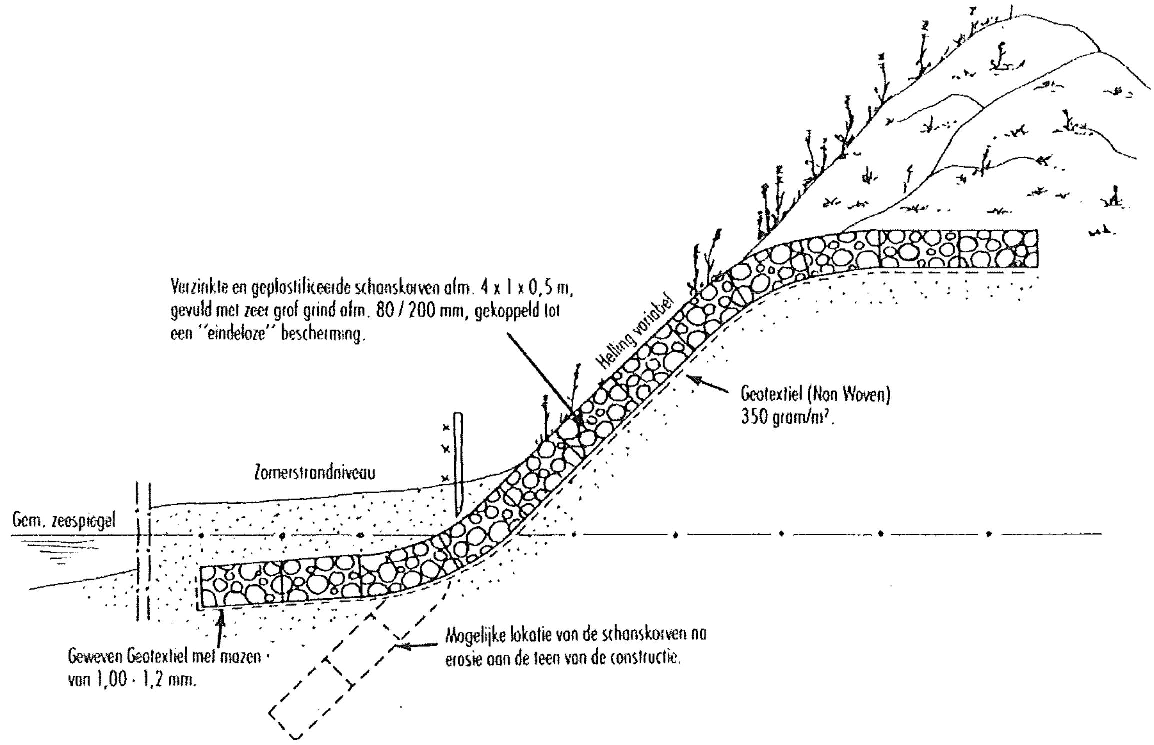


Bescherming van het "Kliffende Haus" op Sylt met SoftRock van Nauhe

Als alternatief kan men ook de hele constructie laten vervormen. Door de constructie op te bouwen uit losse, vervormbare eenheden (bijvoorbeeld zandzakken) kan de constructie meezakken in de ontgrondingskuil en zo zijn beschermende functie handhaven. De consequentie zal wel vaak zijn dat na een zware storm herstel van de constructie nodig is (herstapelen van zandzakken, toevoegen van nieuwe zakken). De bescherming van het "Kliffende Haus" op Sylt met SoftRock van Nauhe is een voorbeeld van zo'n constructie.

In verband met het uiterlijk van dergelijke constructies worden ze vaak toegepast als verborgen kering. Met name voor keringen waarvan verwacht wordt dat ze slechts zelden aangesproken worden is dit een goede oplossing. Bovendien zijn verborgen keringen niet erg kwetsbaar meer voor vandalisme. Er speelt dan echter wel een beheersprobleem. Omdat de kering niet te zien is, is hij ook niet meer te inspecteren.

Een wat minder vervormbare, maar toch ook niet echt starre constructie, is een schanskorfconstructie. Ook hier verdient het de voorkeur om deze uit te voeren als verborgen kering. Omdat het nazakken van deze constructie beperkt is, zal de constructie ook wat ingegegraven moeten worden om het probleem met de ontgrondingskuil te voorkomen. In bijgaande tekening is een ontwerp van een leverancier weergegeven. Hier is de constructie niet uitgevoerd als verborgen kering. He bezwaar is dan wel dat doordat de schanskorven aan de oppervlakte liggen, de kans op schade door vandalisme vrij groot is. Bovendien is de kans op corrosie ook veel groter bij deze wijze van uitvoeren.



Schanskorfconstructie gebruikt als duinvoetverdediging

De ontgrondingskuil kan relatief eenvoudig berekend worden met het programma Durosta [Steetzel, 1993]. Dit programma is gekalibreerd met proeven in de Deltagoot en met een beperkt aantal waarnemingen van ontgrondingskuilen voor Nederlandse duinvoetverdedigingen.

Headlands

Headlands zijn vaste punten in zee, waartussen een parabolische strandvorm ontstaat. Dit kan als een vorm van kustverdediging gebruikt worden. De bedoeling is dat de cellen tussen de vaste punten zo groot zijn dat er geen zandtransport uit de ene cel naar de andere is, en dat er alleen seizoensfluctuaties zijn binnen de cel. Dit concept is enkele jaren geleden ontwikkeld door Silvester [1997], die het ziet als een oplossing voor vrijwel alle kustproblemen. Dit is een wat te extreme visie op deze headlands. Voor zandige kusten zonder rotsformaties is het dan ook de vraag of een kustbescherming met headlands een efficiënte manier van werken is. In dit geval zijn waarschijnlijk strandhoofden of offshore golfbrekers minstens zo effectief en vermoedelijk goedkoper. Het concept van headlands werkt goed in geval van een min of meer natuurlijke baai met een strand dat niet geheel aan de eisen van het gebruik voldoen.

Balneário Camboriú

Als voorbeeld kan het strand van Balneário Camboriú in Brazilië. Dit strand is vrij smal, en doordat het op het oosten georiënteerd is, is er onvoldoende zon op het strand (door hoogbouw). Men wil dit strand nu verbreden door suppletie, en om te voorkomen dat het suppletiestrand naar naburige kustvakken verdwijnt moeten de headlands aan beide zijden van de baai wat meer zeewaarts verplaats worden. Een en ander is te berekenen met een standaard spiraal. Deze spiraal heeft een paar stuurcoëffi-ciënten, die afhangen van de gemiddelde golfrichting. Zij kunnen ook bepaald worden aan de hand van opnemingen in het terrein bij stabiele kustvormen in hetzelfde gebied. De betrouw-baarheid van deze methode is echter vrij klein. Het is heel moeilijk om uit golfmetingen precies te bepalen wat de maatgevende golfrichting is.

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Palm Island Jumeirah and The World

Dedicated Breakwater Design

Stichting Postacademisch Onderwijs

Innovatief denken en doen in kustverdediging
26 april 2005
H38509.2M.010

A COMPANY OF

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CONTENTS

1 LOW CRESTED BREAKWATER DESIGN FOR DUBAI ISLAND PROJECTS

Since 2000 Royal Haskoning Maritime Division is consultant to Palm Island Developers Office in Dubai UAE, presently known as NAKHEEL Properties, for the design of low crested sea defences for their man made island projects at and off the coast of Dubai.

The fairly open and straight coast of Dubai in between Dubai City (at the creek) and the Jebel Ali commercial Port is facing the south-east end of the Arabian Gulf. Jumeirah coast as it is called, offers excellent possibilities to realize the innovative ideas of HH Sheikh Mohammed bin Rashid Al Maktoum through the commercial efforts of NAKHEEL (The Client). Three island projects are presently under construction, i.e. Palm Island Jumeirah, Palm Island Jebel Ali and The World (Figure 1.1). The Client started first with Palm Island Jumeirah which is scheduled to be finished in 2006.

Figure 1.1 The Dubai Island Projects presently under construction.

All three projects are unique in many ways. Upon completion it will have created the world's largest three man made islands, which are expected to be visible from the moon. Palm Island Jumeirah measures approximately four by five kilometre, Palm Island Jebel Ali approximately six by seven kilometre and finally The World seven by nine kilometre.

The two Palm Islands will each provide a mix of exclusive sea-facing residences, hotels and leisure amenities ranging from marinas to shopping malls and tourism attractions. The Palm Islands derive their name from their shape. They are constructed in the shape of a palm tree and consist of a trunk, a crown with fronds and a surrounding breakwater island: the Crescent.

Palm Island Jumeirah has been built about 25 kilometres from the centre of Dubai City. Its trunk is connected to the Jumeirah coastline via a bridge. Palm Island Jebel Ali is being built about 15 kilometres further away to the south-west of Jebel Ali Port and is also connected to the shore via bridges. In total the two Palm Islands will create approximately 120 kilometres of new beachfront in the emirate.

The World is a collection of 300 private islands, each about 2 to 8 hectare in size, that resembles the actual shape of the world's continents. The World is located some 4 km off the coast of Dubai at a point halfway between Palm Island Jumeirah and Port Rashid.

Due to the scale of the man made islands they are extending into relatively deep water, about 10 to 15 meters of water depth. As the Dubai Coast is attacked by the so-called Shamal waves in wintertime, the islands will have to be protected against wave attack under (design) storm conditions. Furthermore one can expect under storm conditions quite a high water level set up due to wind surge effects. In addition this part of the Arabian Gulf encounters an astronomical tidal range of 1.2 meters.

In view of above hydraulic boundary conditions it is necessary to protect both Palm Island projects with a crescent breakwater and The World with an artificial barrier reef. The ultimate challenge for The Client, and thus for their consultant Royal Haskoning, was to create breakwaters that have a natural look and feel, are environmentally sound, enable the inhabitants and tourists to enjoy the sea (undisturbed view) and also are giving enough protection at minimum costs.

To accomplish this, first of all it was decided to use as much as possible natural materials, to be precise: (coarse) sand, quarry run and rocks up to about 5 to 6 ton. Secondly it was chosen to limit the level of the crest of the crescent breakwater to about 1 meter above reclaimed island level in order not to disturb the view on the sea.
After an intensive engineering design effort by Royal Haskoning supported by WL | Delft Hydraulics regarding numerical modelling of hydraulic boundary conditions and extensive scale model tests regarding breakwater stability and wave overtopping, it was found that low crested rubble mound breakwater profiles with a dedicated shape for each of the particular projects satisfied the user requirements the best. The finally developed breakwater profiles were also tuned to the construction methods and available equipment of the marine contractors involved.

One of the interesting engineering challenges was the fact that both the crescent breakwaters and the barrier reef are continuously curved. This required a thorough analysis of the hydraulic loading conditions at each location along the breakwaters and the barrier reef, and consequently a tuned cross sectional design as function of the location.

The environmental impact of the project has been of paramount concern to the developers since its inception and this has been one of the guidelines for the design. Originally the sea bed in the area was largely the same as the nearby desert on land, a barren area consisting of fine sediment over hard, sedimentary rock. There were very few boulders (rocky substrate), but where they existed they were teeming with life (biomass). Because, where there are rocks there is shelter, and where there is shelter there is food for fish. In fact the amount of sea life under boulders is hundreds of times greater than in the rest of the area.

The Crescent and Reef, being built of millions of tonnes of rock, recreates life-sustaining 'ecological networks' on a massive scale. In fact, wherever a breakwater island structure is built, there will be countless jagged rocks, sheltering nooks and crannies for the natural food chain to attach itself to.

ALTERNATIVE SYSTEMS FOR COASTAL PROTECTION - AN OVERVIEW -

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ABSTRACT

This contribution presents an overview of the various available methods for shore stabilization and beach erosion control, with special emphasis on the alternative solutions and novel materials and systems in various design implementations. Within alternative systems special attention is paid to artificial reefs. Novel systems as geosystems (geotubes, geocontainers, geocurtains) and some other (often patented) systems have gained popularity in recent years because of their simplicity in placement, cost effectiveness and environmental aspects. Other alternative systems are overviewed in the Appendix. The related websites are listed at the end.
KEYWORDS: alternative coastal systems, low-crested structures, design aspects, overview.

INTRODUCTION

Coastal users and managers all over the world are frequently faced with serious erosion of their sandy coasts. Possible causes of erosion include natural processes (i.e. action of waves, tides, currents, sea level rise, etc.) and sediment deficit due to human impact (i.e. sand mining and coastal engineering works). Countermeasures for beach erosion control function depend on local conditions of shore and beach, coastal climate and sediment transport. Continuous maintenance and improvement of the coastlines, together with monitoring and studies of coastal processes have yielded considerable experience on various coastal protection measures all over the world.In general, a coastal structure is planned as a practical measure to solve an identified problem. Starting with identification of the problem (e.g. shoreline erosion), a number of stages can be distinguished in the design process for a structure: definition of functions, determination of boundary conditions, creating alternatives, geometrical design and the final choice of functional solution. After the choice of functional solution has been made the structural design starts including creating structural alternatives (ie. using different materials and various execution methods). The final choice will be made after verification of various structural solutions in respect to the functional, environmental and economic criteria.

The present overview provides information on some available methods for shore stabilization and beach erosion control, with special emphasis on the alternative solutions and novel materials and systems in design implementations.

ALTERNATIVE SYSTEMS FOR COASTAL PROTECTION

GENERAL

Various structures/systems can be of use in coastal and shoreline structures, from traditional rubble and/or concrete systems to more novel systems as geotextile, gabions, patented systems and others. However, there is a growing interest both in developed and in developing countries in low cost or novel methods of shoreline protection particularly as the capital cost of defence works and their maintenance continues to rise. The shortage of natural rock in certain geographical regions can also be a reason for looking to other materials and systems. Despite this interest there is little published and documented information about the performance of low cost or patented structures especially at more exposed wave climate.

Below, an overview is given of the various available methods for shore stabilization and beach erosion control, with special emphasis on the alternative solutions and novel materials and systems in various design implementations. Within alternative systems special attention is paid to artificial reefs of various design. Novel systems as geosystems (geotubes, geocontainers, geocurtains) and some other (often patented) systems (Reef Balls, Aquareef, beach drainage, etc) have gained popularity in recent years because of their simplicity in placement and constructability, cost effectiveness and their minimum impact on the environment. An overview is given on design and application of the existing novel systems in coastal engineering

The geotextile systems as bags, mattresses, tubes and containers filled with sand or mortar, and artificial seaweed or geotextile curtains, can be a good and mostly cheaper alternative for more traditional materials/systems as rock, concrete units or asphalt. These new systems were applied successfully in number of countries and they deserve to be applied on a larger scale. Because of the lower price and easier execution these systems can be a good alternative for coastal protection and coastal structures in developing countries. The main obstacle in their application is however the lack of proper design criteria. An overview is given on application of the existing novel systems and reference is made to the design criteria. The details on these systems can be found in Pilarczyk and Zeidler (1996) and Pilarczyk (2000).

ARTIFICIAL REEFS AND EXAMPLES

Wave climate, in combination with currents, tides and storm surges, is the main cause of coastal erosion problems. Various coastal structures can be applied to solve, or at least, to reduce these problems. They can provide direct protection (breakwaters, seawalls, dikes) or indirect protection (offshore breakwaters of various designs), thus reducing the hydraulic load on the coast (Figure 1). Low crested and submerged structures (LCS) as detached breakwaters and artificial reefs are becoming very common coastal protection measures (used alone or in combination with artificial sand nourishment) (Pilarczyk, 2003).

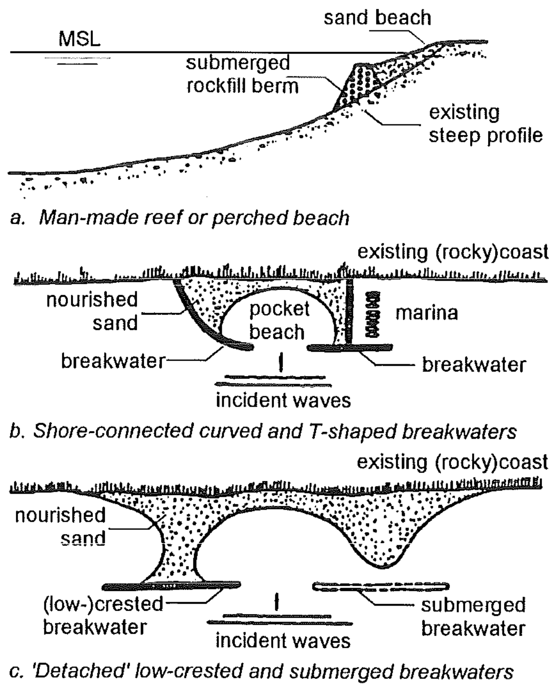

Figure 1. Examples of low-crested structures

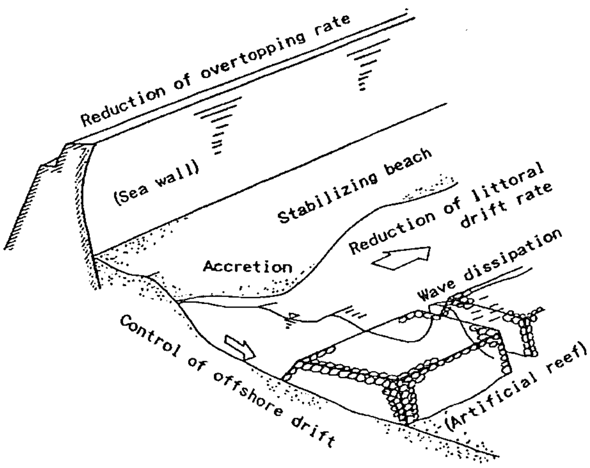

Figure 2. Objectives of artificial reefs

Their purpose is to reduce the hydraulic loading to a required level allowing for a dynamic equilibrium of the shoreline. To obtain this goal, they are designed to allow the transmission of a certain amount of wave energy over the structure in terms of overtopping and transmission through the porous structure (emerged breakwaters) or wave breaking and energy dissipation on shallow crest (submerged structures). Due to aesthetical requirements low freeboards are usually preferred (freeboard around SWL or below). However, in tidal environment and frequent storm surges they become less effective when design as a narrow-crested structures. That is also the reason that broad-crested submerged breakwaters (called also, artificial reefs) became popular, especially in Japan (Figures 2 and 3). However, broad-crested structures are much more expensive and their use should be supported by a proper cost-benefit studies. On the other hand the development in alternative materials and systems, for example, the use of sand-filled geotubes as a core of such structures, can reduce effectively the cost (Pilarczyk, 2000, 2003).

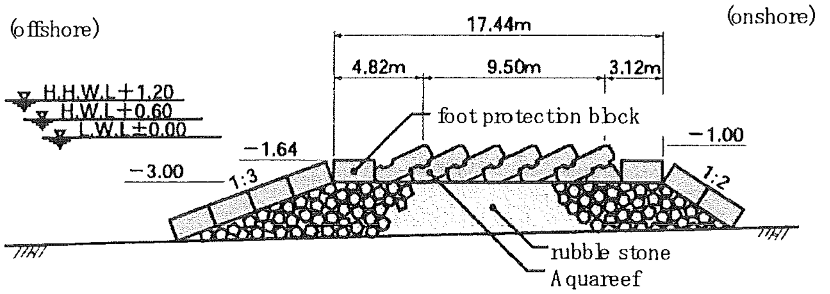

Figure 3. Example of Aquareef

Figure 4. Example of Reef Balls units

The relatively new innovative coastal solution is to use artificial reef structures called "Reef Balls" as submerged breakwaters, providing both wave attenuation for shoreline erosion abatement, and artificial reef structures for habitat enhancement. An example of this technology using patented Reef Ball™ is shown in Figure 4.

Reef Balls are mound-shaped concrete artificial reef modules that mimic natural coral heads (Barber, 1999). The modules have holes of many different sizes in them to provide habitat for many types of marine life. They are engineered to be simple to make and deploy and are unique in that they can be floated to their drop site behind any boat by utilizing an internal, inflatable bladder. Stability criteria for these units were determined based on analytical and experimental studies. Some technical design aspects are treated in publications by Harris, mentioned in references, which can be found on the website. Worldwide a large number of projects have already been executed by using this system. More information can be obtained from: www.artificialreefs.org, reefball@reefball.com, and, on wave transmission and design: http://www.advancedcoastaltechnology.com/science/DrHarrisWavereduction.htm; www.reefball.com/**submerged**breakwater/ Armono%20and%20Hall.pdf ,

WAVE TRANSMISSION AND COASTAL RESPONSE

For shoreline control the final morphological response will result from the time-averaged (i.e. annual average) transmissivity. However, to simulate this in the designing process, for example, in numerical simulation, it is necessary to know the variation in the transmission coefficient for various submergence conditions. Usually when there is need for reduction in wave attack on structures and properties the wave reduction during extreme conditions (storm surges) is of interest (reduction of wave pressure, run up and/or overtopping). In both cases the effectiveness of the measures taken will depend on their capability to reduce the waves. While considerable research has been done on shoreline response to exposed offshore breakwaters, very little qualitative work has been done on the effect of submerged offshore reefs. Therefore, the main purpose of this paper is to provide information on wave transmission for low-crested structures and to refer the reader to recent literature.

The transmission coefficient, K_t, defined as the ratio of the wave height directly shoreward of the breakwater to the height directly seaward of the breakwater, has the range $0<K<1$, for which a value of 0 implies no transmission (high, impermeable), and a value of 1 implies complete transmission (no breakwater). Factors that control wave transmission include crest height and width, structure slope, core and armour material (permeability and roughness), tidal and design level, wave height and period.

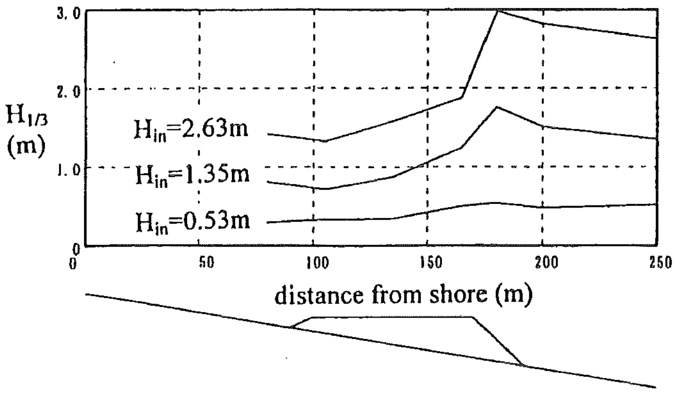

Figure 5. Distribution of waves along the center of reef (Ohnaka&Yoshizwa, 1994, Aono&Cruz, 1996)

As wave transmission increases, diffraction effects decrease, thus decreasing the size of sand accumulation by the transmitted waves and weakening the diffraction-current moving sediment into the shadow zone (Hanson and Kraus, 1991). It is obvious that the design rules for submerged structures should include a transmission coefficient as an essential governing parameter. Example of the transmission over the submerged structures (Aquareef) is shown in Figure 6. More detailed descriptions of the functional and technical design of these reefs can be found in (Hirose et al., 2002).

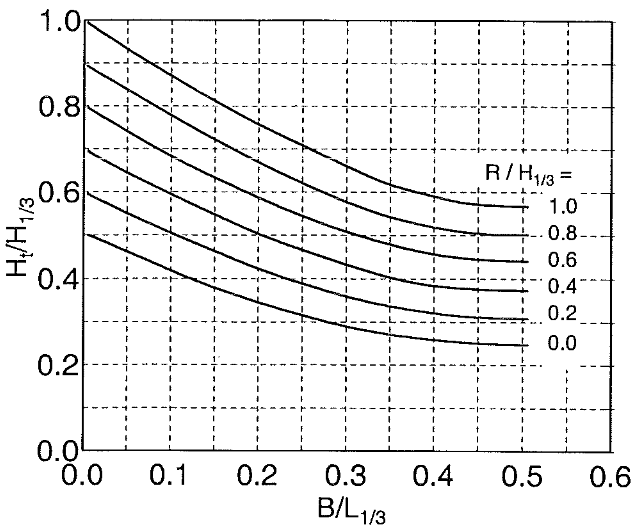

Figure 6 .General transmission characteristic for AquaReef (H_t is the transmitted wave height recorded on the landward side, $H_{1/3}$ and $L_{1/3}$ are the significant wave height and wavelength at the toe of the rubble mound, B is the crown width, and R is the submergence of the crown)

Examples of prototype measurements in Japan can be found in (Hamaguchi et al., 1991, (Funakoshi et al., 1994, (Ohnaka and Yoshizwa, 1994). The Japanese design procedure can be found in Uda (1988) and Yoshioka (1993).

Layout and morphological response

Most commonly an offshore obstruction, such as a reef or island, will cause the shoreline in its lee to protrude in a smooth fashion, forming a salient or a tombolo. This occurs because the reef reduces the wave height in its lee and thereby reduces the capacity of the waves to transport sand. Consequently, sediment moved by longshore currents and waves builds up in the lee of the reef (Black, 2001). The level of protection is governed by the size and offshore position of the reef, so the size of the salient or tombolo varies in accordance with reef dimensions. Of course, one can expect this kind morphological change only if the sediment is available (from natural sources or as sand nourishment).

The examples of simple geometrical empirical criteria for the lay-out and shoreline response of the detached, exposed (emerged) breakwaters can be found in (i.e., Harris & Herbich, 1986, Dally & Pope, 1986, etc.). To include the effect of submergence (transmission) Pilarczyk proposes, at least as a first approximation, adding the factor $(1\text{-}K_t)$ to the existing rules. Then the rules for low-crested breakwaters can be modified to (for example):

Tombolo: $L_s/X > (1.0 \text{ to } 1.5)/(1\text{-}K_t)$ or $X/L_s < (2/3 \text{ to } 1)(1\text{-}K_t)$, or $X/(1\text{-}K_t) < (2/3 \text{ to } 1) L_s$)

Salient: $L_s/X < 1/(1\text{-}K_t)$ or $X/L_s > (1\text{-}K_t)$, or $X/(1\text{-}K_t) > L_s$

For salients where there are multiple breakwaters: $G\, X/L_s^2 > 0.5(1\text{-}K_t)$
Where L_s is the length of a breakwater and X is the distance to the shore, G is the gap width, and the transmission coefficient K_t is defined for annual wave conditions.
The gap width is usually $L \leq G \leq 0.8\, L_s$, where L is the wavelength at the structure defined as: $L = T\,(g\,h)^{0.5}$; T = wave period, h = local depth at the breakwater.

These criteria can be used as preliminary design criteria for distinguishing shoreline response to a single, transmissive detached breakwater. However, the range of verification data is too small to permit the validity of this approach to be assessed for submerged breakwaters. Actually, a similar approach is used for the submerged breakwaters within the scope of the European project DELOS (Jimenez and Sanches-Arcilla, 2002). In general, it can be stated that numerical models (i.e., Genesis, Delft 2D-3D, Mike 21, etc.) can already be treated as useful design tools for the simulation of morphological shore response to the presence of offshore structures. Examples can be found in (Hanson& Krause, 1989, 1991, Groenewoud et al. 1996, Bos et al., 1996, Larson et al., 1997, Zyserman et al., 1999).
As mentioned above, while considerable research has been done on shoreline response to exposed offshore breakwaters, very little qualitative work has been done on the effect of submerged offshore reefs, particularly outside the laboratory. Thus, within the Artificial Reefs Program (Black&Mead, 1999) (www.asrltd.co.nz), Andrews (1997) examined aerial photographs seeking cases of shoreline adjustment to offshore reefs and islands. All relevant shoreline features in New Zealand and eastern Australia were scanned and digitized, providing123 different cases. A range of other statistics, particularly reef and island geometry, was also obtained. Some of these results are repeated below.

To examine the effects of wave transmission on limiting parameters, data for reefs and islands were considered separately. The data indicated that tombolo formation behind islands occurs with L_s/X ratios of 0.65 and higher and salients form when L_s/X is less than 1.0. Therefore, for islands the L_s/X ratios determining the division between salients and tombolos are similar to those from previously presented breakwater research. Similarly, data resulting from offshore reefs indicate that tombolo formation occurs at L_s/X ratios of 0.6 and higher, and salients most commonly form when L_s/X is less than 2. The data suggests that variation in wave transmission (from zero for offshore islands through to variable transmission for offshore reefs) allows a broader range of tombolo and salient limiting parameters. Thus, a reef that allows a large proportion of wave energy to pass over the obstacle can be (or must be) positioned closer to the shoreline than an emergent feature. Thus, from natural reefs and islands the following general limiting parameters were identified:

Islands: Tombolos form when $\frac{L_s}{X} > 0.65$ Salients form when $\frac{L_s}{X} < 1.0$

Reefs: Tombolos form when $\frac{L_s}{X} > 0.60$ Salients form when $\frac{L_s}{X} < 2.0$

Non-depositional conditions for both shoreline formations occur when $L_s/X < \approx 0.1$.
The choice of the layout of submerged breakwaters can also be affected by the current patterns around the breakwaters. The Japanese Manual (1988) provides information on various current patterns for submerged reefs (Yoshioka et al., 1993).

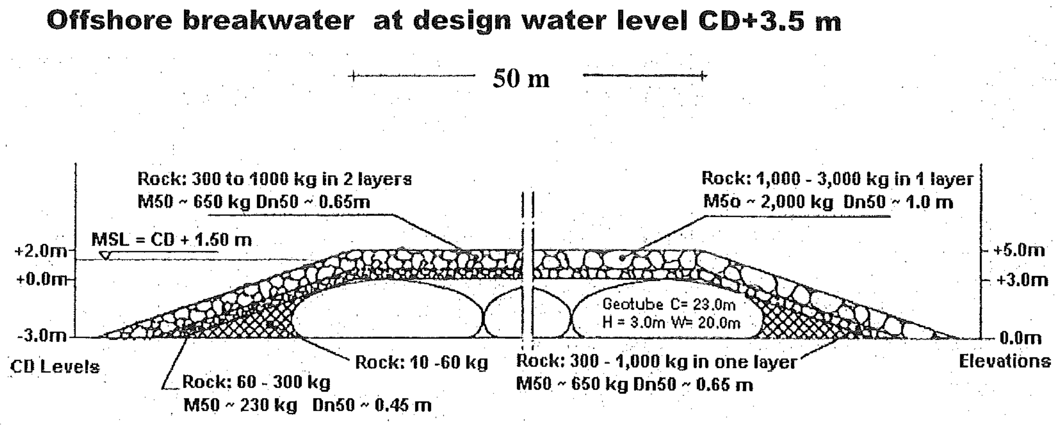

Figure 7 Example of reef structure composed with geotubes (Fowler et al., 2002); http://geotecassociates.com/

GEOSYSTEMS AND EXAMPLES

Geotextile systems utilize a high-strengt synthetic fabric as a form for casting large units by filling with sand or mortar. Within these geotextile systems a distinction can be made between: bags, mattresses, tubes and containers. All of which can be filled with sand or mortar. Mattresses are mainly applied as slope and bed protection. Bags are also suitable for slope protection and retaining walls or toe protection but the main application is the construction of groynes, perched beaches and offshore breakwaters. The tubes and containers are mainly applicable for construction of groynes, perched beaches and (offshore) breakwaters, and as bunds for reclamation works (see Figures 7 and 8). They can form an individual structure in accordance with some functional requirements for the project but also they can be used complementary to the artificial beach nourishment to increase its lifetime. Especially for creating the perched beaches the sand tubes can be an ideal, low-cost solution for constructing the submerged sill .

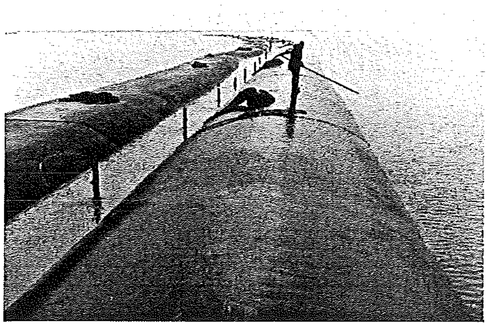

Fig.8. Geotube as a breakwater

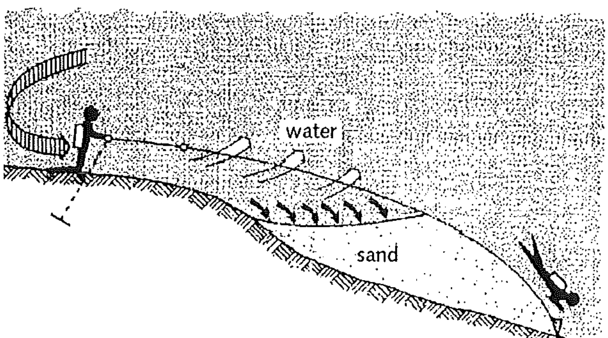

Figure 9. BEROSIN system (inclined curtain)

An interesting application for shore erosion control is the geocurtain known under the name BEROSIN (Figure 9). The BEROSIN curtain is a flexible structure made of various woven geotextiles which after placing by divers near the shore and anchoring to the bed catches the sand transported by currents and waves providing accretion on a shore and preventing the erosion. The horizontal curtain (sheet) can be easily spread (at proper sea conditions) by a small workboat and two divers. The upper (shore-side) edge, equipped with some depth-compensated floaters, should be properly anchored at the projected line. The sea-side edge is kept in position by the workboat. By ballasting some of the outside pockets at the lower edge with sand or other materials and with help of divers, the lower edge is sinking to the required position. The proper choice of permeability of geotextile creates the proper conditions for sedimentation of suspended sediment in front/or under the curtain and at the same time allowing the water to flow out without creating too high forces on the curtain and thus, on the anchors. In case of Pilot project at the coast of Vlieland (NL), some of the horizontal curtains placed in the intertidal zone have provided a growth of a beach/foreshore of 0.5 to 1.0 m within a week while others within a few weeks (Pilarczyk, 2000).

For more detailed information on these and other coastal protection systems and measures applied nowadays throughout the world, together with recommendations and guidelines, the reader will be guided to the relevant manuals and publications .

REMARKS ON STABILITY ASPECTS

Structural design aspects of low-crested structures are relatively well described in a number of publications (Ahrens, 1987, Uda, 1988, Van der Meer, 1987, 1988, CUR/CIRIA, 1991, US Corps, 1993, Pilarczyk&Zeidler, 1996, Vidal et al., 1992, 1998, etc). Some useful information on the design of breakwaters on reefs in shallow water can be found in Jensen et al. (1998).
Usually for submerged structures, the stability at the water level close to the crest level will be most critical. Assuming depth limited conditions (H_s=0.5h, where h=local depth), the (rule of thumb) stability criterion becomes: $H_s/\Delta D_{n50}=2$ or, $D_{n50}= H_s/3$, or $D_{n50}=h/6$, where $D_{n50}=(M_{50}/\rho_s)^{1/3}$; D_{n50} = nominal stone diameter and M_{50} and ρ_s= average mass and density of stone. It should be noted that some of useful calculation programs (including formula by Van der Meer) are incorporated in a simple expert system CRESS, which is accessible in the public domain (http://www.ihe.nl/we/dicea or www.cress.nl). Useful information on functional design and the preliminary structural design of low crested-structures, including cost effectiveness, can be found in CUR (1997). Alternative solutions, using geotubes (or geotubes as a core of breakwaters), are treated in (Pilarczyk, 2000). An example of this application can be found in (Fowler et al., 2002).

CONCLUSIONS

The author does not intend to provide the new design rules for coastal structures. However, it is hoped that this information will be of some aid to designers looking for new sources, which are considering these kinds of structure and improving their designs.

Offshore breakwaters and reefs can be permanently submerged, permanently exposed or inter-tidal. In each case, the depth of the structure, its size and its position relative to the shoreline determine the coastal protection level provided by the structure. To reduce the cost some alternative solutions using geosystems can be considered. The actual understanding of the functional design of these structures needs further improvement but may be just adequate for these structures to be considered as serious alternatives for coastal protection.

Continued research, especially on submerged breakwaters, should further explore improved techniques predict shore response and methods to optimise breakwater design. A good step (unfortunately, limited) in this direction was made in a collective research project in the Netherlands (CUR, 1997). Research and practical design in this field is also the focus of the "Artificial Reefs Program" in New Zealand (www.asrltd.co.nz), the International Society for Reef Studies (ISRS) (www.artificialreefs.org), and the European Project DELOS (Environmental Design of Low Crested Coastal Defence Structures, 1998-2003) (http://www.delos.unibo.it).
These new efforts will bring future designers closer to more efficient application and design of these promising coastal solutions. The more intensive monitoring of the existing structures will also help in the verification of new design rules. The intention of this literature search is to uncover, as far as possible, the technical informations on these systems and make them available for the potential users. It will help to make a proper choice for specific problems/projects and it will stimulate the further developments in this field. International cooperation in this field should be further stimulated.

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APPENDIX A

Review of alternative systems for coastal protection

Krystian W. Pilarczyk

1 INTRODUCTION

Some coastal environments may be regarded as rather stable (rock and reef coasts) while other are more vulnerable (sand and mud coasts). Hence coastal users and managers all over the world are frequently faced with serious erosion of their sandy coasts. Possible causes of erosion include natural processes (i.e. action of waves, tides, currents, sea level rise, etc.) and sediment deficit due to human impact (i.e. sand mining and coastal engineering works). Countermeasures for beach erosion control function depend inter alia on local conditions of shore and beach, coastal oceanology and sediment transport. Continuous maintenance and improvement of the coastlines, together with monitoring and studies of coastal processes have yielded considerable experience on various coastal protection measures all over the world.

This contribution presents an overview of the various available methods for shore stabilization and beach erosion control, with special emphasis on the novel/alternative systems in various design implementations. More detailed information on other coastal protection systems and measures applied nowadays throughout the world, such as the modern techniques of beach nourishment and slope revetments, and their systems, together with recommendations and guidelines on groins and some other older solutions can be found in quite a few manuals and monographs, among which one can name SPM (1984), CEM (2002), CUR/RWS (1987), CIRIA (1991), Pilarczyk (1990, 1998, 2000), PIANC (1992) or Pilarczyk & Zeidler (1996). Information on some new developments can be found on the websites listed at the end of this contribution.

2 GENERAL APPROACH

2.1 Types and functions of coastal structures

Coastal management, in its broadest sense, must take into account all factors, which have bearing on the future of the coastal zone. Politics, policy making, planning, economy and a multitude of economic and non-economic users (e.g. natural reserve areas), environment protection and, by and large, sustainable development, all play significant roles and provide both motivation and background for coastal management. The factors of coastal management may well entail many scientific and engineering disciplines other than coastal engineering and at sites and locations far removed from the costal zone. Some of these factors interact with one another, others are incompatible. The extent to which this applies in any particular region, area, or specific site needs careful evaluation and compromise solutions. This is one of the major roles of coastal management.

The primary objectives of a typical coastal management study are to formulate long-term engineering planning, including financial strategies for the future usage, development and conservation of the coastal zone. In this process, priorities should be defined both for new works and essential maintenance, with estimates for contingency items to cover emergency situations, which inevitably occur (see Figure 1). The key element in any coastal management study is thorough understanding of coastal processes by which is meant the interaction between the hydraulic environment of winds, waves, tides, surges and currents with the geological conditions in the coastal zone. To be effective this may require a very broad view to be taken on a regional basis in the first instance. A regional cell could then be subdivided into smaller cells once the basic coastal processes had been established, and so on, with decreasing cell sizes until the cell in question becomes the specific one of the project itself. It is only in this way that the impact of new works in the coastal zone can be evaluated satisfactorily or long-term planning undertaken.

The basic tools of the coastal engineer are still fairly limited and comprise cross-shore structures (such as groins, jetties, spurs...) shore-parallel structures (offshore breakwaters, seawalls, revetments (generally close to shoreline) and dikes, headland structures and artificial beach nourishment (see Figure 2).

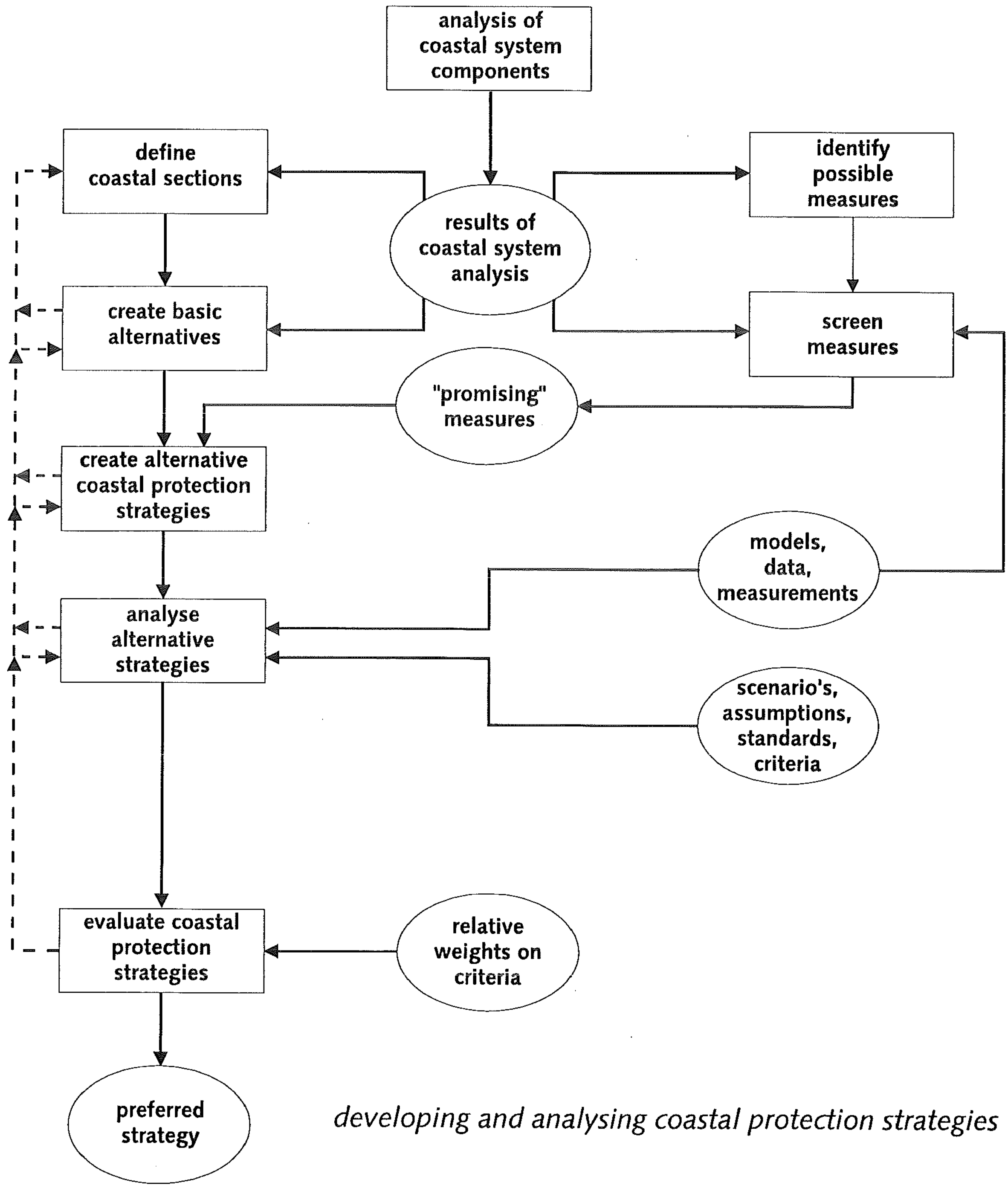


Figure 1 Analysis of coastal strategies

• Groins generate considerable changes in wave and circulation patterns but their basic function - to slow down the rate of littoral drift is sometimes overlooked. In the absence of beach nourishment, groins can redistribute the existing supply and, in a continuous littoral system, may be expected to create a deficiency at the downdrift end where the uncontrolled drift rate is reestablished. Without an adequate supply of beach material, groins are of limited value. In addition to controlling the rate of drift, groins are also used extensively to control the distribution of material along a frontage and to limit the temporary effects of drift reversal. There are unfortunately many examples where either bad design or failure to provide for the downdrift consequences has resulted in an adverse effect on the coastline. In other instances, failure to maintain groin systems might be worser than having no groins at all. Design information can be found in (Fleming, 1990).

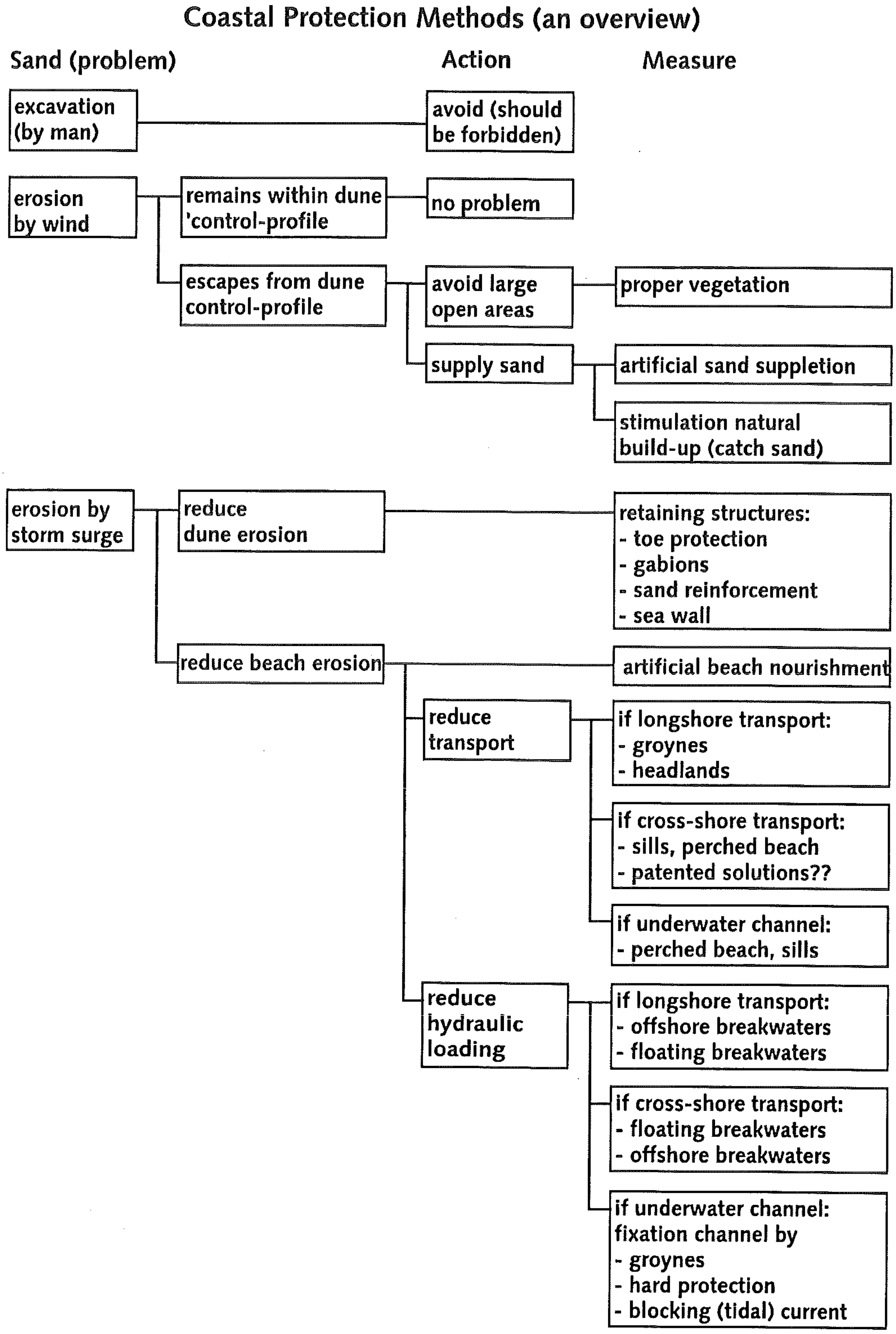


Figure 2 Schematic presentation of various shore protection measures

• Offshore breakwaters are usually provided either to reduce wave energy at shoreline structures or to modify the wave climate and enhance sediment transport patterns so as to improve beach levels and create desirable beach features, such as salients. Offshore breakwaters can be shore-connected or detached, submerged or emerging, longshore or oblique (Pilarczyk & Zeidler, 1996).

• Perched beach is a system consisting of a submerged breakwater (`sill' or `reef'), usually located not far away from shoreline, and artificial beach nourishment providing sand to the area extending between subaeral beach and the sill crest.
• Seawall (wording sometimes used interchangeably with bulkhead) is either a retaining wall intended to hold or prevent sliding of the soil behind it or a massive structure whose primary purpose is to protect the backshore from heavy wave action. Sometimes one speaks of `beach wall' or `shore wall' (Pilarczyk, 1990, CUR/CIRIA, 1991).
• Revetment is placed on a slope to shelter the adjacent uplands from erosion. Wave reflection, a serious disadvantage of vertical-wall bulkheads (seawalls), does not accelerate toe erosion as strongly at revetments as it does at seawalls (Pilarczyk, 1990,1998, CUR/RWS, 1995).
• Dikes are generally intended as means of flood prevention. The crest of a dike is elevated high enough to counteract or confine overtopping in rare storm surge events (Pilarczyk, 1990, 1998).
• Beach nourishment or fill (or recharge), consists in importation of granular material to beach from an external source. It is not new, and has been used in some countries for decades, but is now being applied to an increasing extent and in a greater variety of ways. The resulting beach provides some protection to the area behind it and also serves as a valuable recreational resource. The beach fill functions as an eroding buffer zone, and its useful life will depend on how quickly it erodes. One must be prepared to periodically renourish (add more fill) if erosion continues.
• Headland control has been devised by analogy to the Nature's efforts to keep in equilibrium a certain crenulate shape of erosion bays sculptured for thousands or so years. The crenulate shaped bays can be kept in equilibrium by use of a system of headlands. The headland system is claimed to be in feedback with coast and to combine the advantages of groins and detached breakwaters (shore-parallel or oblique) (Silvester & Hsu, 1993).

All forms of shore protection (i.e., groins, breakwaters, seawalls, revetments, bulkheads, beach-fill, etc.) have certain advantages and disadvantages. A shore-parallel (or obligue) breakwater, placed near the shoreline or off shore and designed either to intercept a portion of moving sediment or to protect a placed beach-fill, has the potential to perform close to the ideal for many types of coastal environments. A number of innovative solutions can be found in Silvester & Hsu (1993). Recent experience with design of beach stabilization structures can be found in (Bodge, 1998).

Various low-cost, environment-friendly, emergency and temporary measures, and their combinations provide alternatives to the principal measures. These systems are often appropriate for application only in sheltered waters. Inherent in the concept of environmental friendliness and low cost is the assumption on the equal importance of physics, engineering, ecology and economy. The approach in this paper is to provide a background for understanding of the physical mechanisms intervening in the operation of various coastal defence schemes (see also Pilarczyk & Zeidler, 1996).

2.2 Functional requirements and design

It is common knowledge that the realm of coastal processes and the interactions of offshore breakwaters and beaches can be broadly classified as far-field and near-field phenomena. The scales are somehow arbitrary but can be roughly identified as those greater and smaller, respectively, than characteristic dimensions of a structure or coastal feature.

The design procedures for coastal structures should embody geometrical design and structural design reflecting respectively the far-field and near-field requirements imposed on structures. This corresponds to the division of design procedures into two basic groups concentrating on overall layout and configuration of a structure as a whole, and its interaction with the coastal environment to produce desirable sedimentation patterns and coastal management effects stability and reliability of the structure and its components, hence dimensioning of structural constituents, associated with possible unavoidable and undesirable hazards due to the loadings exerted by the coastal environment. In other words, the first group involves design parameters producing the best environmental effectiveness of a structure in `ideal' conditions, i.e. upon negligence of possible `harmful by-effects', such as different modes of failures and instabilities, both overall and internal. The second group is concerned about these `by-effects' and provides the tools, which secure the integrity and proper operation of the structure and its components.

Table 1. Overview of systems according to CEM 2002

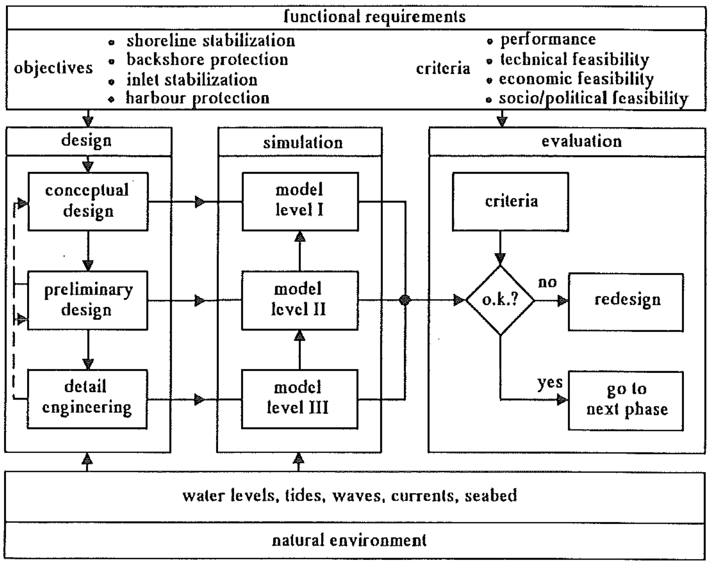
Types and Functions of Coastal Structures

Type of Structure	Objective	Principal Function
Sea dike	Prevent or alleviate flooding by the sea of low-lying land areas	Separation of shoreline from hinterland by a high impermeable structure
Seawall	Protect land and structures from flooding and overtopping	Reinforcement of some part of the beach profile
Revetment	Protect the shoreline against erosion	Reinforcement of some part of the beach profile
Bulkhead	Retain soil and prevent sliding of the land behind	Reinforcement of the soil bank
Groin	Prevent beach erosion	Reduction of longshore transport of sediment
Detached breakwater	Prevent beach erosion	Reduction of wave heights in the lee of the structure and reduction of longshore transport of sediment
Reef breakwater	Prevent beach erosion	Reduction of wave heights at the shore
Submerged sill	Prevent beach erosion	Retard offshore movement of sediment
Beach drain	Prevent beach erosion	Accumulation of beach material on the drained portion of beach
Beach nourishment and dune construction	Prevent beach erosion and protect against flooding	Artificial infill of beach and dune material to be eroded by waves and currents in lieu of natural supply
Breakwater	Shelter harbor basins, harbor entrances, and water intakes against waves and currents	Dissipation of wave energy and/or reflection of wave energy back into the sea
Floating breakwater	Shelter harbor basins and mooring areas against short-period waves	Reduction of wave heights by reflection and attenuation
Jetty	Stabilize navigation channels at river mouths and tidal inlets	Confine streams and tidal flow. Protect against storm water and crosscurrents
Training walls	Prevent unwanted sedimentation or erosion and protect moorings against currents	Direct natural or man-made current flow by forcing water movement along the structure
Storm surge barrier	Protect estuaries against storm surges	Separation of estuary from the sea by movable locks or gates
Pipeline outfall	Transport of fluids	Gravity-based stability
Pile structure	Provide deck space for traffic, pipelines, etc., and provide mooring facilities	Transfer of deck load forces to the seabed
Scour protection	Protect coastal structures against instability caused by seabed scour	Provide resistance to erosion caused by waves and current

As in many other engineering activities, the design of coastal structures should encompass the following considerations and stages:

(a) specification of the structure's function(s);
(b) description of the physical environment (`boundary conditions');
(c) selection of envisaged construction technologies;
(d) inclusion into design of the structure's operation and maintenance;
(e) conceptual design;
(f) preliminary design and selection of alternatives;
(g) geometrical dimensioning basing on far-field considerations;
(h) detailed design basing on near-field factors, including structural design;
(i) inclusion of possible construction constraints affecting the design;
(j) inclusion into design some flexibility allowing for redesign basing on monitoring of the operati on and effectiveness of the structure after construction.

Functional requirements and design outline are depicted in Figure 3. It is seen that the design in various stages is verified through the use of simulation models at different levels of complexity. Boundary conditions (bottom) constitute input to both design considerations and the models



employed, while the functional requirements (top) ensure evaluation of the suitability of the design and provide design objectives at the same time.

The starting point in the design process consists of the identification of the beach erosion problem, followed by the selection of the type of protection measure; the final design can incorporate the risk analysis. Attention is drawn to the proper choice of the shore protection measure. The selection is usually affected by the cost. For example, beach nourishment can be cost-effective for low sediment deficit but might be comparable with offshore breakwaters for higher erosion rates.

Figure 3 Design process

Aside from the cost, many other aspects must also be taken into account upon selection of the shore protection measure. Not shown in the drawing are legal restrictions, regional constraints and priorities, construction, operation and maintenance aspects, etc.

Coastal structures are constructed to protect life and property against storm surges, to combat erosion and/or to create (often artificial) beaches for recreational purposes, and to preserve the natural environment. However, the absolute safety of an area or structure is nearly impossible to achieve. Therefore it is much better to speak about the probability of failure (or safety) of a certain protection system. To implement this concept, all possible causes and outcomes of failure have to be analysed. This concept is actually being developed for breakwaters (PIANC, 1992) and the dike and dune design, mostly in the Netherlands (CUR/TAW, 1989, 1990). The `fault tree' is a handy tool for this aim. In the fault tree all possible modes of failure of elements, which can eventually lead to the failure of a structure section and to inundation are included. They can also badly affect the behaviour of the structure, even if the latter is properly designed on the whole. Although all categories of events which may cause the inundation of a land or damage of structure, are equally important for the overall safety, the engineer's responsibility is mainly limited to the technical and structural aspects. In the case of coastal structures the following major events can be distinguished:

- overflow or overtopping of the structure (i.e. instability of the superstructure);
- erosion or instability of slopes;
- instability of inner sections leading to progressive failure;
- scour and instability of toe-protection;
- instability of the foundation and internal erosion (i.e. piping);
- instability of the whole structure.

All these failure modes must be taken into account in the stage of structural design, by which the undesirable `by-effects' are prevented or eliminated.

3 ALTERNATIVE SYSTEMS FOR COASTAL PROTECTION

3.1 Introduction

In general, a coastal structure is planned as a practical measure to solve an identified problem. Starting with identification of the problem (e.g. shoreline erosion), a number of stages can be distinguished in the design process for a structure: definition of functions, determination of boundary conditions, creating alternatives, geometrical design and the final choice of functional solution. After the choice of functional solution has been made the structural design starts including creating structural alternatives (ie. using different materials and various execution methods). The final choice will be made after verification of various structural solutions in respect to the functional, environmental and economic criteria.

Various structures/systems can be of use in coastal and shoreline structures, from traditional rubble and/or concrete systems to more novel systems as geotextile, gabions, patented systems and others. However, there is a growing interest both in developed and in developing countries in low cost or novel methods of shoreline protection particularly as the capital cost of defence works and their maintenance continues to rise. The shortage of natural rock in certain geographical regions can also be a reason for looking to other materials and systems. Despite this interest there is little published and documented information about the performance of low cost or patented structures especially at more exposed wave climate.

The geotextile systems as bags, mattresses, tubes and containers filled with sand or mortar, and artificial seaweed or geotextile curtains, can be a good and mostly cheaper alternative for more traditional materials/systems as rock, concrete units or asphalt. These new systems were applied successfully in number of countries and they deserve to be applied on a larger scale. Because of the lower price and easier execution these systems can be a good alternative for coastal protection and coastal structures in developing countries. The main obstacle in their application is however the lack of proper design criteria. An overview is given on application of the existing novel systems and reference is made to the design criteria. The details on these systems can be found in Pilarczyk and Zeidler (1996) and in the references.

3.2 Geosystems and applications

Geotextile systems utilize a high strengt synthetic fabric as a form for casting large units by filling by sand or mortar, or as curtains collecting sand. At this moment there is a relative large number of products of this type on the markt provided by some specialistic companies all over the world. The following types and applications of geosynthetic systems can be distinguised (Pilarczyk, 1995, 2000):

1. Closed forms/units filled with sand, gravel or mortar: bags, mattresses, tubes, containers
2. Open-matting bags filled with stone or asphalt
3. Geotextile forms/moulds sand-filled structures
4. Geosynthetic sheets for dune reinforcement
5. Geotextile curtains for shore erosion control
6. Artificial seaweed mainly for scour prevention
7. Silt and pollution screens and fences
8. Geocells for surface (slope) erosion control
9. Geocomposite mats for drainage/erosion control
10. Traditional applications as geotextile filters
11. Water- or air-filled dams and breakwaters
12. Other (unclassified) systems

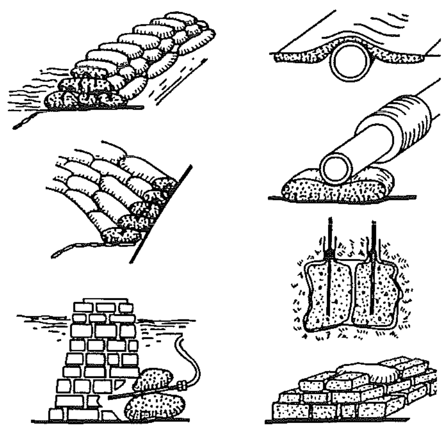


Figure 4 Application of bags

3.2 Geosynthetic forms

Mattresses are mainly applied as slope and bed protection. Bags are also suitable for slope protection and retaining walls or toe protection but the main application is construction of groyns, perched beaches and offshore breakwaters (Figure 4).

The tubes and containers are mainly applicable for construction of groyns, perched beaches and offshore breakwaters. They can form an individual structure conform some functional requirements for the project but also they can be used complementary with the artificial beach nourishment to increase its lifetime. Especially for creating the perched beaches the sand bags and/or sand tubes can be an ideal (often low-cost) solution for constructing the submerged sill (with a low wave loading).

Some coastal engineering concepts are shown in Figure 5. Underwater breakwaters and sills (purched beaches) are not easy to construct with traditional materials. In this respect (sand)tubes, although based on the same principle, are more advanced even by comparison with sandbags, which are only 1.0 to 5.0 m3 in capacity and are time_consuming as concerns both manufacturing and installation while hydraulic filling of tubes provides a few hundreds m^3 of sand in few hours. The sand-filled bags and/or tubes can be of use for constructing of groyns. Up till now there is no reliable design methods concerning the functioning of groyns. When the groyne will work satisfactorily such groyne can be strengthend additionally (if necessary) to get a permanent function. If not, the groyne can be easily demolished. In general, the sand-filled structure can be used as a temporary structures to learn the natural interactions/responses, or as the permanent structures at locations with relatively low wave attack (H<1.5m), or as submerged structures where direct wave forces are reduced by submergence. The units (if necessary) can be interconnected by bars or by creating a special interlocking shape.
These systems can also be applied in hydraulic/river engineering for constructing of spurdikes, guide dams, revetments, bottom groins, bottom protection, etc.. As other possible applications can be mentioned: containment dikes for storage of (contaminated) dredged material, dike or dune reinforcement, moulds for artificial sand structures, etc.

The main advantages of these systems in comparison with more traditional methods (rock, prefabricated concrete units, blockmats, asphalt, etc.) are: a reduction in work volume, a reduction in execution time, a reduction in cost, a use of local materials, a low-skilled labour and (mostly) locally available equipment. That means that in most, not too extreme cases/conditions the werk can be done by a local contractor under supervision of the specialistic experts/company.

Note: more detailed discription of geosystems is presented in 'Overview and design aspects of geosystems'.

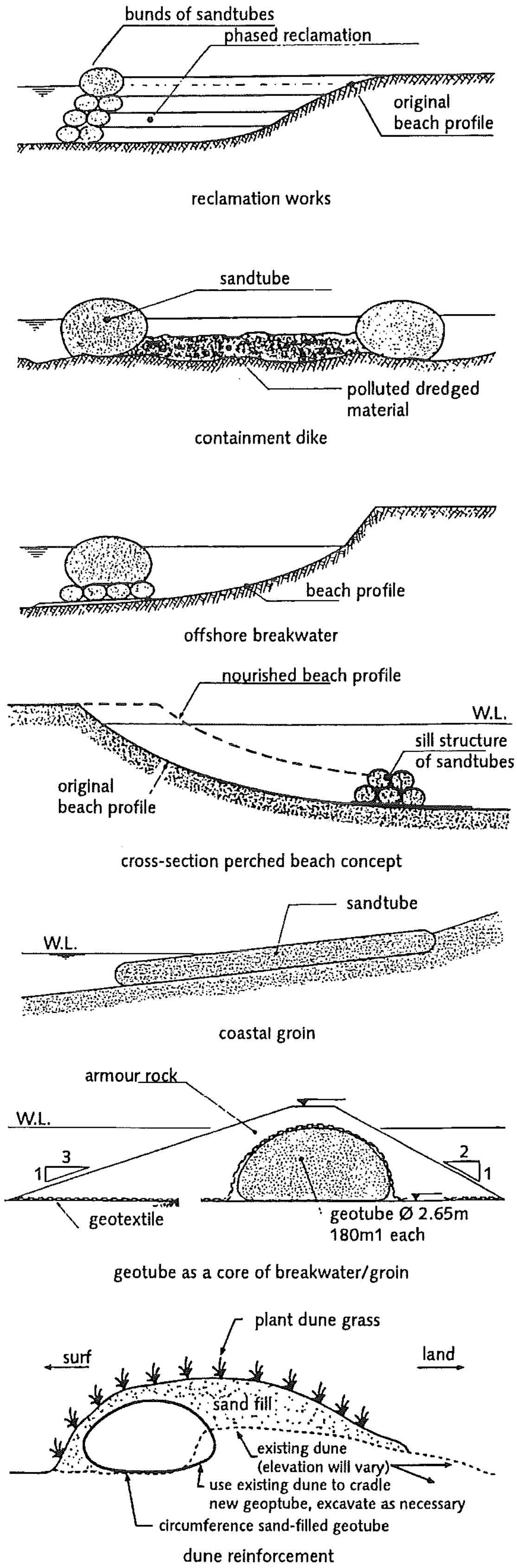
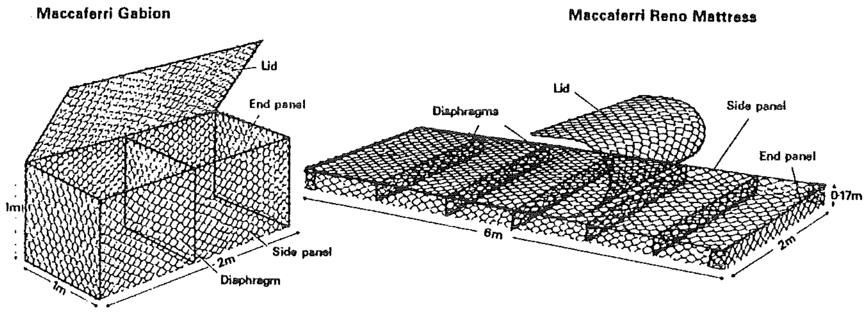
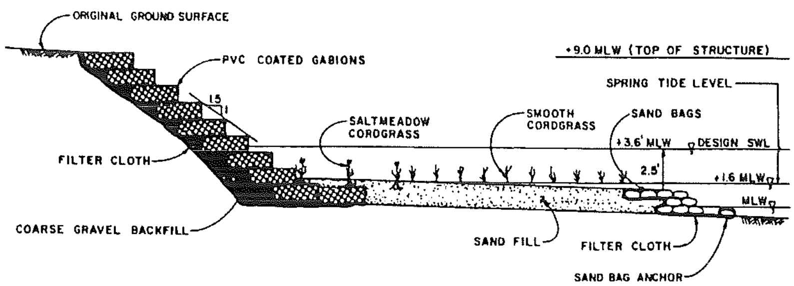


Figure 5 Coastal applications of geotubes and containers

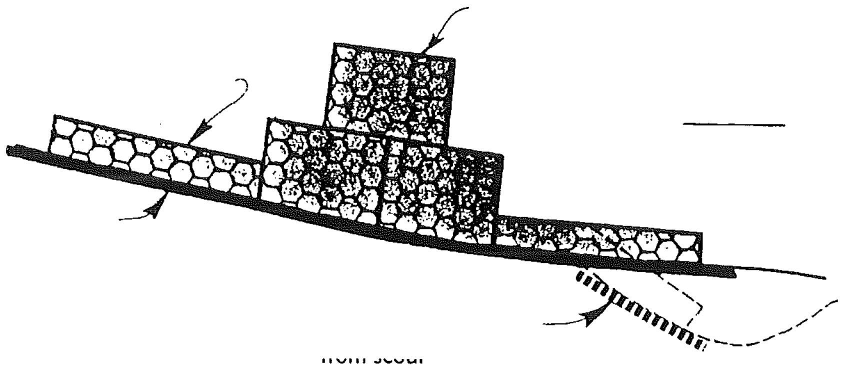
4. PREFABRICATED AND OTHER NOVEL SYSTEMS

There exist a number of other novel and/or low cost materials and methods for shore protection (gabions and stone mattresses, open stone asphalt (Fixtone), used tire pile breakwaters, sheet pile structures, standing concrete pipes filled with granular materials, concrete Z-wall (zigzag) as breakwater, geotextiles curtains (screens), natural and mechanical drainage of beaches, and various floating breakwaters (incl. using tires), etc. Most of them are extensively evaluated and documented (US Army, 1981, Hydraulics Research, 1984, Maccaferri guidelines (Agostini et al, 1981), PIANC publications, Pilarczyk & Zeidler, 1996. etc.). However, more recently, a new family of prefabricated concrete elements as SURGEBREAKER offshore reef system, BEACHSAVER reef, and WAVEblock elements have been developed. Some of them are briefly reviewed below.





6a-perched beach with vegetation and gabion revetment



6b_gabion breakwater section

Figure 6 Typical forms and sizes of gabions and applications

Gabions (Recommended Wave Height Range in respect to direct wave attack: below 1.5 m)

Gabions are rectangular baskets or mattresses made of galvanized, and sometimes PVC_coated, steel wire, in a hexagonal mesh (Figure 6) subdivided into approximately equal sized cells, standard gabion baskets are 1 m wide, and available in lengths of 2, 3 and 4 m and heights of 0.5 and 1 m. Standard mattresses are 0.17, 0.23 and 0.3 m thick, 2 m wide and 6 m long. On request other sizes are also possible. Nominal mesh sizes are from 50 to 100 mm.

At the job site, the baskets are unfolded and assembled by lacing the edges together with steel wire. The individual baskets are then wired together and filled with stones of diameter larger than the mesh size. The lids are finally closed and laced to the baskets, forming a large, heavy mass. One advantage of a gabion structure is that it can be built without heavy equipment. Gabions are flexible and can maintain their function even if the foundation settles. They can be repaired by opening the baskets, refilling them, and then wiring them shut again.

The disadvantage of a gabion structure is that the baskets may be opened by wave action. Also, since structural performance depends on the continuity of the wire mesh, abrasion and damage to the PVC coating can lead to rapid corrosion of the wire and failure of the baskets. For that reason, the baskets should be tightly packed to minimize movement of the interior stone and subsequent damage to the wire. Rusted and broken wire baskets also pose a safety hazard. Gabion structures require periodic inspections so that repairs are made before serious damage occurs. To insure best performance, use properly sized filler rock. Interior liners or sandbags to contain smaller sized material are not recommended. The baskets should be filled tightly to prevent movement of the stone and they should be refilled as necessary to maintain tight packing.

Gabions should not be used where bombardment by water_borne debris or coarse sediment is present, or where foot traffic across them is expected. Stability of gabions/mattresses can be calculated accordingly to criteria mentioned in (Pilarczyk, 1990, Klein Breteler & Pilarczyk, 1997). Some examples of gabion application are given in Figure 6.

The groin or breakwater should be underlain with filter cloth to inhibit settlement and all baskets should be made from PVC_coated wire mesh. A gabion mattress should be provided around the structure to protect against scour. Tight packing of the stone is particularly important to avoid large distortion of the baskets under wave action. Tiers of baskets should be tied together with appropriately sized wire to prevent shifting of upper tiers over lower tiers. A typical cross section a gabion breakwater is shown in Figure 6b.

Surgebreaker (US Army, 1981, Kakuris, 1983)

A Surgebreaker is a modular device constructed with weights about 4000 pounds (2 tons), precast, reinforced concrete modules with vent holes to release wave pressure buildup. The patented triangular modules are 4 feet (1.2 m) high and long, and 6 feet (1.8 m) wide. They are designed to be placed side_by_side (typically in water of 1 to 2.5 m depth) on the existing bottom with the flatter sloped face of the device toward the waves (Figure 7). Installation must be performed by a specialized contractor (these units are often placed by helicopters).

Surgebreakers, with sloped front and back profile and tapered openings, are intended to absorb enough wave energy to prevent erosion of the shoreface, but allow enough passage of energy to transport sediment to the shoreline area; the energy transfer with this pervious sill design should assureproper wave overtopping and proper control of the flow through the enclosed passageways. It is also expected that such permeability will eliminate most of the sinking and scouring that often undermine a solid-faced structure by removing sediments, and will create accretion both seaward and landward of the Surgebreaker. At the number of locations at the United States this system was installed at the end of 70's and in the 80's (i.e. the Gulf Coast of Florida, Lake Forest and other locations in Illinois) and proved to work satisfactorily under mild to moderate wave conditions ($H <= 1.5$ m). However, the installation of this system in Honolulu (Hawaii) has failed because of more extreme wave conditions and lost of foundation stability due to scour. More specific informations on performance of this system can be provided by the US Army Corps of Engineers.

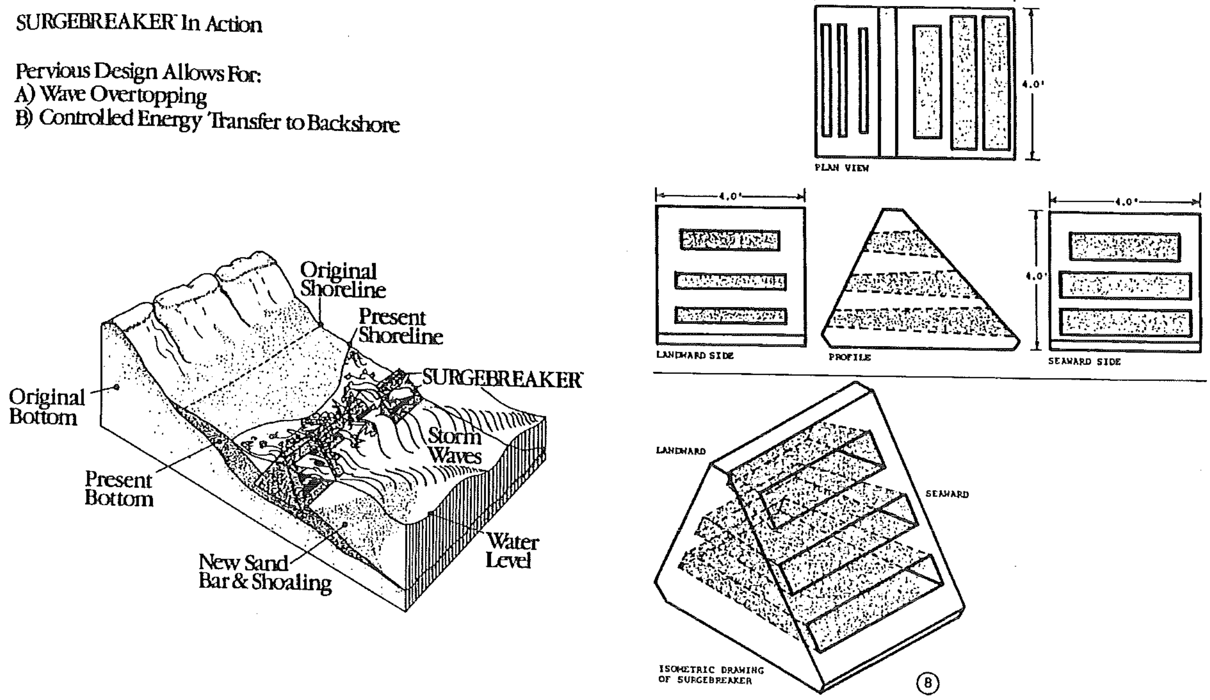


Figure 7 SURGEBREAKER

Beachsaver reef (Info Breakwaters International, 1993)
The Beachsever is a patented shoreline protection device which works in conjunction with natural wave action to reduce the rate of beach erosion and extend the life span of beach replenishment projects. The reef is a derivation of a simple emergent breakwater design, positioned parallel to the shoreline and submerged below the level of mean low tide by approximately six feet (1.8 m). The reef is modular and can be extended to varying lengths, depending upon the beach which requires protection. Each module is 10 feet (3 m) long (alongshore) by 15 ft. (4.5 m) wide by 6 ft. (1.8 m) high (see Figure 8). It is made of precast concrete and each module weighs approximately 21 tons. This device may act as a natural reef providing a protected breeding ground for marine organisms within its internal cavity.

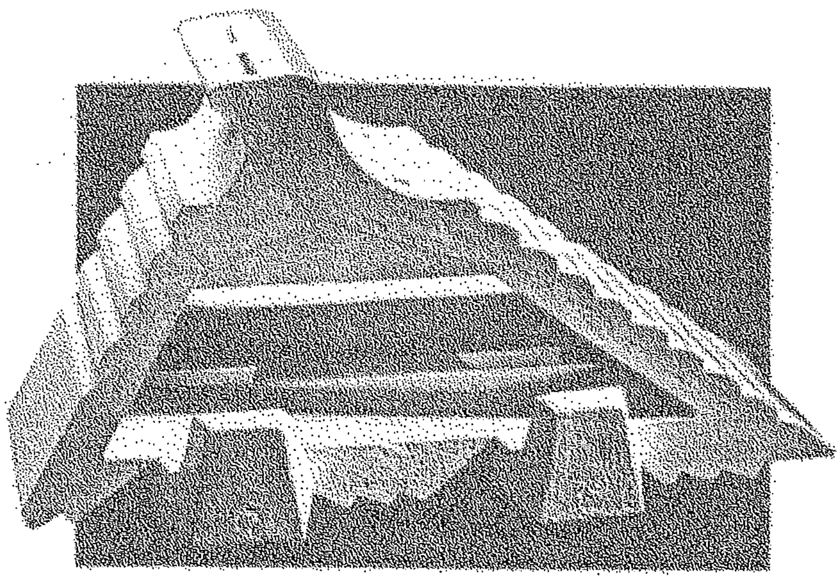


Figure 8 BEACHSAVER element

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The **use** of **submerged narrow-crested breakwaters** for shoreline erosion control.
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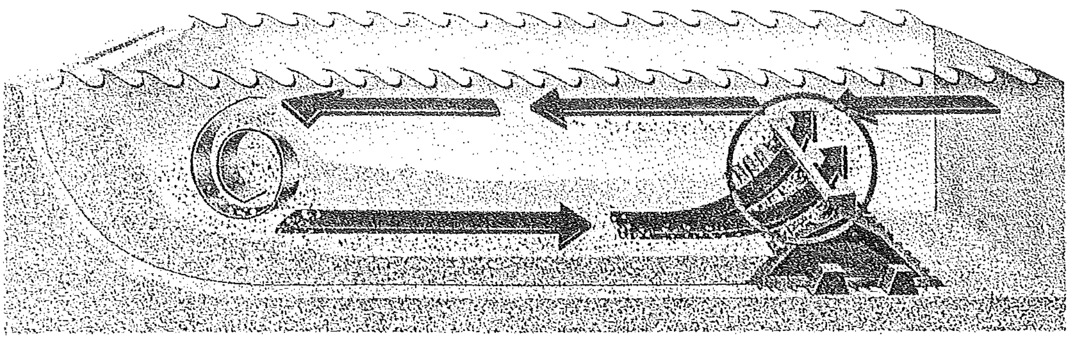


Figure 9 Principles of BEACHSAVER reef

The cross section of the reef module includes several notable features (Figure 9). The modules interlock by a mortise-tenton joint which prohibits lateral, vertical and horizontal movement. The ridged, seaward sloping face of structure dissipates up to 30% of the incident wave energy while avoiding the production of significant reflected wave energy. This feature, in addition to submergence, greatly reduces both the wave-induced force on the structure and the amount of scouring at the seaward toe. The patented system of ridges effectively eliminates toe scour by returning small amounts of sand with each wave to the toe of the structure. The crest of the structure is curved upward on the landward side so as to channel wave-induced return flow vertically toward horizontal openings along the top of the reef. This configuration, known as the "backwash flume", is designed to direct a vertical jet of water upward during the return flow. This return flow is sediment-laden during storms events. The induced "curtain" of water inhibits the offshore migration of the beach berm during storms, essentially cycling the sand back onto the beach.
Testing performed at the Stevens Institute of Technology in New Jersey in Spring of 1992 indicated that the reef limited the offshore movement of sand during periods of forceful waves and increased the likelihood of sand returning to the beach during periods of calm. The system utilizes its submerged weight, low center of gravity and wide, saw-toothed bottom for resistance to overturning and sliding. During wave tank tests performed at the University of Delaware in 1991, it was found that wave forces greater than those found in nature were required to overturn the reef.

Recent experience with Beachsaver system is discussed by Herrington & Bruno (1998) and Stauble, D.K., and Tabar, J.R. (2003). Three submerged breakwater structures were placed as a pilot project along the New Jersey coastline (US) in 1993/94. The effectiveness and structural integrity of each submerged breakwater was assessed over a four year monitoring program. Over the course of the monitoring period, the breakwaters were exposed to over 20 storms with nearshore wave heights greater than 3 m and 10 large swell events generated by hurricanes moving past the coast. The beach profile evolution inshore of all three submerged breakwaters consistently indicated a pronounced scour zone within 15 m of the inshore face of the structure. The maximum measured depth of scour approached 2.2 m but averaged approximately 1.0 m. Due to the large width and depth of the scour zone, the 1.3 m wide geotextile mattresses were ineffective in providing scour protection at the three sites.
The presence of a pronounced scour zone within 10 mmetres of all three of the breakwaters suggests that the structure-induced vertical current is strong enough to suspend sediment immediately inshore of the structure, causing significant settlement. Structure settlement in turn causes an increase in the negative freeboard of the structure, increasing wave transmission and reducing the effectiveness of the structure.
Note: another concept of a prefabricated submerged reef has also been developed in Germany under the patented name "HENKELRIFF" (Henkel, 1990). However, there is no much practical experience with this system.

Prefabricated Erosion Prevention (P.E.P.) reef (Browder et al, 1996, Smith et al, 1998)
The P.E.P. reef is composed of interlocking pre-cast reinforced concrete units, each of which weighs

25 metric tons, and measures 4.6 m (cross-shore) * 3.7 m (alongshore), * 1.8 m (height), Figure 10.

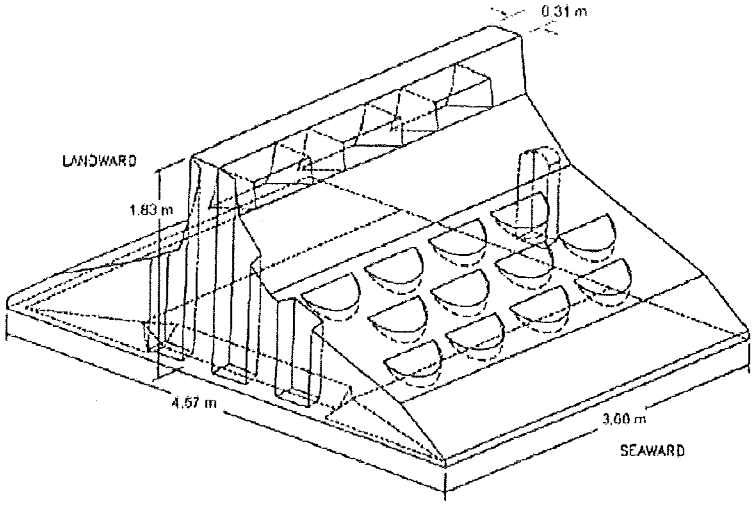


Figure 10 Cross-section of P.E.P. reef unit

The design and performance experience with this narrow-crested submerged breakwater, installed as an erosion and wave energy reduction measure, is based on the model tests and two prototype installations. The first installation took place on the eroded coast at Palm Beach, Florida, in 1992/93 (Browder et al, 1996). The second one was installed also in Florida, at City of Vero Beach, in 1996 (Smith et al, 1998). To evaluate the effectiveness of this type of breakwater both locations were monitored for a three-year period. Reef reduced incident wave heights up to 15%. At the Palm Beach, the submerged breakwater was found to have increased the longshore currents via ponding of water trapped behind the breakwater, which was then diverged alongshore. The shoreline erosion in the lee of the reef was sufficiently severe to warrant removal of the structure in August 1995. In case of the Vero Beach the effect is more positive. However, following settlement of the breakwater, wave height reduction decreased from 12% after initial installation to about 9%. It has been concluded that reef is no longer effective in attenuating wave energy landward of the structure. Future planned assessements include investigating current patterns to document the effect of the breakwater on inducing longshore and cross-shore rip currents.

WAVEBLOCK (Haras et al 1993, ADI Limited 1991, Ortech, 1988)
Waveblock is a steel reinforced concrete modularized structure designed to dissipate the energy of waves. This patented modularized structure, which weights is about 4 tons, consists of horizontal platforms interconnected with vertical columns equally-spaced and arranged in a checkerboard pattern (Figure 11). The actual unit measures 1.22 * 1.83 * 2.44 m, however, the dimensions and weight of the units can be adjusted to allow for different degrees of exposure to wave action and they can be manufactured locally using local materials and labour.

The Waveblock system has been in operation on Lake Huron since 1987. It was designed to dissipate the energy of incoming waves while simultaneously retaining most of the suspended sediment load carried out by these waves. A test section of 40 units was installed (48.8 m) parallelly to the shoreline, in approximately 1.0 m depth of water. The monitoring works have proved that the structure effectively absorbs wave energy, effectively acumulates sand on its landside to generate a stable beach configuration, and effectively accumulates sand on its offshore side while ont impending littoral drift process. These units are now firmly embedded in the sand beach.
It was also obsered that the units themselves were in good structural condition (a few fragments of the concrete slabs were broken probably by ice), there was very little evidence of abrasion, and that some blocks were out of alignment by as much as 0.25 m.

The most promising situations for application of the Waveblock seems to be sandy coastlines experiencing loss of sand under wave induced littoral drift, with modest tidal range and wave climate.

If the project is a small one, where risk exposure is acceptable, a full scale installatio might be undertaken after a visual site assessment by a coastal engineer. On the other hand, where a major installation is contemplated, good engineering practice would call for a comprehensive technical assessment with scale model tests, or limited full scale tests, prior to implementation of the complete project. Fluctuation in water level as a result of tidal action, or over longer cycles, must be taken into account in planning and design of a system (ADI Limited, 1991).

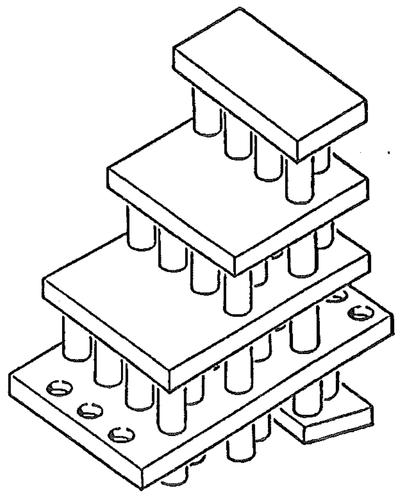


Figure 11 Schematic illustration of a WAVEBLOCK unit

The recent evaluation of performance of prefabricated, narrow-crested breakwaters can be found in (Stauble, D.K., and Tabar, J.R., 2003) and US Army websites..

Distorted ripple mat
A new concept for creating shore accretion is actually developed and applied in Japan. A distorted (precast concrete blocks) ripple mat (DRIM) laid in the surf zone induces a landward bottom current providing accretion of a shore, see Figure 12 (Irie et al, 1994, 1998, Nobuyuki Ono et al, 2004).

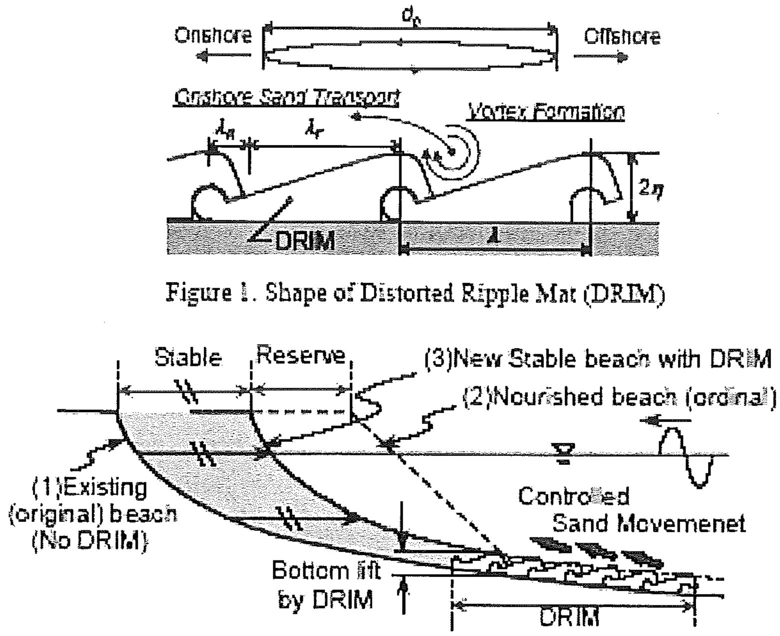


Figure 12 Principle of distorted ripple mat and application

The strong asymmetry of (artificial) ripple profile generates current near the bottom to one direction and thus sediment movement, whose concentration is high near the bottom, can be controlled with only very little environmental impact. The hydraulic condition on which the distorted ripple mat can control the sediment transport most effectively is studied experimentally and numerically and its capability to retain beach sand is tested through laboratory experiments (Figure 18) and field installation.

The definite onshore sediment movement by the control of DRIM is expected if the relative wave height H/h is less than 0.5, where H is the wave height and h is the water depth. The optimum condition for the efficient performance of DRIM is that do/λ»1.7, where do is the orbital diameter of water particle and λ is the pitch length of DRIM, and this condition coincides with the condition in which natural sand ripples grow steadily. DRIM is able to control bottom currents so long as wave direction is within 50 degrees from the direction normal to the crest line of ripples.

Another concept with inclined multiple bottom blades is discussed in (Nobuoka et al, 1996).

Beach drainage (dewatering)systems

Beach watertable drainage is thought to enhance sand deposition on wave uprush while diminishing erosion on wave backwash (Figure 13). The net result is an increase in subaerial beach volume in the area of the drain. The larger prototype drainage by pumping istallations used in Denmark and Florida suggest that beach aggradation may be artificially induced by beach watertable drainage. The state of the art of this technique is presented in (Vesterby, 1996).

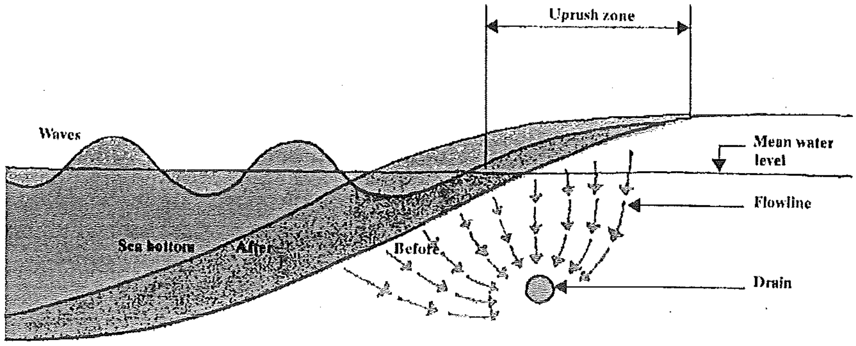
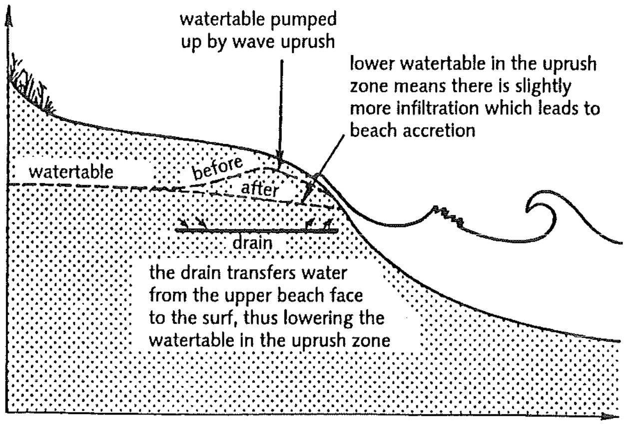


Figure 13 Principles of beach drainage



The idea to achieve lowering of the watertable without pumps by enhancing the beach's own drainage capacity or hydraulic conductivity through the use of strip drains has been applied in Australia (Figure 14), (Davis et al, 1992) and in Japan (Katoh et al, 1994). However, these new techniques are still in rudimentary stage and much more research and practical experience is still needed before application of these systems on larger scale.

Figure 14 Effect of the strip drain on the watertable

5 CONCLUSIONS

* An objective of this review is to show that a large variety of reasonable "low cost" alternatives for shore protection are available. That does not necessarily mean they are "cheap". In fact, practically any properly implemented shore protection method is expensive. The term "low cost" simply means that the various measures are commensurate with the value of property being protected and they are among the lower priced options available. Whether a solution is considered a low cost alternative or not, however, is up to the individual or community installing it. The cost of the project must be weighed against both the objective and subjective value of what is being lost to erosion.

* The methods described here are usually appropriate for use in sheltered waters with mild wave climate. However, a number of solutions/methods have also potential possibilites for application in more exposed (open coast) situations. Use of these structures in such areas is only recomended when based on the detail studies and proper stability calculations supported by reliable model investigation.

* Geotubes and geocontainers offer the advantages of simplicity in placement and constructability, cost effectiveness, and minimal impact on the environment. When applying geotubes and geocontainers the major design considerations/problems are related to the integrity of the units during release and impact (impact resistance, seam strength, burst, abrasion, durability etc.), the

accuracy of placement on the bottom (especially at large depths), and the stability. When applying this technology the manufacturer's specifications should be followed. The installation needs an experienced contractor.

* The technologies related to geotextile systems have been utilized extensively in Europe, Northern America, Mexico, Japan and Australia, producing often successful installations but only few technical details. Technically the methodologies have shown to be feasible but there are design and constructibility uncertainties that still must be addressed. Therefore, further improvement of design methods and more practical experience under various loading conditions is still needed.

* A number of weak points of above reviewed systems can be omitted when the actual knowledge/experience will be applied in the design and technological improving of these systems including such aspects as fabric choice, fabric coating, filling method, installation techniques, stability criteria, and life-time.

* The intention of this literature search is to uncover, as far as possible, the technical informations on these systems and make them available for the potential users. It will help to make a proper choice for specific problems/projects and it will stimulate the further developments in this field.

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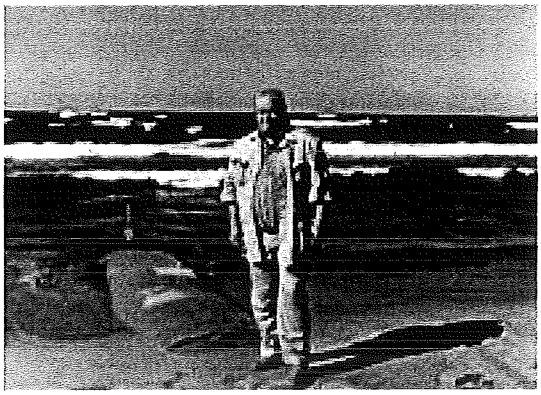
Appendix A1: Holmberg Technologies/**Undercurrent Stabilizers (see also CPNS)**
(the principles of the system are not very clear (confidentional) but it seems to work)



http://www.erosion.com/index.asp

UNDERCURRENT STABILIZER TECHNOLOGY; description in (http://www.vadose.net/beach.html)

Undercurrent Stabilizer Technology is a sediment management system designed to imitated beneficial geology for several types of eroding coastlines. This technology neutralizes the impacts dredged channels have on sandy beaches. This technology has proven to routinely reverse unnatural erosion by D. L. Holmberg (Holmberg Technologies, Inc. 1995). This technology consists of modular, hydrodynamically shaped forms which are cast in place on the nearshore seabed, generally at right angles to the shoreline. Problems normally associated with erosion control structures are avoided through tapered, low-relief shaping, special landward tiebacks and patented filter fabric foundations. The low profile, flow-through array progressively baffles unnatural nearshore turbulence, allowing sand to fall out of suspension in quieted waters within and adjacent to the treated area. Local shoaling progressively decreases remaining nearshore turbulence. As the nearshore progressively shallows out in response to the accretion template, the treated area gets less and less wave and current energy which drives arriving sand away from the treated area. Upon treatment, unnatural erosion ceases and resedimentation begins, often with surprising speed. The accretion template itself is generally buried by rising sand levels as the nearshore beach profile becomes inherently accretional. Adjacent shorelines benefit because an unbounded feeder beach is established. A long-term university study of numerous installations concludes: "Consistent profile volume gain measured in the vicinity of the Undercurrent Stabilizer system plus significant foreshore and backshore beach accretion with no apparent negative impact down drift must be viewed as success in almost any context." Since beach systems are largely governed by non-linear processes, another way of viewing Undercurrent Stabilizer performance is in the language of non-linear dynamics. Self-reinforcing feedback loops are a central element of non-linear systems. Small changes in such systems may "snowball" into counter-intuitively large changes. The weather is a well-known non-linear system. A squall in Africa, for example, can amplify into an Atlantic hurricane through positive feedback mechanisms in the atmosphere. Where coastal systems are concerned, the more a beach profile erodes, the more prone it becomes to further erosion. Deeper water supports greater wave and current energy which increases erosion, further deepening the nearshore, etc. This is the condition of most shorelines today. On the other hand, the more the beach profile accretes, the more efficient it becomes at fostering further accretion.

http://www.erosion.com/index.asp

FREQUENTLY ASKED QUESTIONS

What do Undercurrent Stabilizers look like?

The structures are low profile geotextile tubes that run at right angles from the dune or toe of the bluff, across the beach face to an appropriate distance offshore. Most of the system is submerged. Special foundations and landward tie-ins are employed to eliminate hydrodynamic problems associated with conventional structures.

Are they groins?

Although Stabilizers are perpendicular to the shoreline, the system is designed to baffle nearshore energy without disrupting longshore flows of sand. The system is therefore designed to not trap sand (and divert nearshore energy offshore) as do groin fields. Where groins generally fill in to about half their seaward length or less while causing commensurate deficits of sand downcurrent from the groin, a Stabilizer field has a significantly different effect on sedimentary dynamics. Shorelines downcurrent of Stabilizers are generally the first to benefit with accretion. As the system matures, area-wide accretions occur upcurrent, downcurrent and offshore. Eventually, the structures are buried by rising sand levels as an accretion profile (and feeder beach) is established in response to the accretion template. (see Coastal Restoration Technology.)

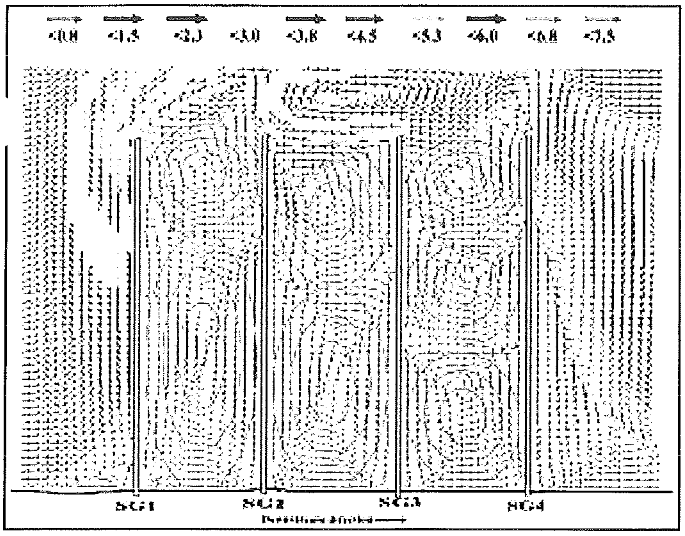
Do Stabilizers need to be placed along an entire shoreline?

Conventional groins, once placed on a shoreline, must generally be installed along the entire shoreline, every several ndred feet, to avoid creating downcurrent problems on untreated sections. A Stabilizer field usually consists of four units spaced about 125 apart, covering approximately 400 feet of coast. Because a single Stabilizer field generally benefits several thousand feet of shoreline or more, Stabilizer fields may be widely spaced along a coast.

Must Stabilizers be prefilled?

Because conventional groins cause downcurrent deficits, groin compartments must generally be prefilled with artificial fill upon construction. The idea is to add sand to the nearshore system in such a way that it will flow around the groins and help to mitigate downcurrent deficits caused by groins. Since groin compartments lose a substantial amount of the fill placed in them over time, an ongoing maintenance program is required to periodically refill the compartments with imported sediment (from inland or offshore sources). Undercurrent Stabilizers have proven to require neither prefilling nor a sediment maintenance program.

See also: Coastal protection and Nourishment System (CPNS), (Goudas *et al.*, 2003)
http://groups.msn.com/softshoreprotectionconfproceedings/_homepage.msnw?pgmarket=en-us



GOUDAS C.L., KATSIARIS G.A., LABEAS G., KARAHALIOS G., PNEVMATIKOS G. (2003a): Soft Protection using Submerged Groin Arrangements: Dynamic Analysis of System Stability and Review of Application Impacts. In: GOUDAS C.L. *et al.* (eds.), *Soft Shore Protection*, Kluwer Academic Publishers, The Netherlands, 227-260.
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http://www.erosion.com/shorepress.html

Holmberg Technologies Affordable Beach Protection's Long-Term Effectiveness Confirmed

PATRAS, Greece, April 16 /PRNewswire/-- Professor Constantine Goudas of the University of Patras has just released the results of the 1st International Soft Shore Protection Conference held in October of 2000. The results confirm the long-term effectiveness of the methods used by Holmberg Technologies, Inc. in halting erosion and storing the shoreline. These further confirm Holmberg's environmental soundness in working with natural forces to protect shoreline habitat without causing erosion to adjacent shorelines.

Holmberg's "Undercurrent Stabilizers" successfully reduce wave energy and current velocities forcing the precipitation of indigenous sediment onto the shore. This reduction in velocity and hydraulic energy also reduces transport of sediment off the shore. Holmberg's system follows the slope of the beach and nearshore soon disappearing under the accreted beachface. It both widens and elevates nearshore shoals and backshore dunes to form. Professor Goudas supports such innovative concepts as Holmberg saying he is "convinced about the usefulness and effectiveness of systems which control only seabed currents." These "soft arrangements" are "physically and environmentally superior" to "hard" protection (seawalls, jetties, etc.) and to beach nourishment. Holmberg Technologies offers a permanent, affordable and environmentally sound alternative. The one-time installation of Holmberg Technologies' Stabilizers prevents the environmental damage and saves the millions of tax dollars required for future renourishments. These tax dollars would then be available to improve and expand coastal communities' infrastructure -- clean water, sanitary sewer, storm drainage systems and wetlands protection -- to protect both public health and the environment.

More information on the 1st International Soft Shore Protection Conference is available on-line at http://communities.msn.com/SOFTSHOREPROTECTIONCONFPROCEEDINGS .

Appendix A2: BeachBuilder (just as an idea, similar but opposite to BEROSIN)

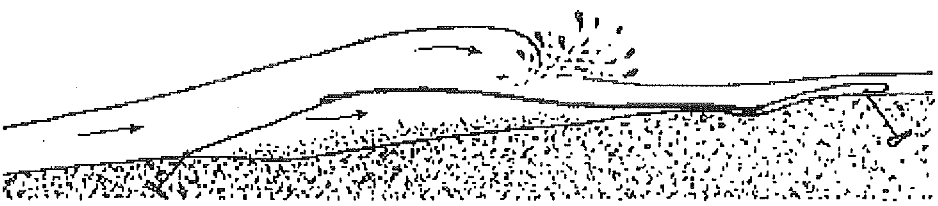
http://www.beachbuilder.com/home.htm

SCHEMATIC OPERATION OF THE BEACHBUILDER TECHNIQUE

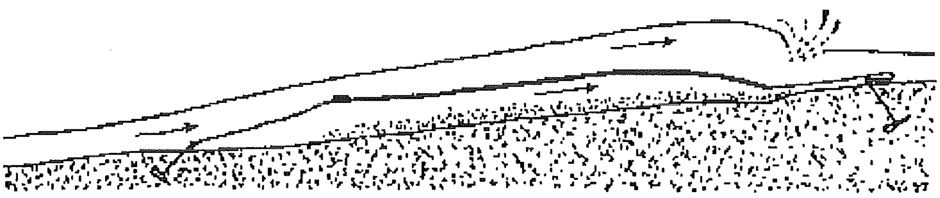
The Beachbuilder uses the Flow-Control Sheet to control erosive waves to build up the beach, rather than erode it.

The erosive action of winter waves is...

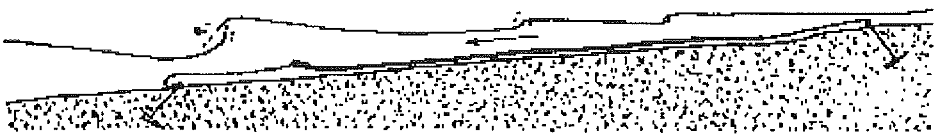
...largely deflected by the Flow-Control Sheet.



Sand moves incrementally toward the shore,



...and the FCS captures incoming sand, and stops seaward loss of the sand with every wave.



The backwash does not carry sand back out to sea.

Beach Builder Technology

Appendix B

ALTERNATIVE SHORELINE STABILIZATION DEVICES

http://www.env.duke.edu/psds/docs.htm

PSDS Documents and Reports

ʿoastal Engineering Models

PREDICTING THE BEHAVIOR OF BEACHES: ALTERNATIVES TO MODELS
Orrin H. Pilkey, Robert S. Young, David M Bush and E. Robert Thieler
LITTORAL 94, September 26-29, 1994 Lisbon-Portugal
[Abstract (PDF/80KB)] [Full Text (PDF/962KB)]

The Use of Mathematical Models to Predict Beach Behavior for U.S. Coastal Engineering: A Critical Review
E. Robert Thieler, Orrin H. Pilkey, Jr., Robert S. Young, David M. Bush, and Fei Chai
Journal of Coastal Research, 16(1):48-70, Royal Palm Beach, Florida, Winter 2000
[Abstract] [Full Text (PDF/5031KB)] [Editorial (PDF/350KB)]

A Discussion of the Generalized Model for Simulating Shoreline Change (GENESIS)
Robert S. Young, Orrin H. Pilkey, David M. Bush, and E. Robert Thieler
ɔurnal of Coastal Research, 11(3):875-886, Fort Lauderdale, Florida, Summer 1995
[Abstract (PDF/77KB)] [Full Text (PDF/2237KB)]

The Concept of Shoreface Profile of Equilibrium: A Critical Review
Orrin H. Pilkey, Robert S. Young, Stanley R. Riggs, A.W. Sam Smith, Huiyan Wu and Walter D. Pilkey
Journal of Coastal Research, 9(1):255-278, Fort Lauderdale, Florida, Winter 1993
[Abstract (PDF/53KB)] [Full Text (PDF/4720KB)][Discussion (PDF/524KB)][Reply (PDF/242KB)]

Beach Nourishment

An Analysis of Replenished Beach Design Parameters on U.S. East Coast Barrier Islands
Lynn Leonard, Tonya Clayton and Orrin Pilkey
Journal of Coastal Research, 6(1):15-36, Fort Lauderdale, Florida, Winter 1990
[Abstract (PDF/35KB)] [Full Text (PDF/2588KB)]

A Comparison of Beach Replenishment on the U.S. Atlantic, Pacific, and Gulf Coasts
Lynn A. Leonard, Katharine L. Dixon and Orrin H. PIlkey
Journal of Coastal Research, SI #6, 127-140, Fort Lauderdale, Florida, Summer 1990
[Abstract (PDF/56KB)] [Full Text (PDF/1557KB)]

Comparison of Beach Nourishment along the U.S. Atlantic, Great Lakes, Gulf of Mexico, and New England Shorelines
Arthur C. Trembanis, Orrin H. Pilkey, Hugo R. Valverde
Coastal Management, 27: 329-340, 1999
[Abstract (PDF/53KB)] [Full Text (PDF/1229KB)]

Summary of Beach Nourishment Episodes on the U.S. East Coast Barrier Islands
Hugo R. Valverde, Arthur C. Trembanis, and Orrin H. Pilkey
Journal of Coastal Research, 15(4):1100-1118, Royal Palm Beach, Florida, Fall 1999
[Abstract] [Full Text (PDF/3750KB)]

Summary of the New England Beach Nourishment Experience (1935-1996)
Tonya C. Haddad and Orrin H. Pilkey
Journal of Coastal Research, 14(4):1395-1404, Royal Palm Beach, Florida, Fall 1998
[Abstract (PDF/40KB)] [Full Text (PDF/1842KB)]

Summary of Beach Nourishment Activity Along the Great Lakes' Shoreline 1955-1996
Michael K. O'Brien, Hugo R. Valverde, Arthur C. Trembanis, and Tanya C. Haddad
Journal of Coastal Research, 15(1):206-219, Royal Palm Beach, Florida, Winter 1999
[Abstract (PDF/39KB)] [Full Text (PDF/2084KB)]

Summary of Beach Nourishment along the U.S. Gulf of Mexico Shoreline
Arthur C. Trembanis and Orrin H. Pilkey
Journal of Coastal Research, 14(2):407-417, Royal Palm Beach, Florida, Spring 1998
[Abstract (PDF/31KB)] [Full Text (PDF/2125KB)]

Beach Replenishment Activities on U.S. Continental Pacific Coast
T.D. Clayton
Journal of Coastal Research, 7(2):1195-1210, Fort Lauderdale, Florida, Fall 1991
[Full Text (PDF/1539KB)]

Coastal Hazard and Mitigation

Reducing Vulnerability in Five North Carolina Communities: A Model Approach for Identifying, Mapping and Mitigating Coastal Hazards
Andrew S. Coburn

North Carolina Hazard Mitigation Planning Initiative, August 1991
[Full Text (PDF/5400KB)]

Quantitative Evaluation of Coastal Geomorphological Changes in South Carolina After Hurricane Hugo
E. Robert Thieler and Robert S. Young
Journal of Coastal Research, Special Issue, No.8 pp.187-200, Fort Lauderdale, Florida, Spring 1991
[Abstract (PDF/54KB)] [Full Text (PDF/1898KB)]

Mitigation of Hurricane Property Damage on Barrier Islands: A Geological View
David M. Bush and Orrin Pilkey
Journal of Coastal Research, Special Issue No.12: Coastal Hazards, pp.311-326
[Abstract (PDF/44KB)] [Full Text

Hard Stabilization

Effects of Hard Stabilization on Dry Beach Width for New Jersey
Mary Jo Hall, and Orrin H. Pilkey
Journal of Coastal Research, 7(3):771-785, Fort Lauderdale, Florida, Summer 1991
[Abstract (PDF/55KB)] [Full Text (PDF/4157KB)]

Seawalls Versus Beaches
Orrin H. Pilkey and Howard L. Wright III
Journal of Coastal Research, Special Issue No.4: pp.41-64, Charlottesville, Virginia, Autumn 1988
[Abstract (PDF/39KB)] [Full Text (PDF/3626KB)]

ALTERNATIVE SHORELINE STABILIZATION DEVICES

Disclaimer

Traditional methods of stabilization - seawalls, groins and breakwaters - are increasingly being identified as leading to the erosion of fronting and downdrift beaches. For the last thirty years, coastal homeowners and town managers have sought low-cost, effective alternatives. In response, a number of "non-traditional" devices with optimistic sounding names have appeared on the market. The far-reaching manufacturer's claims have led these alternatives to be coined "snake oil devices."

The following is a listing and brief description of many alternative devices. Also included is a qualitative analysis outlining the potential negative impacts of each device. It is important to note that few of these devices have been sufficiently field tested (five years or more to accurately assess the ability to withstand severe storms) while some have never been field tested at all.

For further information, or for any questions, please contact Duke PSDS

POTENTIAL NEGATIVE IMPACTS

As an extension of the problems previously listed, devices were assessed as to associated negative impacts. This analysis is based on media reports, first-hand accounts and our own personal experience with shoreline devices and coastal processes. The following negative impacts were evaluated for each device:

- **Reduces Beach Access**: This applies to any device placed on the beach that restricts lateral or cross-shore access, access to the water, or restricts swimming areas.
- **Erosion of downdrift beaches** : Devices that restrict transport of sand, either cross-shore or alongshore will lead to erosion elsewhere.
- **Erosion of fronting beaches** : Applies to devices that cause wave reflection, that prevent landward migration of beaches, or which cut off the local sediment supply.
- **Potential hazard to swimmers** : A device placed in the shallow nearshore zone poses a threat to swimmers. Devices that alter currents or flow present a possible threat, especially when used in conjunction with a groin. Water that flows onshore must flow offshore. When offshore flow is impeded, water will flow along shore until it can be released, creating a separate longshore current. When a breakwater is placed near a groin there is usually a space between the two. This is the preferred path for water flow, and leads to the Venturi effect. Water that flows through a narrow, straight opening, will create an undercurrent suction, similar to that of a rip current. This could result in swimmers being pulled offshore.
- **Impact to water quality**: A key factor in preserving water quality is dilution of pollution through circulation. A reduction in the flow of water, reduces the circulation and can effect water quality. This category applies to those devices that impede water flow, or that trap floating debris.
- **Impacts turtle nesting**: Not only are devices that impede the path of turtles across the shore a concern, but more important, are those that may alter the temperature of the sand. The sex of turtles is determined by the temperature at which they are incubated. Devices that effect the temperature of the soil may cause overselection for one sex.
- **Impacts on clam resource**: Of great concern, especially on the west coast, is the clamming industry. Any device placed on the nearshore, or at the low tide line may impact the access to the clams and the resource itself by compacting the sand.
- **Impairs aesthetics**: One of the primary reasons for development along the coast is because people are drawn to its natural beauty. Devices that are placed on the beach or above water detract from the scenery and overall aesthetics.

Alternative stabilization devices survey:

- Description of devices
- Potential adverse impacts
- Suggested guidelines for evaluating devices

http://www.env.duke.edu/psds/Stabilization/Categories.htm

Description of Alternative Devices for Shoreline Stabilization

.n attempt was made to contact all companies whose products are listed. Not all were successful and the best available data is listed here. Where no information was available, the installation section is left blank.

Devices Placed in the Water	Devices Placed on the Beach
Breakwaters	Groins
Artificial Seaweed	Dewatering
	Seawalls
	Dune Stabilization
	Other

I. Devices Placed in the Water

A. Breakwaters

˜unction

- Shore parallel structure placed offshore, either submerged or floating.
- Dissipates wave energy by forcing waves to break. This creates a "wave shadow" causing sand deposition.

Associated Problems

- Wave action may cause scour in the vicinity of the device.
- May increase downdrift erosion by removing material from littoral current.
- May impact water quality because of reduced water circulation.
- May endanger swimmers or boaters.

Device	Installation	Manufacturer's Claims

Atlas Shoreline Protection System - Stacked timber, laid horizontally, held together by steel supports. Arranged in a sawtooth pattern on the nearshore, parallel to beach.		• Prohibits erosion & allows for accretior inward & outward of system. • Long life, low maintenance.
Beach Prisms - Concrete locks, with a triangular cross-section. Each unit is 6' high, 12' long and 84" wide with a concave, openwork front face.	• 1988 - Chesapeake Bay	• Openwork face allows more water to flow through, which reduces scour.
Beachsaver reef - Interlocking, concrete units, triangular in cross-section. Each is 10'long, 6' high, 16' wide. The front face is ridged to reduce wave reflection, with a slotted opening at the top.	• 1993 - Avalon, NJ • 1994 - Cape May Pt. & Belmar/ Spring Lake, NJ	• Water flows through slotted openings at top, sand is suspended & carried forward by incoming waves. • Stabilizes beach nourishment, requiring less sand for renourishment. • Attracts wildlife.
Menger Submerged Reef - Triangular in cross-section; welded iron frame covered with steel screen mesh & concrete. Submerged offshore by filling with sand.		• Prevents sand from washing seaward by slowing wave energy • Units can withstand severe weather changes, because the materials expand and contract. • Re-usable; not permanent • Environmentally friendly because it can be moved with ease.
MOTO - Primary function is to harness wave energy but also acts as a breakwater to reduce coastal erosion. Installation- 3 toroids, 10' in diameter weighing 4 tons each, placed at least 20' deep.		• Waves lose power by creating energy, thus reducing erosion. • Provides clea3n energy and fresh water.
Pep Reef- Concrete units, triangular in cross-section, 6' high, weighing 20 tons. Placed 2-4' below surface at low tide.	• 1988- Palm Beach, FL (Privately funded) • 1992-1993 - Palm Beach, FL (*removed in 1995 because of increased erosion) • 1996- Vero Beach, FL	• Builds trough and bar areas beyond the foreshore which shifts the foreshore outward. • Stabilizes the shoreline. • Reduces wave energy 40-70%. • Can be relocated easily. • Shelter and habitat for animals.
Sealift-Shoreline breakwater, triangular in cross-section, place beyond foreshore where it is shallow at low tide. Angled, so as to slow wave energy.	• 1990 - proposed for Palm Beach	• Pollution free installation. • Waves lose much of their destructive power. • Reduces long term erosion.

		• Compresses the configuration of wave cells.
Shoreprotector -Submerged sand fence, place 400' offshore. Made of openwork steel frame with 4 baffles on each side; 7' tall, 16' wide at base, weighing 650 lbs.	• 1975 - Virginia Beach, Va. Removed due to failure. Installation cost: $108,000 Removal cost: approx. $67,700	• Flexible design. • Removable.
Surge breaker -Permanent, steel reinforced "prisms", 4' high, 4' wide and 6' deep; placed in 3-8' deep water. Can be joined by steel cables for higher energy environments. Recommend installing 2 systems parallel to each other.	• 1976- Highlands Park, IL • 1979- Bayou State Park, FL • 1984- Kuala Regional Park, Oahu, HI	• Simulates offshore sandbars and reefs. • Stimulates accretion, reduces erosion. • Will work on any beach. • Will withstand extreme weather. • *In 8 months beach accreted 50' in width in IL.
Temple Beach System - Reinforced concrete, triangular in cross-section, placed at mean low-tide/12-18" below high tide, parallel to the shore. Metal rods are used to anchor.		• Does not interfere with boaters, bathers or turtles. • Mitigates against storm damage. • Moves high & low tide line an average of 200' outward. • Protects beach nourishment.
Waveblock - Modularized, permeable, steel reinforced concrete. Structure is an angled tower.		• Absorbs wave energy before it reaches the shoreline.
Waveshield - Floating system made of steel; each unit is 80' long, 20' wide & 18' high, weighing 40 tons. Unit of 3 compartments. Best in 25-30' deep water.		• Provides protection against wave damage and erosion. • Breaks 8-10' roller waves. • Economical, simple & easy to make. • Can be floated to any location, thereby avoiding high accretion on the landward side.
Wave Wedge - Concrete, interlocking units; triangular in cross-section, weighing 5,000 lbs. Three slots/holes on the front face.	• 1985-Michiana, MI	• Slots on front absorb energy. • Builds up foreshore & sandy beach. • Restores sand lost during storms.

Top of Page

B. Artificial Seaweed and Others

Function

- Low-lying devices that are anchored to the seafloor.
- Designed to slow waves and reduce energy, causing sand carried by waves to be deposited.
- Also designed to slow return wave energy, so that sand carried off the shore by return flow is deposited nearshore.

Associated Problems

- Devices are placed in shallow water and may be hazardous to swimmers and boaters.
- Many lack the weight or design to be suitably anchored and do not withstand storms.
- Creates debris on beach when washed out by storms.

Device	Installation	Manufacturer's Claims
Beach Cones - Concrete donut 6" high, 2' across, 40" across the bottom weighing 92 lbs. each.	• 1992 -Shell Island in Lower Plagumines, LA	• Provide hard bottom stabilization for sand accretion. • No loss during Hurricane Andrew of an installation that included 300 cones and 13-72 cu. yds. of sand. • Average accretion, 6', max gain 3'.
Beach Protector Tire Mat -Tires anchored to each other & to the seafloor in a section 30-60' wide & at least 1 mile long. Can be shorter if between 2 promontories & close to end of one of the promontories.		• Slows the return of "sand laden" waves.
Burns Beach Erosion Device -Concrete Block (5'x 2' x8") with rubber tire strips (1"-2" wide) attached to me top of the block. Acts as artificial seaweed.		• Dissipates wave energy reducing offshore transport of sand. • Allows for greater accretion of sand during storm conditions. • May provide protection for turtles & substrate for crustaceans.
Cegrass - Synthetic seaweed made of foamed polypropylene, attached to open grid mat, held to seafloor by ballasts. The length of the mat is tailored to the environment.	• 1985-Germany, to fix scour caused by pipeline • -Italy • -Wetlands in Europe	• Reduces nearshore current velocities, thereby sand is dropped in sandbars which build up to 1.6m high. • Reduces offshore sand movement and scour.
Coil System - 9-gauge wire, 24-30" in diameter, intertwined with smaller wire, attached to the ocean floor. Installed between inlets, 500' to 2000' from shore, in		• Sand is captured within the coil grid and returned to the shore by tides & wind.

grid system 100' between units, which are placed at an angle to the shore.		• Coils interrupt ocean currents, allowing for sand to be trapped while currents pass through. • If properly emplaced, there will be no sand loss to adjacent beaches.
Seabee-A series of six sided concrete blocks, weighing 35 lbs. to 1 ton, with holes (honeycomb design) placed on slope in the nearshore. 20% of construction material is recycled ash.	• 1989, Tidewater Community College-Portsmouth. Monitored by VIMS.	• 20" of sand and silt collected between 1989 and 1996 on Tidewater Test Site. • Reduces energy of wave run-up, causing sand to be deposited.
Seascape -Synthetic seaweed. Plastic filaments attached to a bag which is filled with sand to anchor3 the device.	• 1981- Cape Hatteras, NC • 1983/1984- Barbados	• Controls shoreline erosion. • Fronds reduce current flow, sand is dropped.

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II. Devices Placed on the Beach

A. Groins

Function

- Perpendicular to the shoreline, placed on the beach extending into the water, or submerged.
- Designed to trap sediment that is moving alongshore in the littoral current.

Associated Problems

- Cause erosion of downdrift beaches.
- May create rip currents that are hazardous to swimmers.
- Nuisance to recreational beach use.

Device	Installation	Manufacturer's Claims
Brush Fence - Christmas trees or discarded lumber laid out in a "crib" fashion.; 4' wide, 72' long.	• Jefferson Park, LA	• Protects the shoreline.
Holmberg Undercurrent Stabilizer - "Interlocked network of geotextile forms injected with concrete." Site specific design with longshore & offshore components laid perpendicular & parallel to the waterline.	• 1982-Manasota Key, FL • 1983-Michigan near Buffalo; Captiva, FL	• Slows currents so that inlets & jetties don't divert sand. Nearshore sand stays nearshore. Sand coming from offshore no longer transported downshore by

Accretion template which builds the submerged nearshore profile.	and Ogden Dunes, IN.	littoral currents, therefore beaches accrete. This induces nearshore shoaling.
Parker Sand Web - Series of fish nets (50-100' apart) perpendicular to the shore, strung from the high tide line, into the water. Nets are made of heavy nylon material. Work similar to a groin, trap suspended sand.	• 1987- Pelican Bay Beach, FL. Had to be removed after 20 days because the installation did not have a permit.	
Shoreline Construction Corp. groin - Low profile sill and groin system. Sill placed at an angle to the shore; acts as an artificial bar. Groin, perpendicular to the shore on either end of the sill & in the middle. The groin directs the flow of the sediment & water & reduces currents.		• System is at or below the water level, so waves can still overtop which eliminates scouring, flanking and reflection. • Eventually the whole system is covered by sand.
Stabilito - Plastic groin/artificial ripple, 5m long, 1.8m wide, 60 cm high; placed perpendicular to shore on a submerged beach or dune.		• Slows & "elevates" currents, thereby creating sand ripples. • Stabilizes coastlines, riverside erosion and dunes.

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B. Seawalls

Function

- A wall placed at the base of a bluff, at edge of shoreline property or at the landward edge of a beach.
- Designed to protect land from the impact of wave energy.

Associated Problems

- Cause both active and passive erosion of the front beach. Cuts off local sediment supply.
- Active: waves that hit are reflected and wash downward, which increases scouring at the toe of the wall.
- Passive: seawalls prevent beaches from migrating landward; a natural response to sea level rise.

Device	Site	Manufacturer's Claims
High Energy Return Wall - Concave seawall that causes wind and water to work against each other, thereby flattening the sea surface. Individual sections are 33' by 44' at base. Wall is 30'. Perforations in "splash pad" allow for water to pass through		• Reduces toe scour common with traditional seawalls. • Causes beach accretion.

sand to be deposited on back side of wall.		
Marine Bin Walls - Steel bin filled with "granular material" to withstand freezing & thawing. Placed at shoreline or base of bluff.	• 1973-Michiana, MI • 1977-Dykesville, WI • 1982-Washburn, WI	• Protects homes. • Best suited for Marine construction.
Ravens Retaining Wall - Aluminum, corrugated retaining wall placed at the water's edge at the base of a bluff.	• unspecified	• Protects property from slippage & erosion by tides.
Wave Buster - Seawall with angled top to reduce wave reflection. Associate drainfield above & behind bluff to reduce hydrostatic pressure. Base secured with geotextile bed.	• Great Lakes-unspecified	• Deflects water up & back without reflecting the waves-reduces toe scour.
Z-wall - Low-lying concrete wall placed in a saw-tooth pattern at the base of a bluff or, ideally, offshore, submerged halfway.	• 1973, Buttersville Park, Luddington, MI	• Reduces erosion & encourages the build-up of sand in front of the wall. • Redirects wave energy so that sand is dropped.

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C. Dewatering

Function

- A drain and pump system extracts water from the beach allowing for more percolation of incoming waves.
- When water percolates through the sand, the sand being carried by the incoming wave is deposited at the surface.
- With dewatering devices, there is a pump placed at the high tide line which withdraws water collected in underbeach drainfield.
- As the groundwater is pumped out, it is funneled to the ocean or collected as a resource.

Associated Problems

- Must be turned off during turtle nesting season because groundwater extraction affects the temperature of the sand.
- Has not proven to withstand storms. In Nantucket, the system broke down during every major storm.
- Swimming is prohibited in front of the installation because the pipes pose a possible hazard.

Device	Installation	Manufacturer's Claims
HDSI - Buried wells extract groundwater, thereby leaving an unsaturated zone. Waves		• Easier to install & more cost

run-up & the water percolates below ground, depositing sand.		effective than traditional dewatering devices. • Not susceptible to storm damage. • Environmentally friendly, even to turtles. • Can be operated at variable rates.
Stabeach - System includes a pump placed on the high tide beach with drain pipes attached. The pipes run underground & discharge into the ocean.	• 1988-Sailfish Pt., FL • 1994-Englewood, FL • 1996-Nantucket, MA	• Builds beaches while reducing erosion- less water washes back to the ocean in return flow, so less sand is carried with it. • Installation causes relatively little disturbance to the beach.

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D. Bluff/Dune Stabilization

Function

- Low-lying barrier placed on the beach to prevent erosion.
- May also aid in trapping wind blown sand to build an artificial dune.

Associated Problems

- Revetments protect only the land behind the structure, therefore the front of the beach may continue to erode.
- May lead to passive erosion in that sand is trapped by the revetments, and the beach is then unable to retreat from a rising sea level.

Device	Installation	Manufacturer's Claims
Biodune Sand Gel - Spray gel-mixture of 97% beach sand & water with non-toxic biodegradable aqueous polymer gel.	• St. Augustine, FL • Melbourne Beach, FL • Ft. Fisher, NC	• Stabilizes dunes. • Doesn't deter marine turtles. • Withstood three years of storms (dunes lost elevations, but were not undercut) • Damage can still be caused by walkover. • Does not impede growth of vegetation.
Dune Guard - Similar to sand fencing but made of polymer grid attached to poles.	• Avalon, NJ	• Captures wind blown sand. • Especially suited for storms.

		• Lasts longer than ordinary sand fencing, partially because it resists weathering. • Can resist 9 ton force.
Fabric Fence - Sand fence made from yarn impregnated & coated with foam vinyl plastic, attached to poles & placed at the high tide mark or base of the dune line. Rolls are 150' long, 46" high.		• Highly visible. • Easy to install. • Stable & weather resistant.
Nicolon Geotubes - textile tube made from woven polyester; 30' in circumference & variable lengths. Bags are filled with sand and placed in a trench at the toe of a dune.	• 1995-Atlantic City, NJ	• Stabilizes dunes & prevents landward erosion. • Can also be used as a groin.
Soukup Rubber Tire Revetment - Tires placed in a 16-18" deep, 15' wide trench, lined with filter cloth on the low-tide dry beach. Tires are covered with the sand that is dug out.		• Tires act as a more stable sandbag. • Stabilize the shoreline behind the revetment.
Subsurface Dune Restoration - A dune is created by burying sandbags on a re-contoured slope. Vegetation is then established to protect the dune.	• Caledon Shores • 1997-Long Island, NY	• Dissipates storm wave energy which reduces erosion. • Designed for a 25 year storm. • Also allows for percolation of waves which builds up sand on the surface.
Triton Marine Mattress - Stone filled mattresses used for bluff or dune stabilization.	• Trinidad • Boston Harbor, MA	• Stabilization of bluffs & dunes. • Protection from scour.

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'. Other

Device	Installation	Manufacturer's Claims
Beachbuilder Technique - Elastomer coated industrial fabric, 25' wide, anchored from the high beach to the tide line. Uses the energy of waves to build the beach (maximum winter buildup) by preventing the removal of sand during wave retreat.		• Restricts the return flow of sand carried by a retreating wave. • "Accretion concentration of 60cu yd/ft in less than 4 days."
Stabler Disks - Concrete disks, 4' in diameter, attached to pilings & placed at the storm high tide line.	• 1993-Spring Lake, NJ • 1996-Myrtle Beach, SC	• Protects beaches & dunes by reducing storm wave energy. • Waves are slowed, sand is dropped & disks are covered.

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Potential Negative Impacts Associated with Alternative Devices

DEVICE	Harms Beach Access	Erosion of downdrift beaches	Erosion of fronting beaches	Potential hazard to swimmers	Impact on water quality	Impact on turtle nesting	Impact on clam resource	Impairs Aesthetics
IN WATER								
Atlas Shoreline Protection System	X	X		X	X			X
Beach Prisms		X		X	X			
Beachsaver Reef		X		X	X			
Menger Submerged Reef		X		X	X			
MOTO		X		X	X			
Pep Reef		X	X	X	X			
Sealift		X		X	X			
Shoreprotector		X		X	X			
Surge breaker		X		X	X			
Temple Beach System	X	X		X	X			X
Waveblock		X		X	X			
Waveshield		X		X	X			X
Wave Wedge		X		X	X			
Beach Cones		X		X	X			
Beach Protector Tire Mat								
Burns Beach Erosion Device		X		X	X			
Cegrass		X		X	X			
Coil System		X		X	X			
Seabee				X				
Seascape		X		X	X			
ON THE BEACH								
Brush Fence		X		X	X			X
Holmberg Undercurrent Stabilizer		X	X	X	X			
Parker Sand Web	X	X						X
Shoreline Construction Corp.		X	X	X	X			
Stabilito		X	X					X
High Energy Return Wall		X	X	X				X
Marine Bin Walls		X	X	X				X
Ravens Retaining Wall		X	X	X				X

Wave Buster		X	X	X			X
Z-wall		X	X	X			X
HDSI	X				X	X	X
Stabeach	X				X	X	X
Biodune Sand Gel			X				
Dune Guard			X				X
Fabric Fence			X				X
Nicolon Geotube			X				X
Soukup Rubber Tire Revetment			X				X
Subsurface Dune Restoration System			X				
Triton Marine Mattress			X				X
Beachbuilder Technique					X	X	X
Stabler Disks	X				X		

Guidelines for Evaluating Alternative Shoreline Stabilization Devices

The following points should be considered when evaluating individual engineering devices:

- Beware of success claims made for distant beaches, i.e. Europe, Australia, or Bermuda.
- Be wary of claims that a single device works on all beaches.
- Every manufacturer touts their device as the one that really works, as opposed to those of their competitors. Think about it.
- Note that before and after photo comparisons of installations are often complicated by changes in tide levels or lake levels.
- Don't believe everything you read. Many reports documenting success are made by the manufacturer or a consultant hired by the manufacturer.
- Don't be lulled by academic research used to support success. Again, look at who provided the funding for the research, the time length of the study and whether the device was subjected to high-energy events.
- Beware of elaborate and pleasant sounding names for devices; they are often misleading.
- Failure of a device is frequently explained away by "unusual" or "unexpected" storms. Such storms are one of the greatest threats to beach width, and devices should be designed to withstand them. Don't accept excuses.
- Wave tank experiments cited as documenting success are meaningless. Documentation should be in the form of the device protecting a beach and property for a significant length of time against the forces of nature.
- "Low-cost" shore protection is an oxymoron.
- In the Great Lakes these new "solutions" come and go with changing lake levels.
- If the device does succeed in holding sand on the beach, it will be at a cost to other beaches; typically the downdrift neighbors.
- Is success of a device accomplished at the cost of loss of the beach?

See also:

http://www.unesco.org/csi/pub/source/ero18.htm

Example of Multi Criteria Analysis
http://www.cne-siar.gov.uk/minch/coastal/coastal1.htm#TopOfPage
The Minch Project; Coastal Erosion In the Western Isles
http://www.cne-siar.gov.uk/minch/coastal/coastal1-06.htm#P1407_126482
Outline for a Shoreline Management Plan

A useful method of assessing management options is a technique known as a Multi-Criteria Analysis. Two sites in the Western Isles are analysed in this way: Management Unit 3 (Aiginis to Melbost Point) and Management Unit 30 (Kirkibost and Baleshare Islands).

This technique allows the assessment of different management options. Criteria which are judged more important than others, for a particular stretch of coast, have this reflected by an appropriate weighting. In the example given for Aiginis, the comparative weighting of criteria for the frontage at Aiginis is given as follows:

	Criterion	A	B	C	D	E	F	Total Weight
	Protection of land	-	2	2	2	2	2	10
B	Modest capital costs	0	-	1	1	2	1	5
C	Low maintenance cost	0	1	-	1	2	1	5
D	Maintain beach levels	0	1	1	-	2	1	5
E F	Public acceptance	0	0	0	0	-	1	1
	Low environmental impact	0	1	1	1	1	-	4

If the criterion in a row is:

- more important than the criterion in the column, then score = 2

qually important as the criterion in the column, score =1

- less important than the criterion in the column, score = 0

Next a list of feasible methods of managing the coastline is drawn up, including the "Do Nothing" option, i.e. to allow the coastline to continue to evolve without (further) intervention. A comparative assessment of the alternative schemes is made by marking how well each scheme is likely to achieve each criterion on a scale of 1 to 10 (where 10 means a very good achievement of the criterion). Finally, for each option, the rankings are multiplied by the weighting assigned to the relevant criterion (from the table above), and a total weighted score evaluated (bottom row).

For the Aiginis example, the comparative assessment of defence alternatives are as follows:

Option Description

Number

1 Do nothing, but rebuild infrastructure at Aiginis further inland

2 Build sea wall to protect eroding cliffs at Aiginis

3 Rock revetment to protect eroding cliffs at Aiginis

4 Offshore breakwaters/sill to protect eroding cliffs at Aiginis

5 Shingle recycling from western end to eastern end of frontage

(Note that there is no "Do Nothing" option here, since the continued erosion of the cliffs at Aiginis would result in serious threats to the infrastructure and possibly the village.)

Criterion	Weight	Alternative Options				
		1	**2**	**3**	**4**	**5**
A Protection of land	10	1	9	8	7	4
B Modest capital cost	5	1	1	3	3	6
C Low maintenance cost	5	1	5	5	5	3
D Maintain beach levels	5	4	2	5	8	8
E Public acceptance	1	3	7	8	7	7
F Low environmental impact	4	7	3	4	4	6
TOTAL WEIGHTED SCORE		71	149	181	173	156

<u>Discussion of alternatives</u>

From the table above the best two of the suggested alternatives for protecting the soft cliffs at Aiginis are ranked almost equally, namely the rock revetment (option 3) and the offshore breakwaters or rock sill (option 4). To separate these alternatives a more detailed analysis, including costing, would have to be conducted.

Related websites

Coastal Page & Links
http://www.coastal.udel.edu/coastal/coastal.html
http://udel.edu/mailman/listinfo/coastal_list
http://coastal.tamug.edu/links.html
http://www.coastal-guide.com/
http://www.coastalguide.org/code/coasdef.html
http://www.usace.army.mil/inet/usace-docs/eng-manuals/cecw.htm
http://www.nap.edu/openbook/0309041430/html/94.html
http://books.nap.edu/books/0309041430/html/related.html
http://lab.nap.edu/nap-cgi/discover.cgi?term=coastal%20protection%20structures%20and%20their&restric=NAP
http://www.lacoast.gov/reports/index.htm
http://www.dbw.ca.gov/csmw/
http://whyfiles.org/091beach/4.html
http://www.unesco.org/csi/pub/source/ero18.htm
ttp://users.coastal.ufl.edu/~sheppard/eoc6430main.htm
http://www.oas.org/cdcm_train/
http://www.poemsinc.org/oceano/beachnew.htm
Technology Transfer:http://www.grida.no/climate/ipcc/tectran/index.htm
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http://www.coastal.crc.org.au/coast2coast2002/proceedings.html
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http://adaptation.nrcan.gc.ca/perspective/toc_e.asp
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http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Publications;8
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http://users.coastal.ufl.edu/~sheppard/eoc6430/Coastal_Engineering_Manual.htm

Geotextile tubes and containers
http://www.geotubes.com/

http://geotecassociates.com/

http://coastal.tamug.edu/am/StudentPowerpointPresentations/Laura_Mullaney_Geotubes_on_Galveston_Island%20ppt/Geotubes_on_Galveston_Island.ppt
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http://www.advancedcoastaltechnology.com/protectperfect.htm
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http://www.scourcontrol.co.uk/index.html

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[PDF] **Long Term Remedial Measures** of **Sedimentological Impact due** to ...
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... 321 **Long Term Remedial Measures** of **Sedimentological Impact due** to **Coastal**
Developments on the South Eastern Mediterranean Coast ...
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http://www.unesco.org/csi/pub/source/ero11.htm
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Gravity drainage: http://www.pari.go.jp/bsh/ky-skb/hyosa/hpj/english/02menb/yana/yana.htm
Pressure Equalizer: http://www.ecoshore.com/

[PDF] A **Trial** of the **Pressure Equalisation Module Method** of **Beach** ...
Bestandsformaat: PDF/Adobe Acrobat - HTML versie
Page 1. A **Trial** of the **Pressure Equalisation Module Method** of **Beach Protection**
at **Hervey Bay, Queensland** SM Hunt 1 , GB Stuart 2 ...
www.cdesign.com.au/ Coast%20to%20Coast%202004%20(CD%20Proceedings)/pages/coastfinal00037.pdf - Gelijkwaardige pagina's

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http://www.nautilus-consultants.co.uk/pdfs/Nautilus%20artificial%20reefs%20report.pdf

Other systems
http://www.unesco.org/csi/pub/source/ero7.htm
Pile rows: http://www.dmc.nl/publications/coastalprotection.pdf
http://wave.oregonstate.edu/Education/REU/2003_REU/papini_report.pdf.
Screens: www.han-padron.com/papers/2004-01.pdf
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ıttp://www.marine.ie/mrm/section3/sec3_page10.html
http://www.scourcontrol.co.uk/specials.html
http://www.asbpa.org/summit_pres/JoanPope.pdf
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