EFFECTIVENESS of COASTAL DEFENCE MEASURES

DESIGN BACKGROUND
for
COASTAL DEFENCE & MANAGEMENT

NEAR-FIELD and FAR-FIELD FACTORS
in
OVERALL (Geometrical) & STRUCTURAL DESIGN

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The following major symbols have been used throughout this document:
'b' = breaking subscript
'B' = structure (berm, breakwater, etc.) subscript
D = pile diameter; screw (propeller) diameter
d = grain diameter
g = acceleration due to gravity
H = wave height
H_s = significant wave height
h = depth of water
L = wave length
L_p = peak (spectral) wave length
'o' = initial or deep-water subscript
Q = flow rate
q = unit flow rate (per unit width)
R_n = run-up height
T = wave period
T_p = peak (spectral) wave period
α = slope angle (to horizontal)
$\xi = \frac{\tan \alpha}{\sqrt{H/L}}$ = Irribaren or Battjes index

PREFACE

This report is a final document prepared under a research programme titled *'Dimensioning of Coastal Structures'* commissioned by *RIJKSWATERSTAAT* and *DELFT HYDRAULICS*. The major objective of the study was to provide a state-of-the-art review of coastal defence measures and practices, aimed at deeper understanding of the physical processes affecting coastal structures, thus putting forth a design background for coastal engineers.

The realm of coastal processes and the interactions of coastal structures and the marine environment 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 description of the transformation of shore in larger scales, i.e. the far-field effects, is summarized in Chapter 2 while the near-field mechanisms, encompassing different overall and local failure modes, stability and performance are dealt with in Chapter 3. Both are followed by Chapters 4 and 5 with design computations and examples, respectively. Unconventional design is illustrated in Chapter 6, and national policies of coastal management and defence are depicted in the closing Chapter 7.

The design procedures for coastal structures should include geometrical design and structural design reflecting respectively the far-field and near-field requirements imposed on structures. This corresponds to our 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.

In this report we concentrate on the numerous hydrodynamical, sedimentological, geotechnical and structural factors and processes, both far-field and near-field, controlling the effectiveness of shore protection measures. Hence a background is provided for understanding of the behaviour of coastal structures. Together with the modelling techniques for the processes outlined for various defence schemes, this background information is shown to be useful in establishing criteria for the selection and design of shore protection and management schemes.

Hence the concept of this study can be formulated as physical and mathematical *description of coastal phenomena, parametrization of far-field and near-field effects, and an outline of the pertinent geometrical and structural design procedures and practices.* Aside from this unified approach, frequent reference is made to the less accessible eastern sources, somehow exotic and hopefully thought-provoking for a western reader, cf. numerous Soviet, Polish, etc. bibliographical entries. Cross-references are also provided to Shore Protection Manual (US Army, 1977, 1984), the broadly cited worldwide standard, so as to shed light on the diversified topics and problems of coastal engineering, and to identify the 'grey spots' in our knowledge and technology, at the same time.

In most chapters we provide certain overview comments followed by highlights deserving most attention. It should be stressed, however, that it is not our intention, nor is it possible in this report to summarize all achievements in the field covered herein. An attempt to provide a unique, precise and comprehensive summary is obviously an overly ambitious task, in view of the wide spectrum of concepts in shore protection, approaches to implementation of these concepts, and interpretation of the effectiveness of coastal measures.

CHAPTER 1

Coastal defence and design background; Boundary conditions

1.1 GENERAL: OBJECTIVES, OPTIONS, METHODS...

Coast Protection Acts all over the world empower various authorities to construct works and take measures to protect land from erosion or encroachment by sea. Erosion is tacitly taken to mean loss of land while "encroachment" concerns the subsequent incursion of the sea. Works so constructed are known as "coast protection" means. Sea defence works are usually understood as those the primary function of which is to prevent or alleviate flooding. No clear-cut distinction is made in this report between coast protection and sea defence schemes.

There is a growing awareness of the need to design coastal works compatible with the marine environment, in particular the beaches, which provide the main natural protection to the coastline. This design is a rapidly developing science but nevertheless also remains an area of engineering, and art, in which experience plays a major role. Even more so, **coastal defence** gradually evolves into **coastal management**, in terms of which a single defence project for an isolated location must be looked upon in its broader environmental perspective. An individual structure should not only provide sufficient protection for its immediate neighbourhood but also minimize its detrimental effects on the adjacent section of coastline. Hence coastal management means rational perception and control of coastal processes, in a variety of scales, aimed at preserving the coastal environment system.

Coastal management, in its widest sense, must take account of all factors which have a bearing on the future of the coastal zone. This may well entail many scientific and engineering disciplines other than coastal engineering and at sites and locations far removed from the costal zone. For instance, the construction of a dam far upstream from a river mouth can lead to a significant and sometimes dramatic reduction in the sediment concentration in the river. The effect of this on the coastline can be catastrophic and irrevesible. Many other factors could be cited, some of which are listed below (Stickland et al. 1987):

- sand and gravel extraction

- commercial and fishing harbours
- yacht marinas
- waste disposal
- water cooling inlets and outfalls
- dredged channels
- reclamation schemes
- river training works
- dune conservation
- protection of flora and fauna
- artifical islands
- cliff erosion
- requirements of leisure and tourism
- coast protection works
- pollution
- water quality.

Some of these factors interact with one another, others are almost uncorrelated. The extent to which this applies in any particular region, area, or specific site needs careful evaluation. This is one of the major rolés of coastal management.

The primary objectives of a typical coastal management study are to formulate long-term engineering planning and 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.

The key element in any coastal management study is a thorough understanding of coastal processes by which one means 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 sub-divided 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 satisfactorily evaluated or long-term planning undertaken.

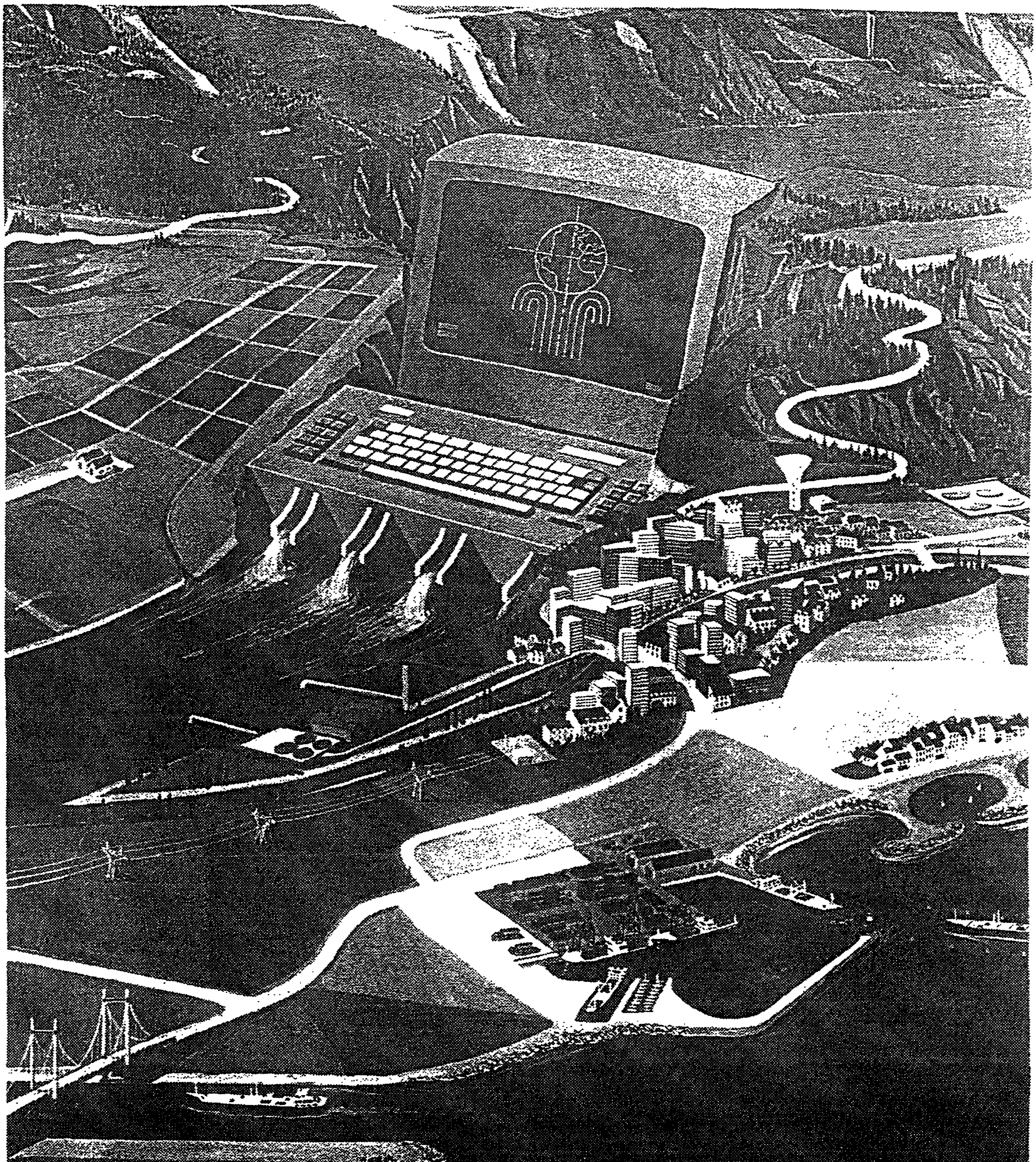


Figure 1.1. Natural and Anthropogenic Factors Affecting the Coastal Zone, its Management, and (Mathematical) Modelling; SOGREAH Artist's Vision.

b. intercept and dissipate wave energy,

c. directly armour the shore, and

d. retain dune bluffs, cliffs and any other earth slopes against sliding.

The basic tools of the coastal engineer are still fairly limited and comprise **cross-shore** structures (such as groynes, jetties, spurs...) **shore-parallel** structures (offshore breakwaters, sea walls, dykes, revetments (generally close to shoreline)...), beach nourishment and, to lesser extent, headland structures.

environment-friendly measures **Groynes** (Fig.1.3) 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, groynes 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, groynes are, of course, of no value.

In addition to controlling the rate of drift, groynes 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 groyne systems has proved worser than having no groynes at all.

Offshore breakwaters (Fig.1.4) are usually provided either to reduce wave energy at shoreline structures or to modify the wave climate and redisturbute 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, etc.

Sea wall (Fig.1.5 a), often used interchangeably with bulkhead (Fig.1.5 c) 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".

Dykes are generally intended as means of flood prevention. The crest of a dyke is elevated high enough to counteract or confine overtopping in rare storm surge events.

Revetment (Fig.1.5 b) is placed on a slope to protect it and the adjacent uplands from erosion, with no defence of the neighbouring areas. The wave reflection, a serious disadvantage of vertical-wall bulkheads (sea walls), does not accelerate erosion as strongly at revetments as it does at sea walls. **Beach nourishment** or fill, or recharge, (Fig.1.7) consists in importation of granular material to a beach from an outside 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

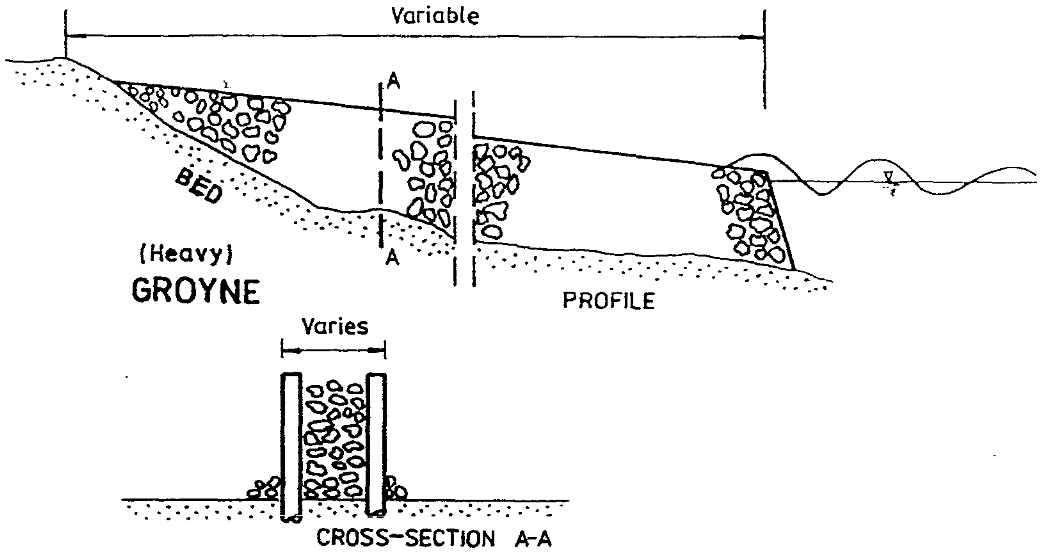


Figure 1.3. Example of Cross-Shore Structure: Groyne.

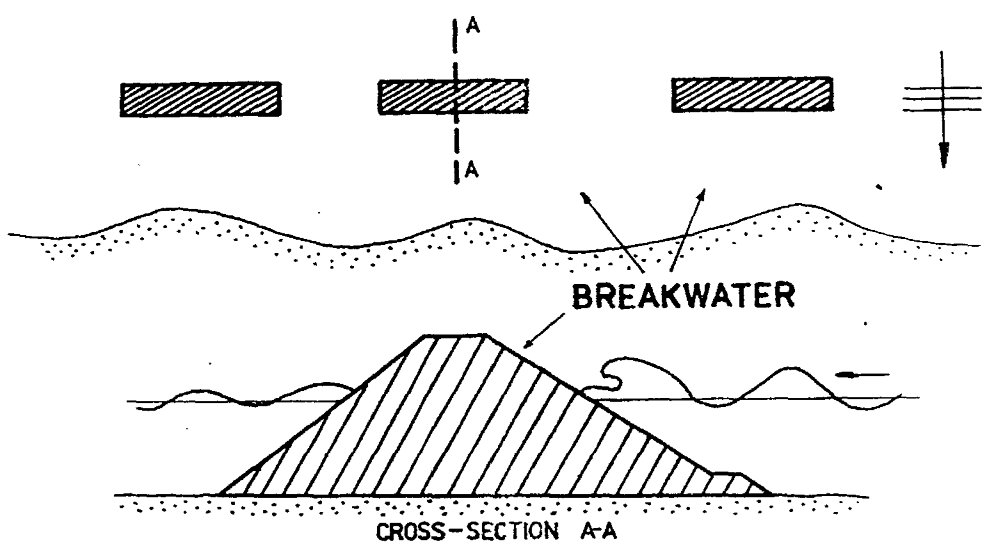


Figure 1.4. Example of Shore-Parallel Structure: Offshore Breakwater.

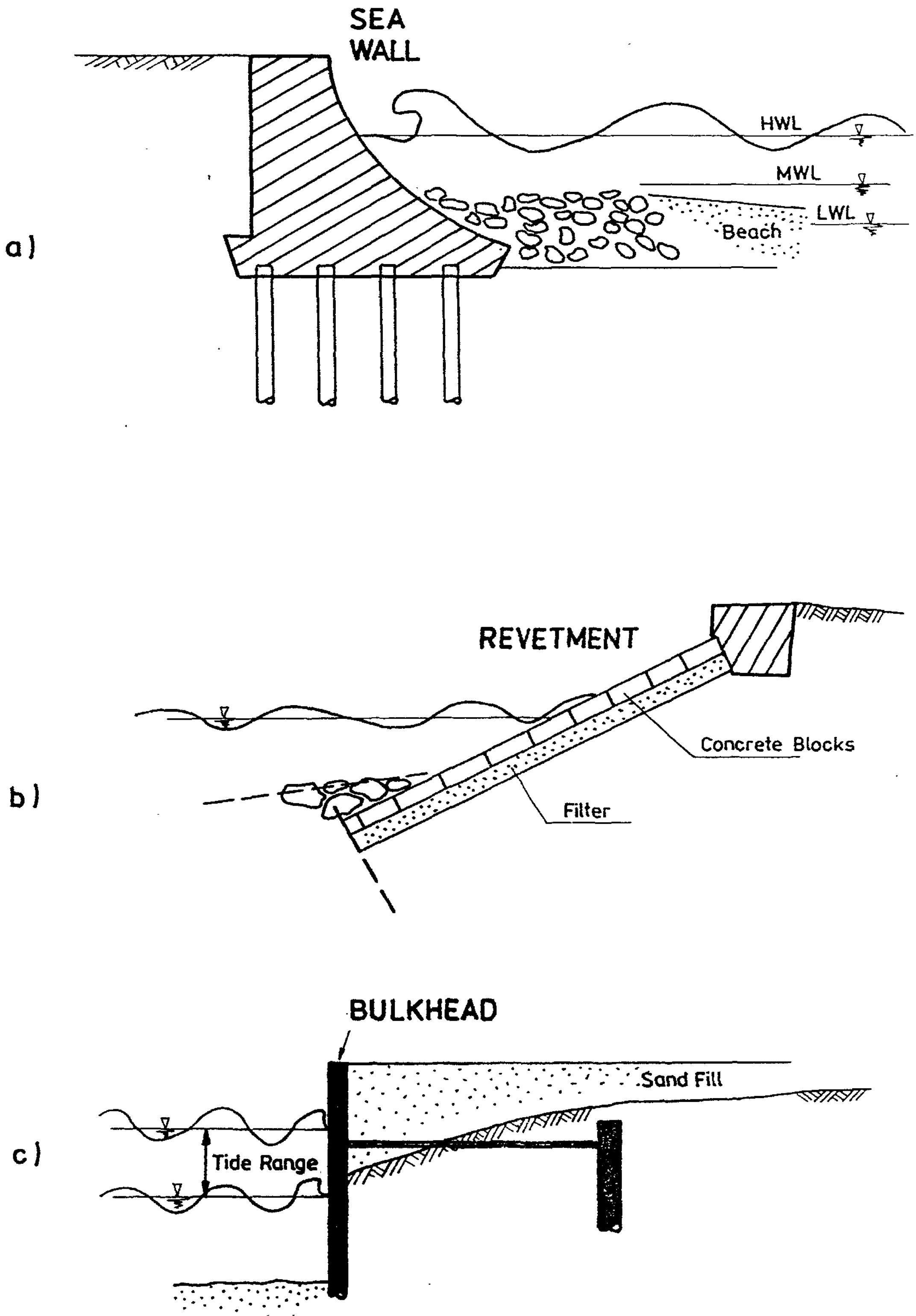


Figure 1.5. Examples of Shore-Parallel Structures: Sea Wall (a); Revetment (b); Bulkhead (c).

Figure 1.6. Cross-Shore and Longshore Structures (Groynes at bottom and Sea Wall at top left) on the Dutch Coast near Petten (HydroDelft 68, 1984).

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 (Silvester 1972, 1976, 1980). The crenulate shaped bays can be kept in equilibrium by the use of a system of headlands. The headland system is claimed to be in feedback with coast and to combine the advantages of groynes and detached breakwaters (shore-parallel or oblique).

Various **low-cost, environment-friendly, emergency and temporary measures, and combinations** therof (Chapter 6) are shown as 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. Hence the approach in this study to provide a background for understanding of the physical mechanisms intervening in the operation of various coastal defence schemes.

From the physical perspective, the coastal processes can be classified as **longshore** and **cross-shore**. The schematization depicted in Figures 1.9...1.12 visu-

Figure 1.7. System of Detached (upper) and Shore-Connected (lower) Breakwaters Combined with Artificial Beach Nourishment to Regenerate Pedregalejo Beach and Maintain One of the Most Used Recreational Area in Malaga (Spain).

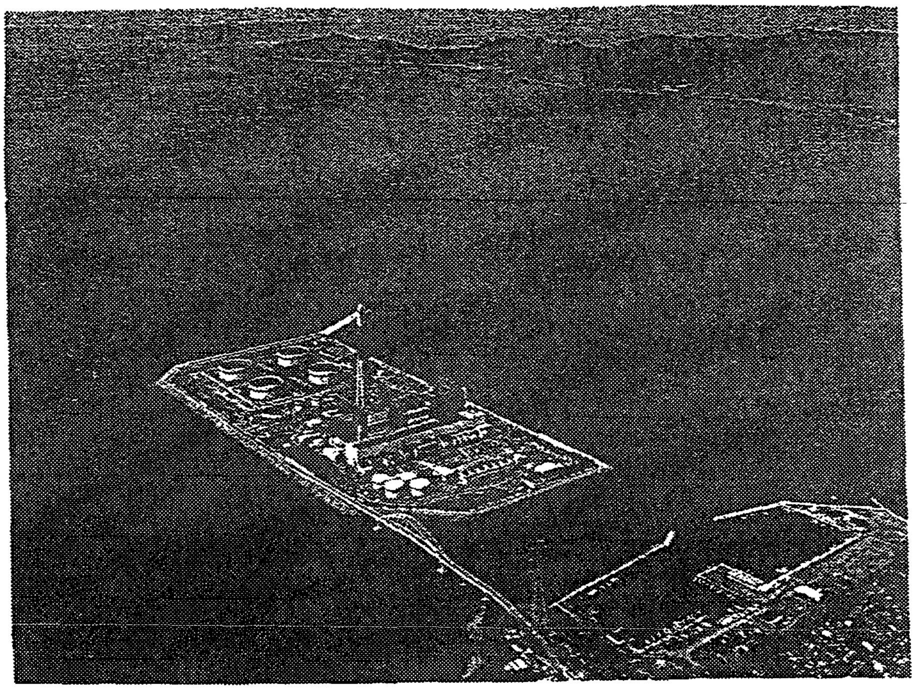


Figure 1.2. Construction of Artificial Island in Japan, as Example of Offshore Engineering Activities Affecting the Coastal Zone.

Given that a thorough understanding of coastal processes is central to any coastal management study, it follows that the fundamental prerequisite is the collection, analysis and interpretation of data. At present much of this data collection is carried out for specific projects and frequently for limited lengths of coastline.

The emphasis in this study is placed on coastal engineering, with fringe interest in planning and financial strategies, and other non-engineering activities. Particular attention is focused on the understanding and description of coastal phenomena in the presence of various defence schemes. Such knowledge is deemed crucial to coastal management, and is therefore collated in the following chapters of this report.

Three basic options are possible in response to the coastal erosion problem:

1. do nothing and suffer the inevitable loss or sell and pass the problem on the someone else

2. relocate or move the endangered structures

3. attempt to control or reduce the erosion through some protective measures.

This report considers the third option only and thus presents the measures that

a. interact with wave, current and sedimentation patterns to produce a desirable degree of coast control

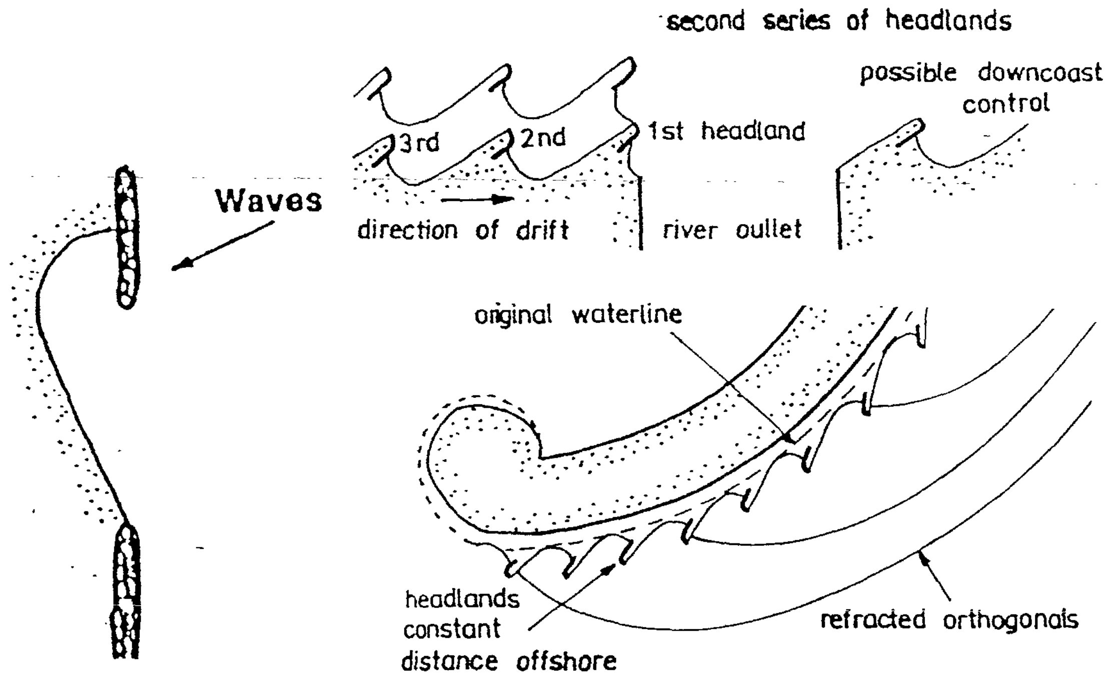


Figure 1.8. Concept of Headland Control and Some Implementations.

alises the distinction, with the additional feature of two-dimensionality in the vertical (Fig.1.9...1.11) and horizontal (Fig.1.12) plan.

The basic wave processes intervening in the coastal zone with structures are illustrated in Fig.1.9. The incident wave energy E, is reflected from a structure, (E_r), transmitted across it (E_t), dissipated through breaking, mixing and friction in various types of flow (E_{diss}), and may be utilized for generation of currents (E_{curr}), or other modes of motion. The variety of transmission, dissipation and reflection patterns brings about diversified responses of protected shores, outlined in Fig.1.10, still in the two-dimensional vertical view.

Accretion and erosion trends shown schematically for faultless submerged and emerging breakwaters in Fig.1.10, may become much more complex if the structures, such as bulkheads, seawalls or revetments in Fig.1.11, are subject to continuing overtopping and undermining, resulting in gradual worsening of the structure's functioning. More still, the three-dimensionality of the coastal processes, the horizontal aspect of which is brought to the reader's attention in Fig.1.12, may dramaticaly alter the overall picture and thus considerably modify the effectiveness of coastal protection.

Outflanking alone, being an additional contribution of Fig.1.12 to the overtopping and undermining in Fig.1.11, and the wealth of wave motions, currents and combinations thereof illustrate the importance of the problem of complete and reliable description of all coastal processes likely to occur on natural and engineered coasts.

The manner in which waves interact with a structure, is predominantly governed by the form of the structure itself. Rigid structures resist wave attack through holding a very high mass, and thus a very high inertia. In this, they act as almost a single unit of armour that is so large that wave energy should be incapable of moving it. A rigid structure can destroy wave energy in at least three ways:

(a) The wave impacts upon the structure, such that it is converted into a large volume of spray, which rises high into the air. This is an efficient destruction mechanism, but the impact forces are very high. Shock waves are induced in the structure and these can be dangerous, if transmitted into a non-rigid foundation.

(b) The wave is almost totally reflected. This is a much more efficient process than impact, since the wave forces are minimum, but full reflection is usually only economically possible, for the smaller or non-extreme waves. Wave reflection however may generate dangerous toe scour if the rigid structure is founded upon an erodible sea bed, and the water depth is significant. Wave reflection is very common in Nature, on hard rocky or cliff coastline.

(c) The wave is forced to expend its energy by uprush across a sloping rigid hard artificial "beach". The wave lifts a weight of water a distance (with only partial breaking) to expend its volume of potential work. This is again an efficient process, but on a smooth rigid slope, the uprush height that must be allowed, becomes excessive. A reflective wall at the top of a well sloping rigid seawall, is thus a common feature, often allied with a stepped or roughened surface on the sloping surface, to increase the uprush drag.

The degree to which any of these three processes become dominant, is generally controlled by the slope of the rigid structure. Large vertical structures attract the impact mechanism, whilst gently sloping structures follow the uprush solution. All rigid structures however, tend to hold a high reflection coefficient and this will control the wave behaviour, for a fair percentage of the total input, of the more moderate waves. Some relatively steep rigid structures may demonstate a combination of all three wave destruction mechanisms, simultaneously for much of the time.

An unfortunate feature of rigid structures, is their sensitivity to overload. Larger waves than the design capacity of the structure can rapidly lead to massive overtopping, even if the structure holds its structural integrity. Quite small errors in the design wave exposure can then result in widerspread damage, landwards of the structure.

In the use of flexible rubble structures, mankind is in fact closely following Nature. Sandy and gravel beaches are Nature's own rubble mounds - it is just that the armour units (sand grains and gravel stones) are much smaller than most of mankind's armour. Natural beaches follow exactly the same "rules" as man-made

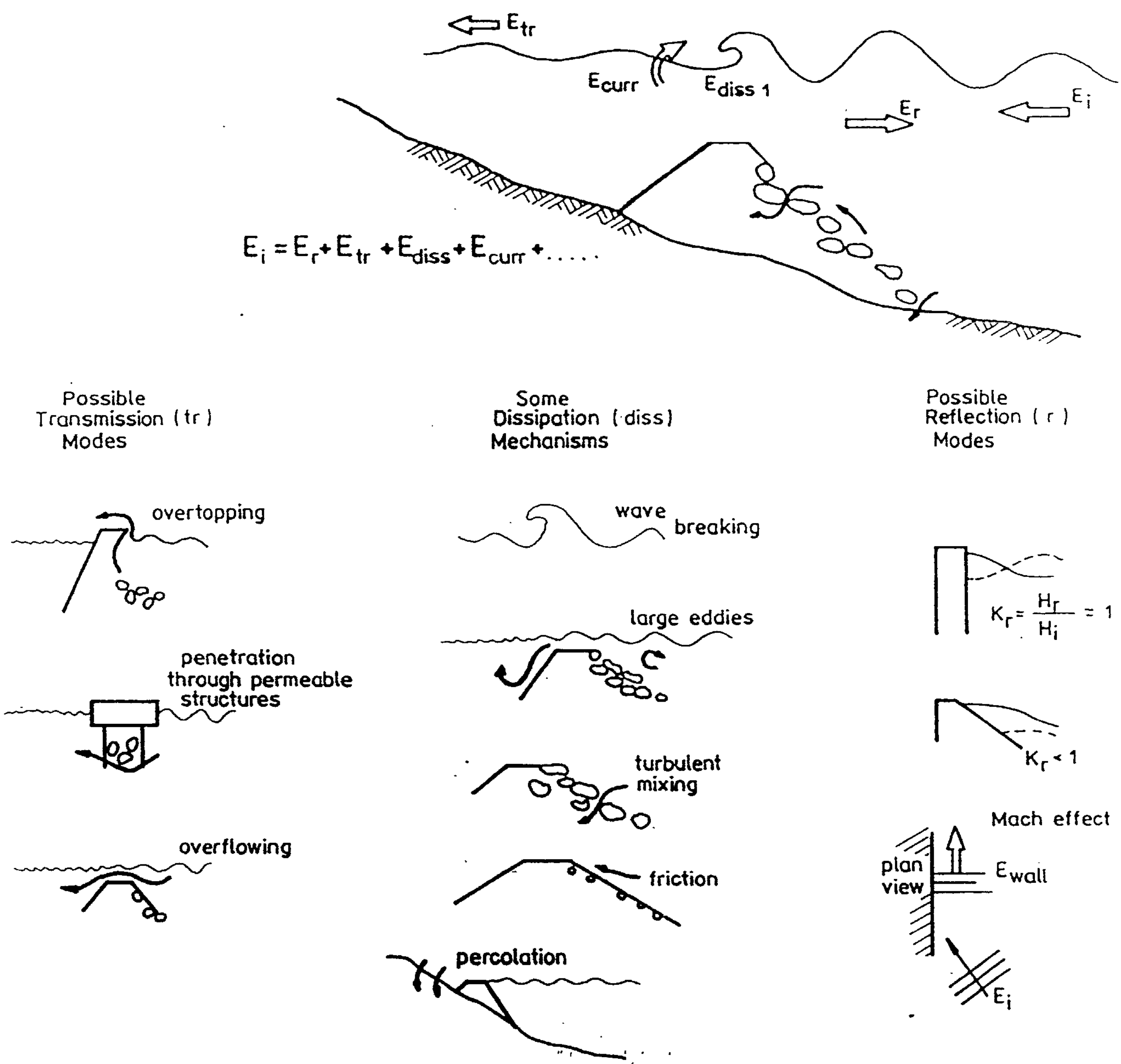


Figure 1.9. Wave Mechanisms Encountered in Coastal Engineering
E_{curr} = *energy lost for generation of currents, nonwave modes, etc.*

mounds, there exists a quite precise relationship between wave energy, beach slope and particle (i.e. mini-armour) size.

The manner in which the "final" wave that reaches the structure, interacts with it to discharge its hydraulic energy, is therefore largely controlled by the type of the structure itself. But the properties of the final wave are controlled by its previous shoaling history. This wave history is again pre-controlled by two other parameters, the type of seabed and the depth of water. Of these two parameters, the class of seabed is the most important. The final shoaling wave that has propagated over an impervious seabed holds a cross-section over twice that of the porous seabed wave, and its length is also greater.

The realm of coastal processes and the interactions of coastal structures and the marine environment 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 far-field effects have been discussed at length in our Chapter 2 while the near-field phenomena have been outlined in Chapter 3. These chapters are followed by Chapters 4 and 5 with design computations and examples, respectively. Unconventional design is illustrated in Chapter 6, and national policies of coastal management and defence are presented in the closing Chapter 7.

The design procedures for coastal structures should include **geometrical design** and **structural design** reflecting respectively the far-field and near-field requirements imposed on structures. This corresponds to our 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.

In this report we concentrate on the numerous hydrodynamical, sedimentological, geotechnical and structural factors and processes, both far-field and near-field, controlling the effectiveness of shore protection measures. Hence a background is

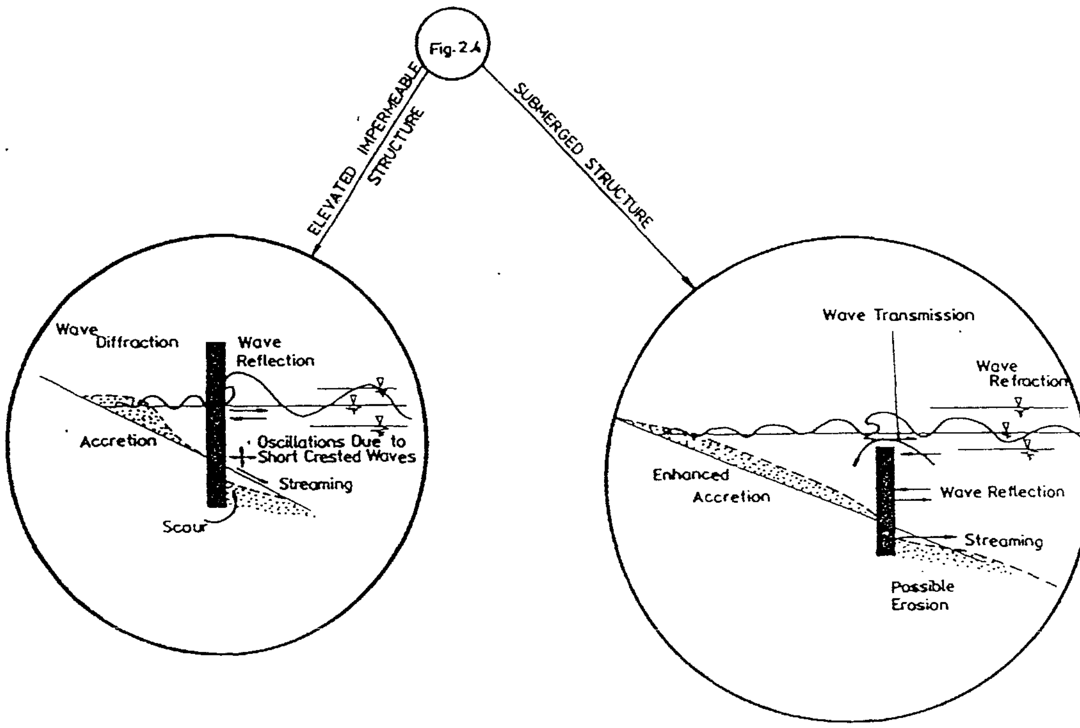


Figure 1.10. Interaction of Waves and Engineered Shore (2-DV).

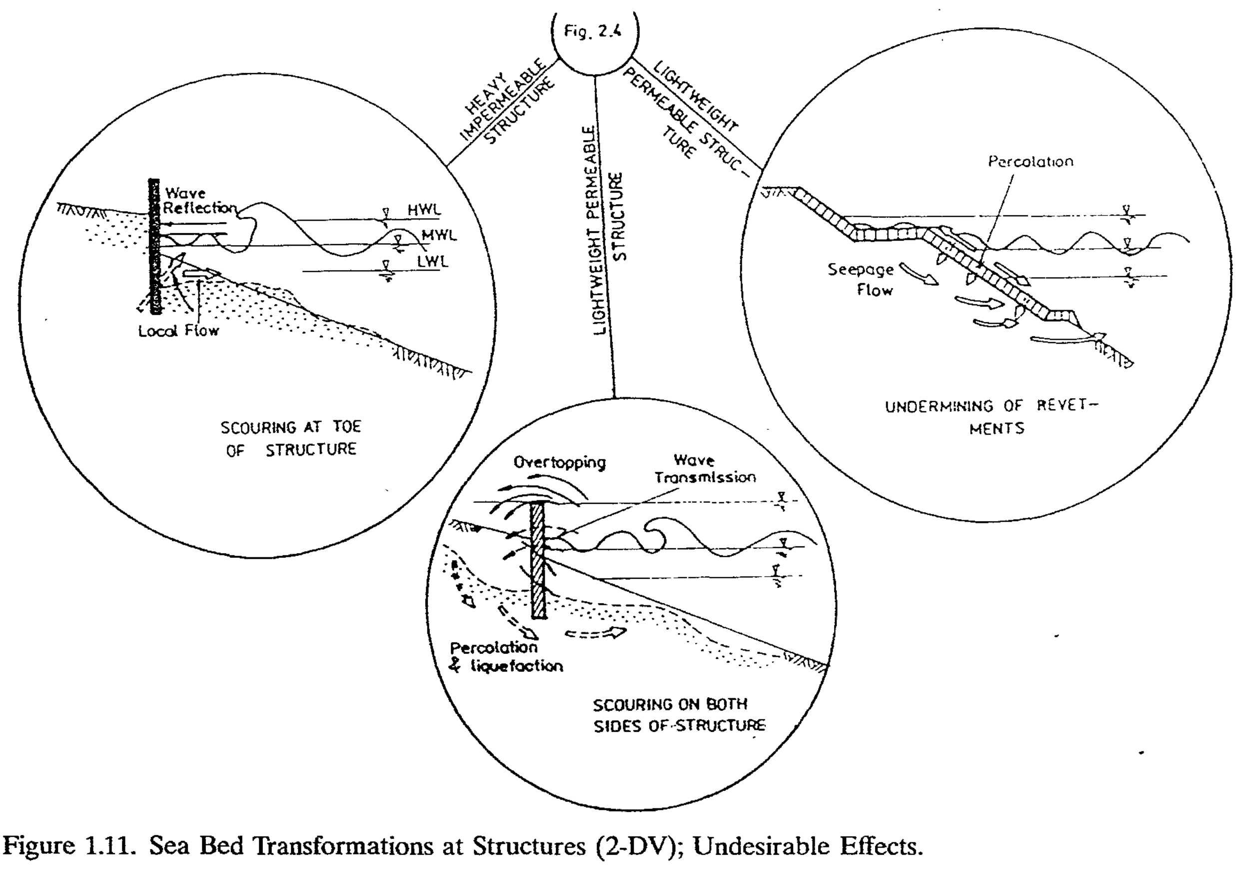


Figure 1.11. Sea Bed Transformations at Structures (2-DV); Undesirable Effects.

provided for understanding of the behaviour of coastal structures. Together with the moddeling processes outlined for various defence schemes, this background information is shown to be useful in establishing criteria for the selection and design of shore protection and management policies.

Hence the concept of this study can be formulated as physical and mathematical **description of coastal phenomena, parametrisation of far-field and near-field effects, and an outline of the pertinent geometrical and structural design procedures and practices.** Aside from this **unified approach,** frequent use is made of less accessible **eastern sources**, somehow exotic and hopefully thought-provoking for a western reader, cf. numerous Soviet, Polish, etc. bibliographical entries. Cross-references are also provided to Shore Protection Manual (US Army, 1977, 1984), the broadly cited worldwide standard, so as to shed light on the diversified topics and problems of coastal engineering, and identify the "grey spots" in our knowledge and technology, at the same time. In most chapters we provide certain overview comments followed by highlights deserving most attention. It should be stressed, however, that it is not our intention, nor is it possible in this report to summarize all achievements in the field covered herein. An attempt to provide a unique, precise and comprehensive summary is obviously an overly ambitious task, in view of the wide spectrum of concepts in shore protection, approaches to implementation of these concepts, and interpretation of the effectiveness of coastal measures.

VOCABULARY

Some notions which are referred to throughout this document include the following concepts of coastal management and defence: effectiveness, reliability and risk. Therefore it is appropriate to formulate them more precisely.

Effectiveness of shore protection is a combination of morphological effects, reliability of protection measures, total cost of investment and maintenance, and possible outcome of failures. Effectiveness is evaluated through the achieved coastal defence and management goals measured in terms of shore stabilization or transformation, with inclusion of unavoidable and unwanted by-effects.

In passing, **damage** can be defined as a certain change in the state of structures, with respect to (1) external boundaries ; (2) configuration and cross-sections; (3) integrity of constituents.

Changes (1) and (2) often correspond to a certain physical loss or displacement of material of the structure and its surroundings. This in turn may cause a certain loss of functions.

In practice there may be a gradual loss of functions with increasing damage. Therefore **failure** can be regarded as a phenomenon corresponding to the state of a

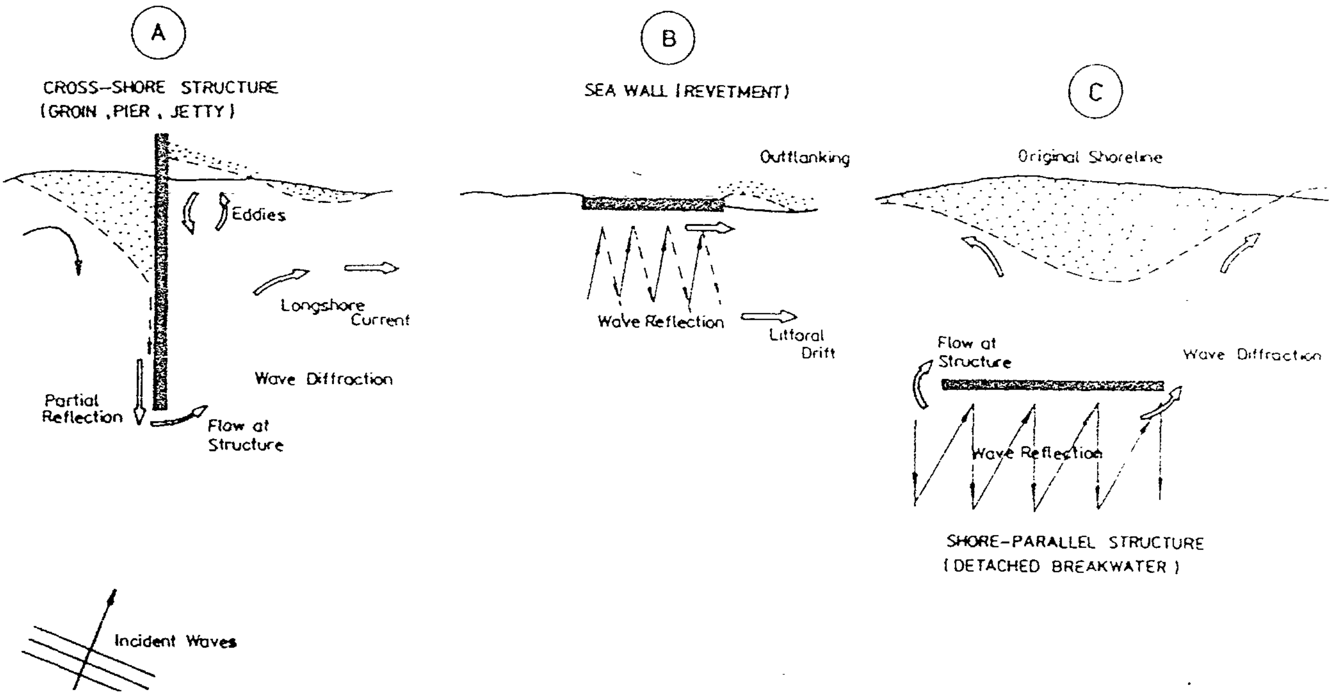


Figure 1.12. Shore Evolution at Structures (2-DH).

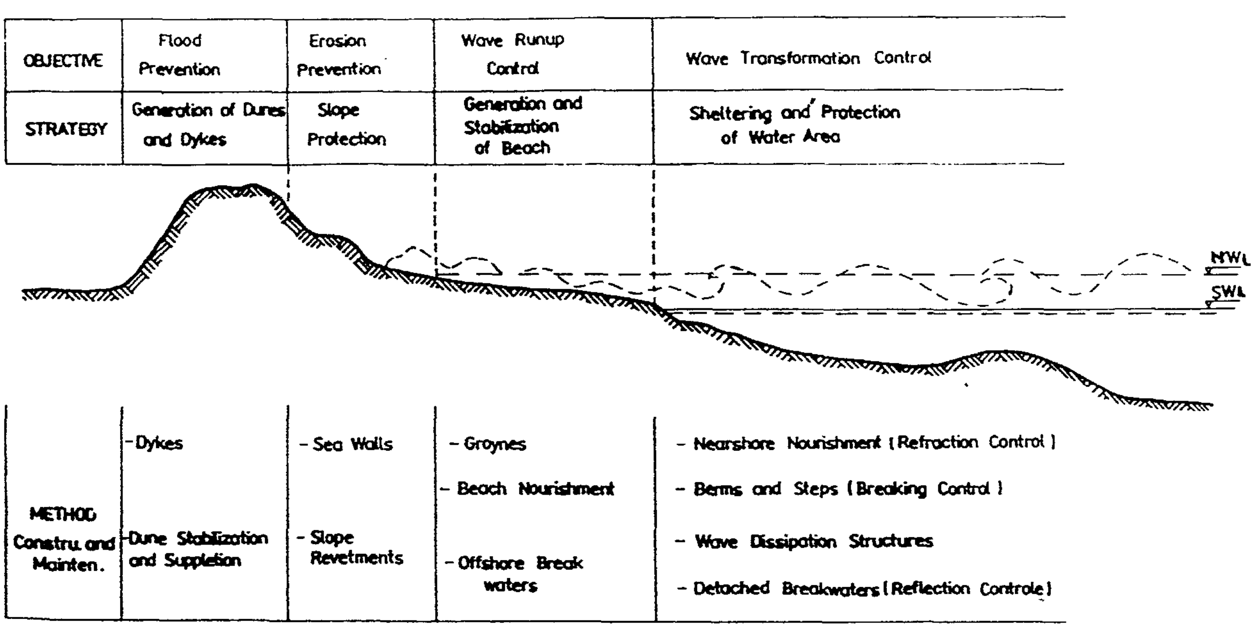


Figure 1.13. Review of Objectives, Strategies and Methods of Coastal Defence and Management.

certain **ultimate** degree of damage, denoting irreversible changes.

Reliability of a defence mesure is referred to as effective operation of the measure within its design lifetime, in (variable) design conditions, both external (the so-called 'boundary conditions') and internal.

Risk is commonly associated with failure of a structure or measure. Ideal effectiveness and reliability are equivalent to zero risk, which is hardly conceivable in view of the randomness of coastal phenomena (rare events do occur no matter how low their probability) and our limited knowledge of the factors of coastal defence. Risk is assessed more quantitatively in the forthcoming sections.

Effectiveness, reliability and risk depend on quality of defence concept and system chosen, design, materials and performance of construction.

The factors, or ingredients of the coastal defence mentioned, can be grossly identified as the **design factors** encompassing the following items

- coastal morphology
- hydraulic boundary conditions
- geological and geotechnical boundary conditions
- sedimentation, shore evolution, and lithological interactions with defence measures

Coastal morphology in the design stage requires the knowledge of general topography of the site, and the shore profile and its transformation. Bathymetric and topographic charts for the surf zone, stretching by depths about 10 m, and the strip of land endangered by runup, overtopping etc., including the flooded hinterland, are important background documents. Together with the type of shore profile and its characteristic seasonal slopes one should identify shoals and bars, beaches, dunes and cliffs, shoreline configuration and its discontinuities, lagoons, lakes and estuaries, vegetation and anthropogenic effects.

Hydraulic boundary conditions should incorporate design data on water levels, wind, waves, currents, tides, ice and other hydrological phenomena, including the effects of lagoons, lakes, estuaries, intracoastal waters and groundwater. A variety of hydraulic interactions with land and coastal structures, such as runup, overtopping, percolation etc., should be considered. Statistical description seems quite obvious, and joint probability distributions are most desirable.

Geological and geotechnical boundary conditions should also be quantified in a design-oriented manner. Geological surveying reflected in charts and cross-sections provides the configuration of various soil kinds of which the coast consists and the thickness of active strata of sand, fairly immobile cohesive beds, etc. Soil characteristics, such as grain size distributions, densities, porosity, angle of internal friction, specific cohesion, etc. should make up a sound basis for adequate design of external and internal loads on protection structrures, both static and dynamic, resulting from combinations of hydraulic and geotechnical boundary conditions. Seepage flow, buoyancy and uplift, suffosion and percolation are examples of processes being controlled by both types of boundary conditions.

Sedimentation and lithological processes control the behaviour and evolution of shore in both far and near field, with or without structures. For design purposes, one should determine the kind, structure and properties of sediment over the coastal zone protected, configuration and capacities of natural and artificial resources of sand being supplied to the coast (including entrapment, bypassing, dredging etc.), sediment transport rates, either as a map of local rates or more simply as gross longshore and cross-shore figures, and the like. This data should be used for evaluation and design modelling of *shore evolution*, which is clearly related to daily, monthly, seasonal, yearly, and long-term variation of sediment transport patterns, hence changes in shore topography, and alternating accretion and erosion. Modelling of shore evolution, so crucial for adequate selection of a defence measure and the design of its ultimate shape, is unfortunately susceptible to even slightest inaccuracies in design input parameters.

Hydraulic boundary conditions and the "static" aspects of lithological processes, i.e. coastal morphology, are dealt with in this chapter, while the "dynamical" processes of the shore and its interaction with structures are treated in Section 2.7.

1.2 MORE ON FUNCTIONAL REQUIREMENTS

Usual design stages embody

(a) assessment of coastal hazards and potential damage, failure or disaster

(b) formulation of functional requirements, general concepts, and alternative solutions;

(c) determination of the hydrological, sedimentological and geotechnical input (so-called "boundary conditions");

(d) assessment of morphological effects of the alternative design;

(e) review of failure mechanisms;

(f) cost analysis, assessment of environmental impact, and other non-engineering tasks;

(g) selection of design version and its components;

(h) follow-up design of structural details, specifications for construction materials etc.

Let us first concentrate on general design policies, i.e. formulation of **functional requirements**, as in item (a) above. They determine the area of performance in which the structure will operate satisfactorily.

Examples of some of the most prominent functional requirements of coastal structures are:

- flood prevention of low hinterland (sea dykes and walls) or inland waters (barriers);
- prevention of wave overtopping (sea walls, barriers);
- prevention of wave propagation (breakwaters, jetties);
- dissipation of the energy of waves and currents (offshore breakwaters, groynes);
- flow control (jetties);
- slope erosion and scour prevention (revetments, bottom protection...).

It should be emphasized that these functions may vary in time. Additional functions may also be assigned to an existing structure during its lifetime. Temporary structures, for example, are often used during construction. In such cases, requirements with respect to materials and damage may be treated differently than those of final structure.

Important issues in the design of a coastal structure system and its constituents are:

- type and magnitude of loadings to be expected during the lifetime;
- accepted damage level given the expected loadings;
- permeability of the system and its foundation, core, etc.;
- flexibility of the system;
- construction and transportation aspects;
- management aspects;
- environmental impact.

All of these elements impose certain (sometimes temporary) functional requirements given below.

Design requirements	Remarks
Stability & risk of damage	protection of external boundaries; protection of internal boundaries;
Permeability	dissipation of wave energy; reduction of internal hydraulic gradients (pore pressures) and internal flow velocities; retaining of (finer) core- and construction materials;
Flexibility	neutralization of small deformations and initial damage
Construction	requirements of handling and equipment
Management	placement and recycling of units stock piling of materials; accessability of structure for inspection, maintenance and repair;
Environmental acceptability	landscape, vegetation, recreation;

The near- and far-field phenomena may be looked upon from different perspectives, and can incorporate further divisions. For instance, one may distinguish the aspects of stability, flexibility, durability, maintenance, safety, acceptability, cost, etc, as already done above. Let us follow the wording accepted by PIANC WG 4 (1988).

(a) **Stability:** - structure must be capable of withstanding the imposed loads and must have the necessary strength characteristics to resist displacement. It must stabilize and prevent erosion of the subsoil

(b) **Flexibility:** - structure must be capable of accommodating deformation without its other functions being adversely affected

(c) **Durability:** - structure must not suffer loss of function due to ageing during its design life. Resistance to attack by chemicals, ultraviolet light, microbes, vandalism etc. embodies aspects of durability

(d) **Maintenance:** - should incorporate features to facilitate maintenance which will include repair of local damage and renewal of degraded materials. Elements requiring periodic maintenance must be easily accessible for inspection and renewal

(e) **Safety:** - potential hazards to construction workers and users should be eliminated by design. Safety features should be incorporated to take account of all activities which may take place on or about the structure, whether they are authorised or not

(e) **Social and environmental acceptability**: - structure will become part of the landscape and local ecological system. It will provide employment during its construction and maintenance, and opportunities for recreation. The design must encompass more than purely technical considerations to meet broader requirements

(f) **Cost**: -the design must fulfil all its functional requirements within the cost allocation available for the scheme. Cost is a function of many elements, including installation, maintenance and replacement costs - discounted to present values, i.e. the life cycle cost. Cost comparison between two different schemes should be done by comparing life cycle costs.

It can be seen that near-field and far-field phenomena are archifactors in the stability, etc. aspects. Hence all these aspects but cost are dealt with in this document.

Critical elements of the design of shore protection and coastal management measures include (1) generation of desirable **far-field effects** induced by construction of the control measures (2) provision of the **stability** of structures and their members, (3) prevention of undermining, outflanking and other modes of unwanted **morphological changes in the near field** of the structure, (4) minimization of settlement, seepage, removal of structural units, and many modes of **internal failure**. Adequate attention must also be paid to elements of (5) **external failure modes**, such as overtopping.

1.3 SOME BOUNDARY CONDITIONS

1.3.1 *General; Coastal Hydrodynamics*

Definitions

This section contains a certain broad description of a selection of the aforementioned **hydraulic** boundary conditions and **shore morphology**. The **dynamic** interactions of the water, shore and structures are discussed in Chapter 2, i.a. Sec.2.7 on mathematical modelling.

The **coastal zone** can be identified as shown in Fig.1.14, where it stretches from land to "deep water", i.e. the area of practically negligible effect of wave motion on sea bed. It encompasses coast proper, beach and nearshore zone, hence modifies the Shore Protection Manual terminology, in which the coastal area ends at the seaward edge of the surf zone.

Wind, waves, surges, currents, and other phenomena combine to cause coastal disasters. Wind is an important factor which generates surface waves and causes storm surges. The combined action of wind waves and wind- and wave-induced set-up brings about substantial transformations of shore.

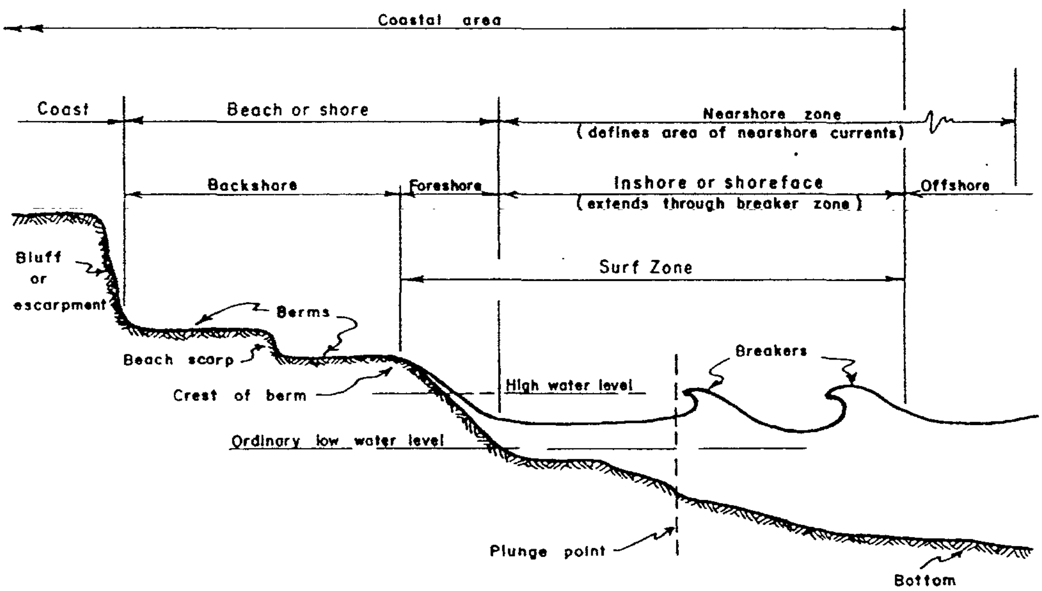


Figure 1.14. Schematisation of Coastal Zone.

In assessment of extreme erosional effects due to exceptional storms it is natural to resort to an analysis of rare hydrological events that create so many problems in coastal defence and may seriously affect success of any project. Attention to extreme events should always be paid.

A particular problem facing coastal engineers is a rational assessment of the probable deep-water wave to which the structures are most susceptible. It is known that coastal structures are most prone to change from the largest waves only, which are generated during rare storm events. Severe storms become superimposed on the normal weather as an added variable.

The major problem is that each of the probabilistic distributions attributed to "normal" and "rare" events cannot be simply added or combined. The coastal structure designer thus has to assess the exposure of each structure from separate ocean wave climates - each of differing severity and probabilistic distribution.

This range of probabilities of each class of these events creates the greatest problems in all wave climate forcasting exercises. If the coastal engineer relies on usual weather data he will probably vastly underrate the extreme event wave properties. If he attempts to synthesize the properties of the extreme events, his predicted wave train, wind set-up and surge could be widely in error due to lack of data. For many years, coastal structures were proportioned to accept the "design wave". Recently however, the concept of the measured prototype wave spectrum has gained greater support.

In addition to "mean" and "rare", extreme "freak" waves do exist, although their probability will almost certainly not be detected within normal weather system wave statistics. This phenomenon, together with a better understanding of the behaviour of natural wave groups, will probably lead in the future to the use of a "design wave train" for assessing coastal structure exposure. A single wave seldom destroys a structure, but three or four "freak" waves or groups of them, can cause the same havoc as several hundred smaller waves.

In more accurate analysis one must also take into consideration numerous nonlinear energy dissipation and attenuation processes which, along with inertia effects, control the growth of wind in high-speed atmospheric phenomena. Duration of different wind events is yet another quantity to be included in the realm of all statistics possible.

Wave Forecast

Hence different statistical techniques must be harnessed to assess extreme, or generally all possibly important, wind-wave-surge... climates. In our discussion we merely mention the simplest available and usable predictive tools.

To obtain the quantities characterizing the wind-induced waves, it is assumed that the controlling parameters of wind field are:

- average wind speed V_w at a certain height (z) above still water surface (usually $z = 10\ m$),
- wind fetch X,
- wind duration t.

Nondimensional characteristics are often selected for the computation of the effects of wave growth. The results are commonly presented in terms of the *fetch-limited* or *duration-limited* graphs. When both, the fetch and duration, are sufficiently large for significant wave height and period, H_s and T_s, to reach limiting values, these will become dependent only on the wind speed V_w and the condition of fully developed sea will arise. In order to attain the *fetch-limited* condition, a certain time t_{min} is needed.

At a location where no information on the wave climate is directly available, the characteristics may be estimated by application of existing wind data. At present, for the engineering practice the following empirical prediction methods are suggested:

–SMB method (Shore Protection Manual, 1973) with the following prediction "fetch graph" (providing the significant wave height and period H_s and T_s):

$$\frac{gH_s}{{V_w}^2} = 0.283 \tanh \left\{ 0.0125 \left(\frac{gX}{{V_w}^2} \right)^{0.42} \right\} \tag{1. 1}$$

and

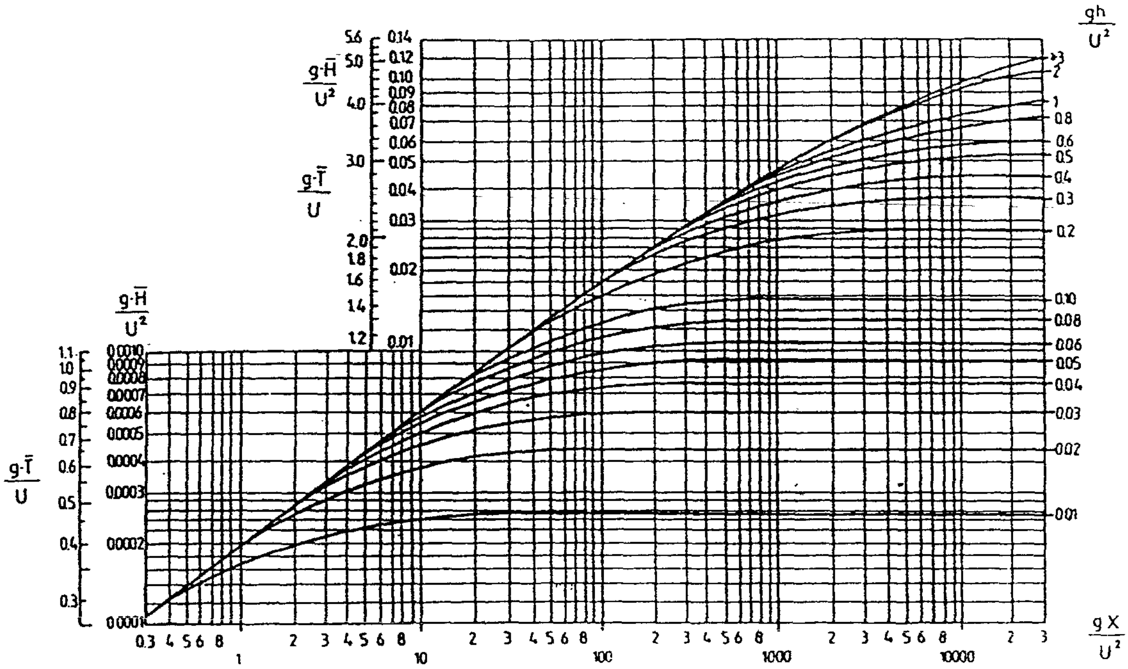


Figure 1.15. Wave Forecast Graph Recommended by Soviet Standards SNiP.

$$\frac{gT_s}{V_w} = 1.200 \tanh \left\{ 0.077 \left(\frac{gX}{V_w{}^2} \right)^{0.25} \right\} \tag{1. 2}$$

–Krylov method (Krylov et. al., 1976, yielding the mean wave parameters $\bar{H}$ and $\bar{T}$):

$$\frac{g\bar{H}}{V_w{}^2} = 0.16 \left\{ 1 - \left[1 + 0.006 \left(\frac{gX}{V_w{}^2} \right)^{0.5} \right]^{-2} \right\} \tag{1. 3}$$

as illustrated in Fig.1.15.

Joint probability functions for wave height and sea level are not available, at least as general guidelines or particular matrices for a given site. High water levels at a certain station may be caused by remote storms and set-up of water due to other reasons. At the same time, the local waves may be quite small. Yet it is quite likely that enormous waves are also generated at a rather low water level. Hence one may claim there is no clear-cut correlation between sea level and wave height. On the other hand, extreme water levels are often caused by the same atmospheric pressure systems which generate high storm waves. This would suggest a significant correlation of both factors. Hence the best way to find a way out is to construct an empirical correlation matrix basing on site data. In its absence, it seems reasonable that a certain design compromise is made between no correlation and significant correlation. However it must be realized that wind set-up, storm surges, and wave

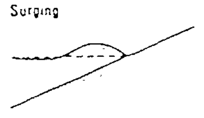
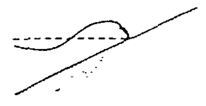
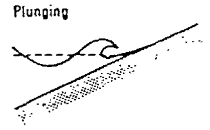
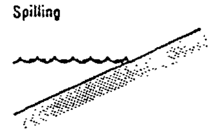
Type of breaking wave	Smooth slope	Slope with rip-rap protection
Surging 		$\xi>3.0$
Collapsing 	$\xi>3.3$	$2.0<\xi<3.0$
Plunging 	$0.5<\xi<3.3$	$\xi<2.0$
Spilling 	$\xi<0.5$	

Figure 1.16. Types of Wave Breaking.

set-up all share the common feature that they are governed by meteorological events, which are generally impossible to forecast accurately, although some simple formulae for wind set-up and wave set-up are available, while storm surge levels can be predicted by using mathematical modelling techniques. **Wave transformation** processes are very important in the coastal interactions. For the sake of brevity we are not dealing with them in this study report, and the reader is addressed to numerous relevant books and manuals. Since it is basic, and besides introduces the important quantity ξ, we are presenting the types of wave breaking (Fig.1.16), a phenomenon that has so many bearings on coastal effects. The quantity ξ is called Irribaren number but is also referred to as Battjes number or wave index, or breaking index, in various implementations with a variety of subscripts denoting deep water, as proposed originally by Irribaren, breaking area, as intended by Battjes, etc. Despite our restrictions we are however describing in more detail two of the most spectacular phenomena controlling the stability and overall dimensions of structures, viz. wave run-up and overtopping.

1.3.2 *Wave Run-up and Overtopping*

Introduction

For many coastal structures, the most important dimension is the height of the crest, which has to be related not only to the design still water level, but also to the wave action. Waves striking a coastal structure will **run up** its face to a certain elevation: if this is greater than the crest elevation, then a certain degree of overtopping will occur. With random waves, it is not possible to define a coastal structure crest level which will never be overtopped: however high the crest there is always a statistical chance that a severe storm or a rogue wave will occur which will give a run-up level greater than the crest level. The designer therefore has to specify the tolerable limit either for the percentage of waves which can be allowed to overtop the coastal structure, or for the volume or discharge of the overtopping water. In principle, quantifying the overtopping water is to be preferred, since it is this which governs the degree of flooding in the hinterland, and the amount of damage to the coastal structure, or to people or property behind the structure. In general however rather more information is available for quantifying wave run-up levels than for overtopping discharges.

Overtopping is defined as the transport of significant quantities of ocean water over the top of a seawall, either as greenwater, splash, or spray. Overtopping causes damage in several ways, by exerting direct vertical and horizontal forces,and by eroding material from behind walls.

In most coastal environments it is not practical to built a seawall that will not be overtopped during severe storm conditions. At many sites, cost is a limiting factor. For example, for a rip rap structure with a triangular cross section, and fixed seaward slope, the volume (and cost) of rock required is proportional to the square of the height. Where property behind the wall is at a relatively low elevation (less than 5 metres above Mean Lower Low Water), aesthetic and beach access problems may limit seawall height. Few coastal residents or cities are willing to build seawalls which will significantly block their view of the ocean.

As the wave retreats from the seawall, the water level falls below its still water value, reaching a minimum run-down just as the next wave arrives. For various reasons, for instance on slope revetments, the greatest potential for damage to a revetment occurs between the points of minimum drawdown and the still water line. Prediction of the minimum wave drawdown is therefore also important.

Wave Run-up

Wave run-up is always defined as the vertical distance R between still water level and the highest point reached on the beach, dune or coastal structure. The run-up level depends on details of the coastal structure, including the slope, roughness and porosity of the seaward face, and the dimensions of any berm which may be incorporated into the design. The run-up level also depends on the wave height, period, direction, and spectral width. For **plane smooth** slopes most authors agree that the relative run-up R_n/H_s is a function of the Irribarren number ξ

$$\xi = \tan\alpha/\sqrt{H_s/L_p} \qquad (1.\ 4)$$

For Irribarren numbers less than about 2.0 (seawall slopes typically flatter than $1:2$), a linear relationship generally exists (Pilarczyk 1987):

$$\frac{R_n}{H_s} = C_n\sqrt{2\pi}\xi \qquad (1.\ 5)$$

where C_n is a constant depending on the type of wave spectrum, and on the percentage exceedance n. For revetments, the design run-up level is usually taken as $R_{2\%}$, the level exceeded by 2 percent of the waves. From laboratory experiments, field measurements and theoretical considerations, different authors have estimated slightly different values for C_2, from about 0.55...0.6 for narrow band wave spectra to about 0.7...0.8 for broad band spectra.

Using $C_{2\%} = 0.70$ and wave steepness of about 5 % (typical storm value for the North Sea Coast) one obtains the so-called "Old Delft Formula" commonly used in the past for calculation of 2-% run-up ($R_{2\%}$) on the Dutch sea dikes, viz.

$$R_{2\%} = 8H_s \tan\alpha \qquad (1.\ 6)$$

which is valid for $\cot\alpha > 3\%$ and relatively smooth revetments.

As a safe approach it is recommended to use $C_{2\%} = 0.70$ for determining the run-up due to the wind-waves. In this case one has:

$$\frac{R_{2\%}}{H_s} = 1.75\xi_p \;\text{ for }\; \xi < 2.5 \;\text{ or }\; R_{2\%} = 3.5 \;\text{ for }\; \xi > 2.5 \qquad (1.\ 7)$$

For random waves with an Irribaren number greater than about 2.0 (relatively steep walls and/or long swell waves) there is a considerable and apparently random scatter in the run-up results, both among tests carried out by the same authors, and between different authors. Fig.1.17 shows the ranges of results. Ahrens (1987) published different equations for different revetment gradients based on his test results, although in fact the differences between the equations are less than the standard deviations in most cases.

The effective run-up on an inclined structure can be defined as

$$R_n = R_{ns}\nu_R\nu_B\nu_\beta \qquad (1.\ 8)$$

where

R_{ns} = run-up on smooth plane slopes, defined as the vertical height above still water level,

ν_R = reduction factor due to slope roughness and permeability,

ν_B = reduction factor due to berm

ν_β = reduction factor due to oblique wave attack. For rough impermeable slopes there is very little quantitative data on run-up of random waves. For regular waves, a roughness factor ν_R has been introduced as the ratio of the wave run-up level on the roughened slope to the run-up level on an otherwise identical smooth slope.

Table 1.1 shows the typical values: these can be used to gain first estimates of the run-up levels due to random waves. However it should be pointed out that the

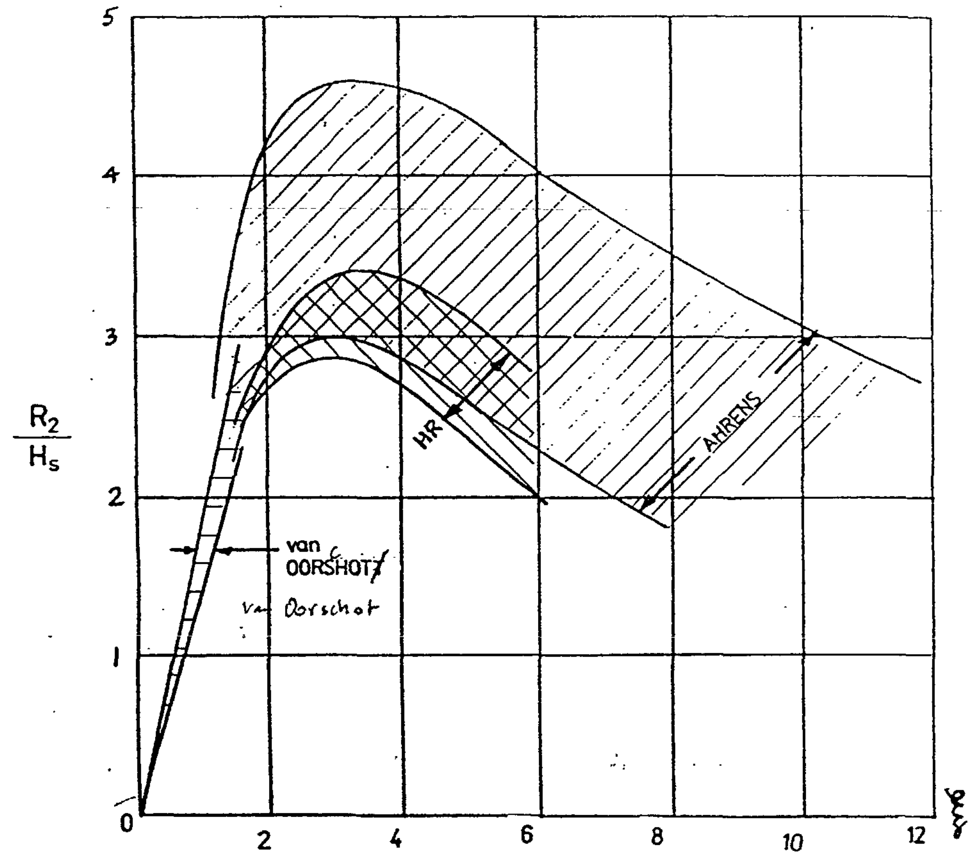


Figure 1.17. Run-up on Smooth Plane Slopes.

Table 1.1. Roughness Values for Various Types of Sea Wall Construction.

Seawall construction	Roughness value
Smooth, impermeable (e.g. asphalt, smooth concrete)	1.0
Stone blocks, pitched or mortared	0.95
Geotextile mat, grass mat, open stone-asphalt	0.95
Concrete blocks	0.9
Stone blocks, granite sets	0.85 to 0.9
Turf	0.85 to 0.9
Rough concrete	0.85
One layer of stone rubble on impermeable base	0.8
Stones set in cement, ragstone etc	0.75 to 0.8
Gravel, gabions	0.70
Dumped round stones	0.6 to 0.65
Two or more layers of rubble	0.5 to 0.6

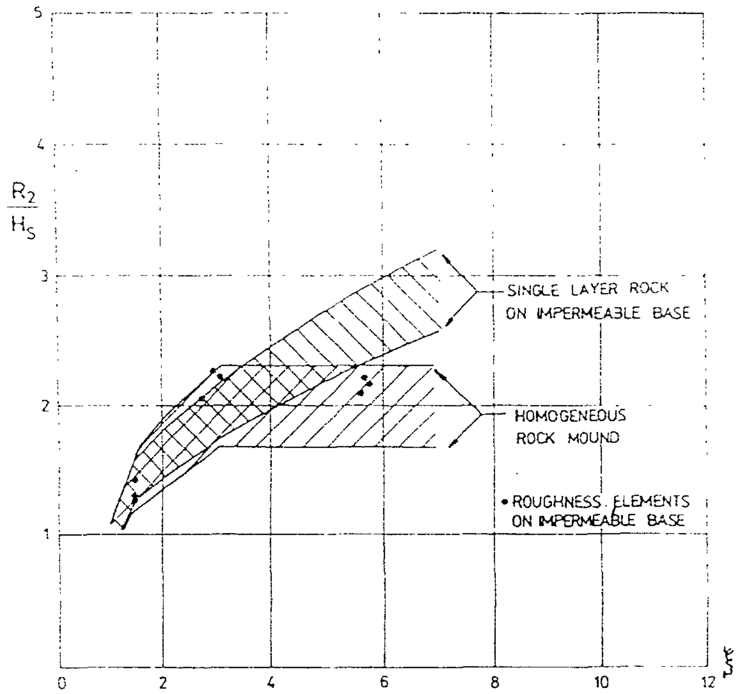


Figure 1.18. Run-up on Rough or Porous Plane Slopes.

roughness factor changes with wave height, wave period, and revetment slope. For example, Fig.1.18 shows the relative run-up levels for plane slopes with roughness elements, and for slopes consisting of a single layer of rock over an impermeable base. In both cases, the geometrical roughness was the same for all slopes but the run-up results show that the roughness factor ν_R increases as ξ increases (steeper slopes). The expected values of ν_R would be about 0.65 for the roughness elements used, and about 0.8 for the rock: at small values of ξ (shallow slopes) the roughness factor ν_R is less than expected, whereas at larger values of ξ (steep slopes) ν_R is greater than expected.

The most common type of **rough permeable** slope would be a rock revetment consisting of two or more layers of stone, for which Table 1.1 would indicate a roughness factor of about 0.5 to 0.6. However random wave tests have shown that the extra porosity makes very little difference to the run-up levels for ξ less than about 3, although at higher values the relative run-up becomes constant, Fig.1.18.

Introducing a **near horizontal berm** onto the front face of a revetment can make a substantial difference to the run-up levels, especially if the berm can be placed close to the design water level (h_b/H_s less than about 0.5). At this level, wider berms produce less wave run-up, although for berms which are wider than about $0.3L_o$ there is very little further reduction. For berms at this optimum depth and width, the wave run-up levels on plane slopes may be reduced by the following factors (Pilarczyk 1987):

Slope	ν_B at $h_B < 0.5H_s$
$1:5$ to $1:7$	0.75 to 0.80
$1:4$	0.6 to 0.7
$1:3$	0.5 to 0.6

All the above methods of estimating wave run-up apply to the situation where waves strike the structure orthogonally. Until comparatively recently this was thought to produce the worst case, with wave run-up reducing quite rapidly for oblique wave attack. However, some recent research studies have indicated that for angles of incidence between 0 and about 40° there is very little change in run-up levels, with even a slight increase at angles of about 15...20°. The actual increase depends on the structure slope, but Tautenhain et al (198.) have suggested an expression of the form

$$\gamma_\beta = cos\beta(2 - cos^3\beta)^{1/3} \tag{1. 9}$$

where β is the angle between the line of the structure and the wave crests. This expression peaks at a value of 1.09 at an angle of about 22°. Alternatively, an oblique wave attack, at an angle β can be roughly taken into account by

$$\nu_\beta = cos(\beta - 10^o) \quad for \quad \beta < 0.65^o \tag{1. 10}$$

For $\beta > 65^o$ one has $R_n > H_s$ (but not less than H_s !) (N.B. β is reduced by 10^o on account of variation of β).

For bermed slopes, slopes with roughness elements, and rough porous slopes it is likely that orthogonal waves do indeed give the wave run-up. For oblique waves the length of the roughened slope, or the width of the berm, does effectively increase, causing a reduction in wave run-up.

Note: *Depending on the wave spectrum, i.e. the anticipated maximum wave height, and the type and permeability of revetment, type of subgrade, the run-up can vary substantially. Hence the slope protection has to be more or less extended. For particular cases model investigation may provide proper answer.*

Run-up on a shingle beach is depicted in Fig.1.19.

Run-down

The only published data which is available for run-down under random waves relates to plane, smooth slopes. In these situations the lower limit of slope area attack by waves (where primary protection is necessary) can be defined roughly as (PIANC 1987):

$$\frac{R_d}{H_s} = (0.8\xi + 0.5) \quad for \quad \xi < 2.5 \tag{1. 11}$$

and

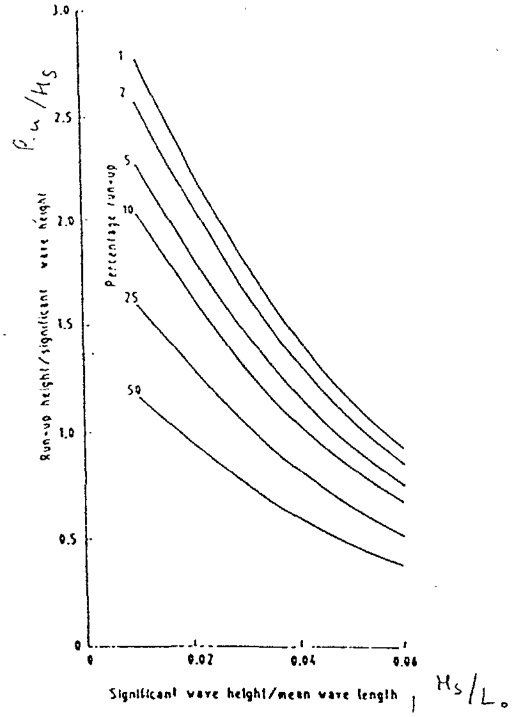


Figure 1.19. Run-up on Shingle Beaches, Owen (1989).

$$\frac{R_d}{H_s} = 2.5 \quad for \quad \xi > 2.5 \tag{1. 12}$$

The addition of berms, or of slope roughness, is likely to make a significant difference to the run-down level, and for these structures model tests would be necessary to define the lower limit of the required slope protection.

Overtopping Discharges

Standard run-up calculations for seawalls (or any other structure) typically consider only the frequency of overtopping by 'green water'. The height of this run-up is usually calculated using empirical or theoretical formulae based on water depth, beach slope, significant wave height, wave period, maximum expected sea level, and the type of structure involved. Unfortunately, these calculations often ignore the potential effects of overtopping by wave splash, and the erodibility of materials behind and above the seawall.

With random waves, it is not possible to define a structure crest level which will never be overtopped: however high the structure, there is always a statistical chance that a severe storm or a rogue wave will occur which will give a run-up level greater than the crest level.

Usually therefore the designer has to consider the consequences of the structure being overtopped. Principally, this means estimating the volume or the discharge of the overtopping water, since it is this which governs the degree of flooding in the

Table 1.2. Threshold Overtopping Discharge for Damage to Sea Walls

Type of seawall and construction	Threshold overtopping discharge $m^3/s.m$
Embankment type seawall:	
Crown and back slope unprotected (eg.clay, compacted soil; grassed)	$2 \cdot 10^{-3}$
Crown protected,back slope unprotected	$2 \cdot 10^{-2}$
Crown and back slope protected	$5 \cdot 10^{-2}$
Revetment-type seawall:	
Apron (promenade) unpaved	$5 \cdot 10^{-2}$
Apron (promenade) paved	$2 \cdot 10^{-1}$

Note: To be used as general guidelines only.

hinterland, and the amount of damage to the structure or to people or property behind the structure. For the design of the structure, a "tolerable" overtopping discharge has to be defined. Depending on the nature of the particular scheme, this tolerable or design discharge has to satisfy various requirements including

(a) The stability of the crest and back face of the structure

(b) The discharge capacity of any drainage channels behind the structure

(c) The total volume available for storage of flood waters behind the structure, or alternatively the depth and extent of flooding which can be tolerated

(d) The possibility of damage to buildings or vehicles behind the structure, or of injury to members of the public.

Items (b) and (c) above are probably self-explanatory, except to note that the discharge capacity of the channels has to be related primarily to the overtopping discharge at the peak of the storm, whereas the degree of flooding is related to the total volume of water overtopping the structure for the total duration of the storm. The stability of the structure is much more difficult to determine, since it is based on so many different factors. However, Table 1.2 gives general guidelines on the tolerable discharges, which should not be exceed if damage is to be avoided. Similarly, Table 1.3 provides figures for discharge values not to be exceeded to avoid damage or injury to buildings, vehicles or people behind the structure.

Estimating overtopping discharge

All the available methods for predicting overtopping discharge relate only to "green water" (or solid) overtopping, and do not include the effects of "white water" or spray. This is mainly because spray overtopping cannot be successfully reproduced during model tests, and most predictive equations are based on such tests. Because wind-driven spray cannot be quantified, efforts should be made to design a structure

Table 1.3. Threshold Overtopping Discharge for Damage or Injury to Buildings, Vehicles or Personnel

Category	Threshold overtopping discharge $m^3/s/m$
For a person to walk immediately behind the seawall:	
with little discomfort	$4 \cdot 10^{-6}$
with little danger	$3 \cdot 10^{-5}$
For a vehicle to pass immediately behind the seawall:	
at high speed	$1 \cdot 10^{-6}$
at low speed	$2 \cdot 10^{-5}$
For a house located immediately behind the seawall:	
to suffer no damage	$1 \cdot 10^{-6}$
to suffer no structural damage, although experiencing partial damage to windows and doors	$3 \cdot 10^{-5}$

Notes: (1) To be used as general guidelines only

(2) At 10 m behind rather than immediately behind the seawall the above discharges can be increased by a factor of about 10.

that is not too susceptible to spray generation.

For most revetments, the mean overtopping discharge caused by random waves can be predicted from an equation of the form

$$q_* = A \cdot exp(-BR_*) \tag{1. 13}$$

where dimensionless discharge

$$q_* = \frac{\bar{q}}{(gH_S^3)^{1/2}}(\frac{S}{2\pi})^{1/2} \tag{1. 14}$$

dimensionless freeboard

$$R_* = \frac{R_c}{H_s}(\frac{S}{2\pi})^{1/2} \tag{1. 15}$$

wave steepness

$$S = \frac{2\pi H_s}{gT^2} \tag{1. 16}$$

Table 1.4. Values of the Coefficients A and B for Simple Sea Walls

Seawall slope	A	B
1:1	$7.94 \cdot 10^{-3}$	20.12
1:1.5 *	$1.02 \cdot 10^{-2}$	20.12
1:2	$1.25 \cdot 10^{-2}$	22.06
1:2.5 *	$1.45 \cdot 10^{-2}$	26.1
1:3 *	$1.63 \cdot 10^{-2}$	31.9
1:3.5 *	$1.78 \cdot 10^{-2}$	38.9
1:4	$1.92 \cdot 10^{-2}$	48.96
1:4.5 *	$2.15 \cdot 10^{-2}$	55.7
1:5 *	$2.5 \cdot 10^{-2}$	65.2

Bold type - values determined from model tests

Asterisk - values derived by interpolation based on published run-up data.

where R_c and $\bar{q}$ are respectively the actual freeboard (difference between crest elevation and still water elevation) and the actual mean overtopping discharge (expressed as volume/time/unit length of structure).

A and B are dimensionless coefficients whose values depend on the geometrical profile of the revetment, together with the roughness and porosity of its seaward face. To a lesser extent, the coefficients also depend on the width of the wave energy spectrum.

For **simple, impervious, smooth** sloping revetments, the values of A and B determined from model test are given in Table 1.4. This table also includes interpolated values for other slopes not examined by model tests. It is interesting to note that there is very little difference in overtopping performance between 1 : 1 and 1 : 2 sloping revetments.

For **simple rough slopes**, (porous or non-porous) theoretical considerations suggest that the overtopping discharge can be predicted from the equation

$$Q_* = Aexp(-BR_*/\nu_R) \qquad (1.\ 17)$$

(where ν_R is taken from Table 1.1). However, recent studies indicate that the discharge estimated from this equation is probably conservative, i.e. roughening the seaward slope of the revetment reduces the overtopping discharge more then expected. This equation should therefore only be used for initial estimating purposes, and specific model tests carried out to refine the overtopping prediction.

For **smooth, non-porous, bermed** revetments, the values of the coefficient A and B can be found from literature . The effect of introducing a near-horizontal berm onto the front frace of the revetment depends on the width of the berm relative to the wave length,B/L_o, on the berm elevation relative to the still water line and the wave height, h_B/H_s and to some extent also on the relative freeboard R_*. For very low freeboard (R_* less than about 0.04) introducing a berm can actually increase overtopping discharge, because of pronouced wave set-up on the horizontal berm.

However, for most practical revetments the effect of the berm is to reduce overtopping discharge. Berms located close to the still water line give the greatest reduction in discharge, with the effect diminishing rapidly when h_B/H_s is greater than about 0.5. Increasing the width of the berm also reduces the overtopping discharge: initially this effect increases rapidly with berm width, but beyond a certain width the effect is very much less marked. The optimum berm width depends on the berm elevation: for a berm close to still water level a berm width of about $0.15L_o$ will give about 80% reduction in overtopping discharge: Fig.1.20 shows the berm width required to give the same reduction for other berm elevations.

For **rough, impervious bermed** revetments recent tests have indicated that introducing the published roughness coefficient "r" into the overtopping equation results in a very serious overestimate of the overtopping discharge, and this is also likely to be the case for **rough porous bermed** revetments. Where the overtopping discharge is critical in design, for these types of revetment the overtopping should therefore be quantified from model tests.

The values of the coefficients A and B which are quoted above relate to revetments which have a very narrow crest, without any form of vertical or recurved wave return wall mounted on the crest. The overtopping discharge can be reduced slightly by increasing the crest width, since some of the water reaching the crest will drain seaward. The discharge can also be reduced significantly by adding a wave return wall, whose purpose is to intercept the overtopping jet of water, and deflect it back out to sea. The effectiveness of the return wall depends very much on its geometry, and on its dimensions relative to the depth of the overtopping water. Exact quantification of the reduction in overtopping which is achieved can only be assessed by model testing, but reductions of up to 90% are often possible. In general, it is preferable that the water is returned seaward at a very mild angle above the horizontal: if the water is deflected vertically a moderate onshore wind can easily transport it landward over the revetment.

Overtopping discharge is also affected by the angle between the wave crests and the seawall. Contrary to expectations, the worst overtopping does not necessarily occur under normal wave attack. For relatively steep, simple, smooth slopes, and for revetments incorporating rather narrow berms, the worst overtopping occurs at an angle of about 10...20° off normal, in some cases reaching as much as 65%

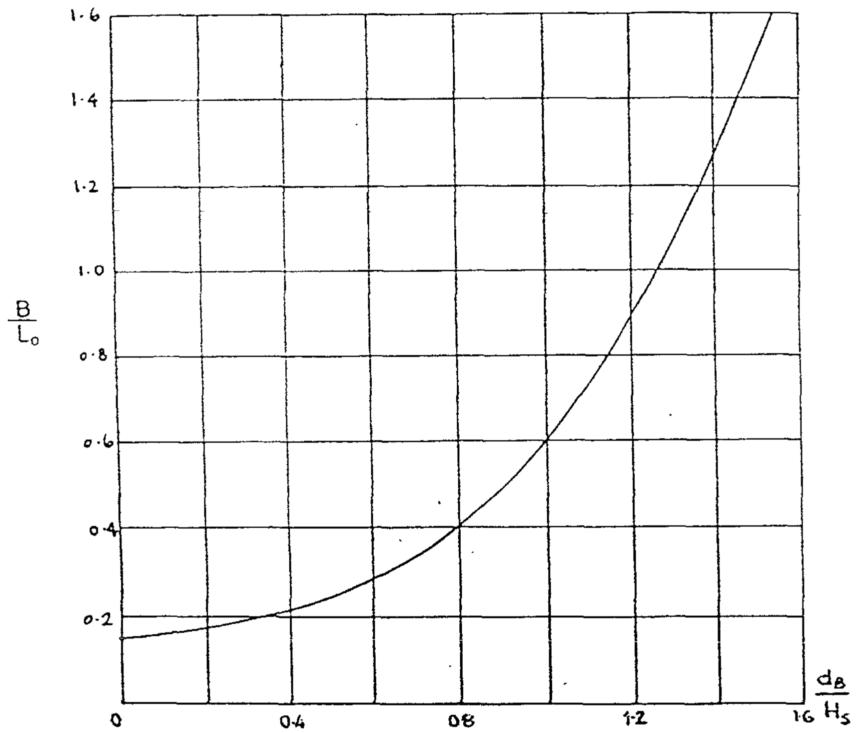


Figure 1.20. Optimum Berm Width B for Reduced Overtopping Discharge.

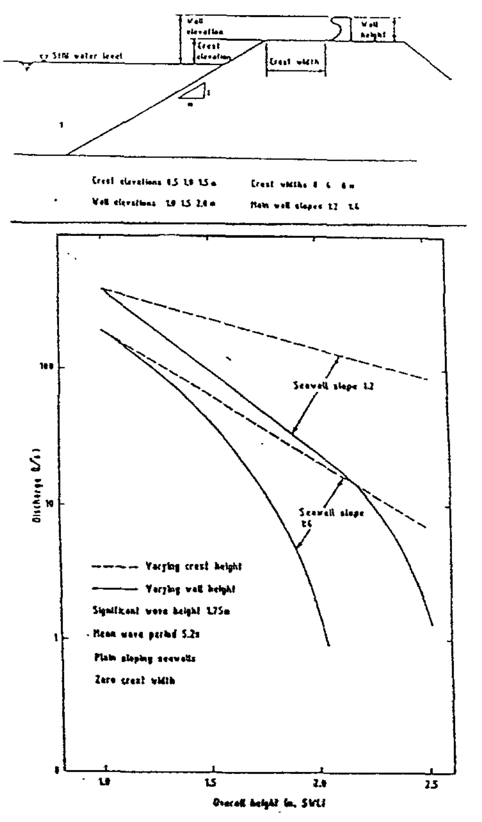


Figure 1.21. Overtopping on Recurved Walls, Owen (1989).

greater discharge. For those revetments, the overtopping discharge reduces below that occuring 0° only for directions greater than about 30°. However, recent tests have indicated that this general effect applies only for long crested random waves, such as might occur in relatively shallow water: for short-crested random waves the overtopping discharge stays reasonably constant over the directions 0 to 30°, before falling away at larger angles. For revetments with a rather mild slope, or wide, shallow berm width or the effective length of roughened slope, and therefore reduces the overtopping discharge consistently.

Goda (1985) provides extensive material on overtopping caused by a wide variety of structures, mostly port and harbour ones, in the presence of random waves. An example for a particular case of recurved wall configuration is depicted in Fig.1.21.

Vertical Forces due to Overtopping

Both greenwater and wave splash can exert strong upward, vertical forces on structures and materials near the crests of seawalls, especially those with vertical faces. Where moderately deep water lies in front of a vertical seawall, even relatively small (3 to 4 feet high) breaking waves can send jets of water up to twenty feet in the air. In high-surf conditions, the vertical forces exerted by such jets have destroyed overhanging decks and floors of ocean-front homes.

At several sites, vertical wave forces have lifted rip rap and other rocks up to one to two feet across from the base of vertical walls and erosion scarps, and thrown them inland. Where vertical seawalls or rock cliffs face deep water, waves have broken off immense blocks of stone (up to 50 cu. ft.) from the crest of the wall or bluff, and rolled them landward.

Horizontal Forces due to Overtopping

Water overtopping seawalls in the form of green water or splash can also exert significant horizontal forces on structures or materials behind the wall. Matthews (1934) documents damage to buildings twenty feet inland from vertical masonry seawalls. Where low seawalls are overtopped by white-water bores, these bores may cause great flooding and property damage hundreds of feet inland. This damage may be increased if floating logs and debris are floated or thrown over the wall.

Rip rap walls are generally assumed to have a lower run-up coefficient than smooth concrete or timber walls. However, because of their sloping seaward face, splash or green-water overtopping, rip rap walls may have a greater horizontal component of velocity than splash over vertical walls of the same height. Wave splash has been observed to travel further inland at locations where rip rap has been placed than where waves hit vertical bluffs. Waves overtopping high rip rap (19 feet above MLLW) knocked out entire home fronts at Via Gaviota in northern Monterey Bay, during storms in 1983 , cf. Fulton-Bennet and Griggs (1987).

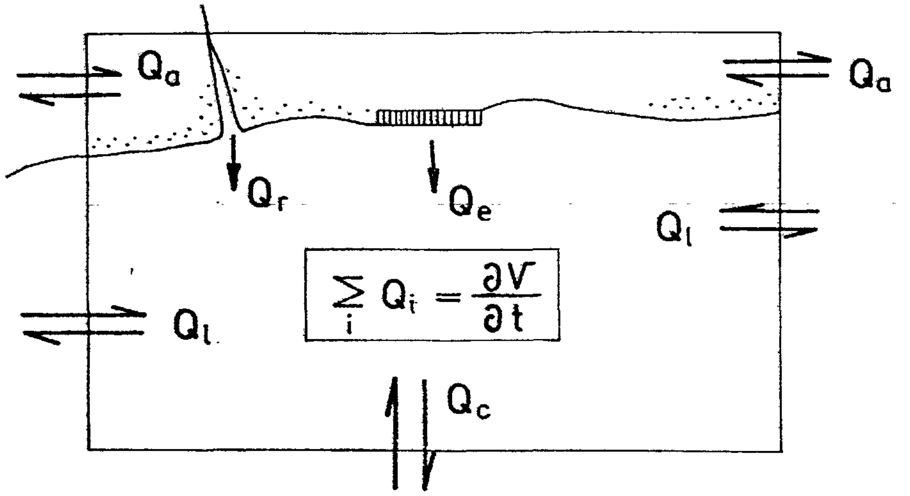


Figure 1.22. Sediment Budget in a Coastal Cell.

1.3.3 *Coastal morphology*

Introduction

Any coastline, natural or artificial, is subjected to coastal processes, of which some are depicted in Fig.1.22. The sediment compartment shown may incorporate easily erodible segments (such as sandy beaches) and more resistant segments (such as cliffs or coastal structures). Sediment budget within the compartment includes longshore and cross-shore sediment tranport rates, Q_l and Q_c, aeolian transport rate Q_a, sediment discharge from rivers Q_r, and Q_e, the rate of sediment eroded about cliff or structure. The sum of all Q components within the compartment determines the rate of accretion or erosion, i.e. the temporal variation of the volume of sediment, V. Hence, modelling of coastal evolution requires the knowledge of respective transport formulae, together with the intervening waves and currents, which control both sedimentation and operation of coastal structures.

Shore evolution takes place in different time scales (Fig.1.23). Discussed below are short-term changes (days to months), longer processes (years), and the effect of coastal structures. Although he must be aware of long-term processes, such as eustatic changes and sea level rise (e.g. caused by the greenhouse effect), and many other geological and climatological factors, the coastal engineer is usually concerned about time scales of the order of 100 years, i.e. his own and his grandchildren's playground. Of course, this does not prevent him from assigning long return times, or very low probabilities, in his design on extreme events, occurring once per one thousand or ten thousand *statistical* years.

One may distinguish **far-field** effects, with length scales exceeding a characteristic

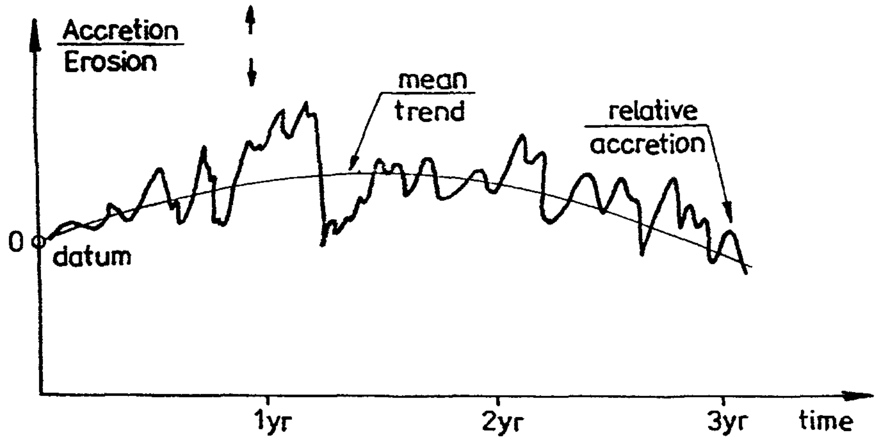


Figure 1.23. Position of Coastal Feature (*e.g. Shoreline*): Multiscale Variability.

dimension of the structure or coastal feature considered, and **near-field** processes of smaller linear scales. The former are discussed in Chapter 2 while the latter are dealt with primarily in Chapter 3.

Shore evolution in any scale depends primarily on coastal climate (waves, currents, etc. or **hydraulic boundary conditions**), properties of shore **topography** and bed **sediment**, and features of coastal **structures**.

Beach profile

The cross-shore profile of a nearshore zone, referred to as a **beach** or **shore profile**, must be known for at least two objectives: (1) assessment of shore evolution and (2) control of the effect of a coastal structure on coastline.

Dean (1977) postulated the following time-averaged two-dimensional

$$h = \alpha_n x^m \tag{1. 18}$$

in which h is the depth below still water line in terms of the distance x seaward from the shoreline. A later modification, with a rectilinear stretch at shoreline is depicted in Fig.1.24. The average exponent m has been shown about 2/3, in agreement with the theory assuming uniform energy dissipation across surf zone. Dean (1987) noted a close fitting of data on α_n and the settling velocity W_s (cm/s):

$$\alpha_n = 0.067 W_s^{0.44}; \alpha_n = m^{1/3} \tag{1. 19}$$

Vellinga(1986) has proposed

$$\alpha_n \approx 0.39 W_s^{0.44} \tag{1. 20}$$

Boon and Green (1988) found that the parameter α_n may be readily calculated from the beach slope $\tan\beta$ (subaqueous nearshore slope at shoreline, which may be replaced by subaerial beach face slope on steep, highly concave barless beaches):

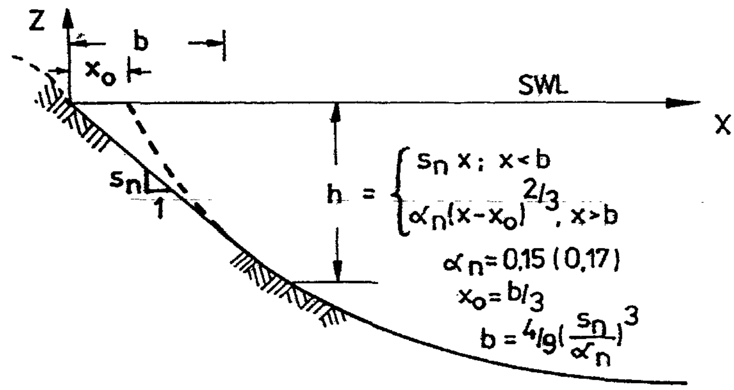


Figure 1.24. Dean Profile.

$$\tan\beta = \alpha_n^{1/m} \tag{1. 21}$$

The Caribbean beaches investigated by Boon and Green (low-energy ones consisting mainly of skeletal calcium carbonate sands of marine origin) display average m = 0.5 and good correlation of α_n with wave steepness:

$$(\alpha_n)^{1/m} = \frac{0.013}{H_b^2/gdT^2} + 0.12 \tag{1. 22}$$

in which
- H_b, T = breaker height and period,
- d = mean grain diameter,
- g = acceleration due to gravity.

Close agreement with laboratory findings of Sunamura (1984) is noteworthy.

Schematisation of shingle beach profile is depicted in Fig.1.25. Powell (1990) argues that the profile can be described by three distinct curves; the resulting nine predictors are correlated with the available model data, so that the shingle profile can be predicted for any set of input variables (although for normal wave attack only at present). Owen (1989) notes that the height of the shingle beach crest and the level of wave run-up depend only on the wave height and period.

Storm waves transport beach material offshore and form an underwater bar. Waves of low steepness, between storms, give rise to beach accretion and generation of a berm. Earlier laboratory studies emphasized the role of plunging breakers in the production of bars and determined the role played by wave steepness and beach slope, cf. Fig.1.26. Recent experiments in a large wave flume (CRIEPI, Horikawa, 1988) demonstrated that spilling breakers also create bars. The CRIEPI experiments relate the water depth of both bar crest and bar trough, h_c and h_t to the breaker height H_b

$$h_c = 0.59H_b = 0.59h_t \tag{1. 23}$$

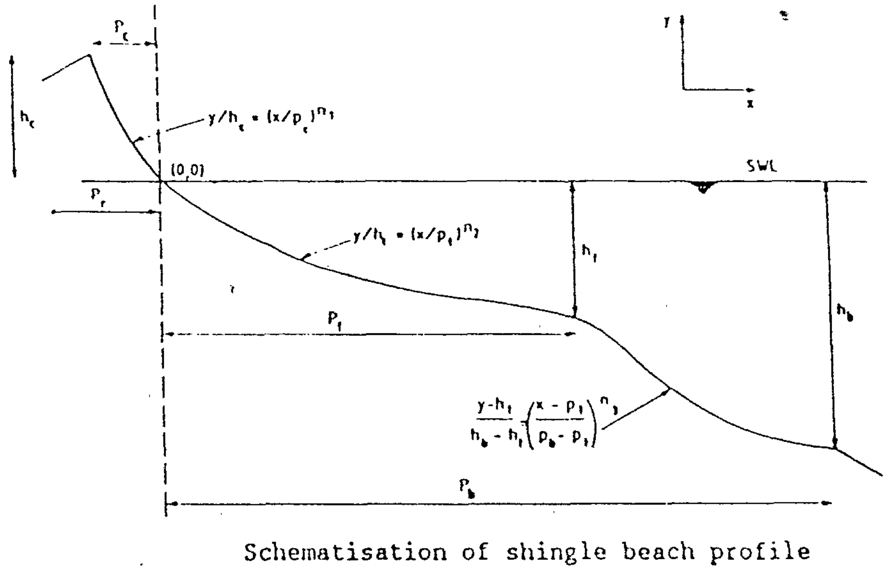


Figure 1.25. Shingle Beach Profile, Owen (1989).

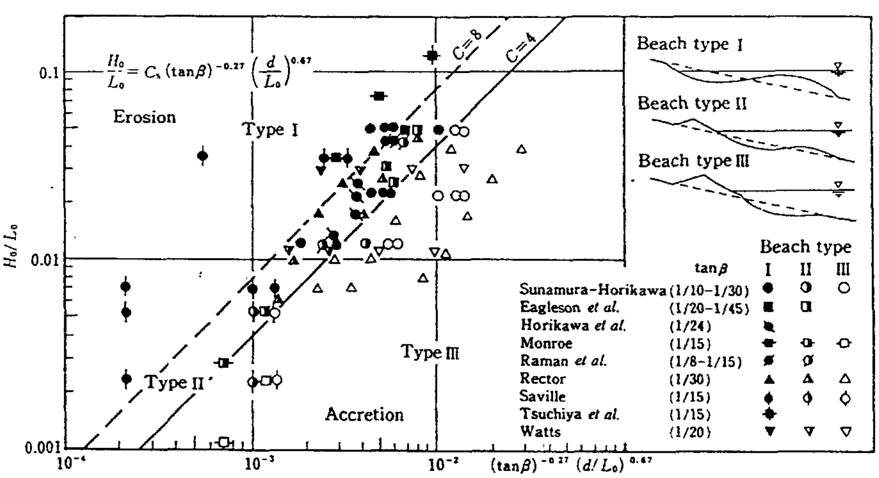


Figure 1.26. Classification of Sandy Beach Profile, Sunamura & Horikawa (1974).

There are other theories linking generation of bars to longshore currents, edge waves, Bragg resonance etc.

Short-term beach changes

Again, a short "static" outline is provided in this subsection. More details on the dynamical interactions of water, sea bed and structures are given in Sec.2.7.

Three-dimensional models of shore evolution have been proposed by Sonu (1973), Short (1978, 1979), Davis and Fox (1972, 1975), Sasaki (1983) and others. Some of these studies have been synthesized by Sunamura (1985) in a model summarized by Horikawa (1988), cf. Fig.1.27. This model can be applied to beaches of moderate-to-high energy environment, microtidal range (i2 m), moderate nearshore bottom slopes (1/50 - 1/200), fine-to- coarse-grained material (0.1 - 2 mm in diameter), and the dominant shore normal sediment-transport regime. The inner bar is reproduced only in zones with multiple bars.

The model consists of eight topographical stages; two limiting stages, erosional and accretionary, and six transitory stages. A dimensionless parameter K_*, is employed here to explain stage movement through the model

$$K_* = \bar{H}_b^2 / g\bar{T}^2 h \tag{1. 24}$$

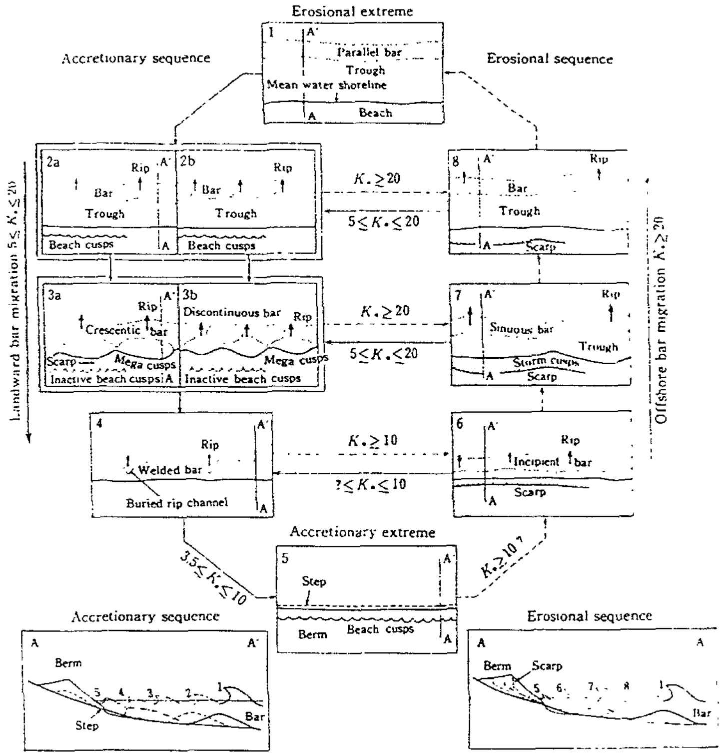
where $\bar{H}_b$ and $\bar{T}$ are the daily average values of the breaker and wave period.

The stages and associated shore transformations, along with characteristic morphological features (bar system, cusps, mega cuspes, storm cusps, berm-step systems, and beach-face slopes) are described by Horikawa (1988).

The dynamic response characteristics of the beach system, recently examined by Wright and Short (1984) and Wright, Short, and Green (1985) on the basis of extensive data on Australian coasts, are instructive. The presence of (1) morphological hysteresis and (2) time lag of beach stage occurence to input waves elucidated and six representative beach stages (called "beach states") were defined using the Dean parameter H_b/W_sT. A heuristic model for prediction of beach stage change was also proposed by Wright and Short (1984) and Wright, Short, and Green (1985).

Mathematical models become increasingly important in prediction of waves, currents, sediment transport and shore evolution. In view of the multiscale phenomena, depicted in Fig.1.23, time-dependent 3-D models are hardly feasible. Therefore one may recommended a semi-analytical quasi-3D model by de Vriend and Ribberink (1988) which describes tidal motion, waves, nearshore currents and sediment transport in complex coastal areas; although, at present the prediction is not better than that provided by the 2-DH version of the model.

Protected (or engineered) coastlines may be modelled in a variety of ways. A degree of sophistication reflected in a mathematical model depends i.a. on desired accuracy. Complex 3-D models may be substituted by simpler one-line or multi-line models using sets of equations for the geometry of one or more characteristic



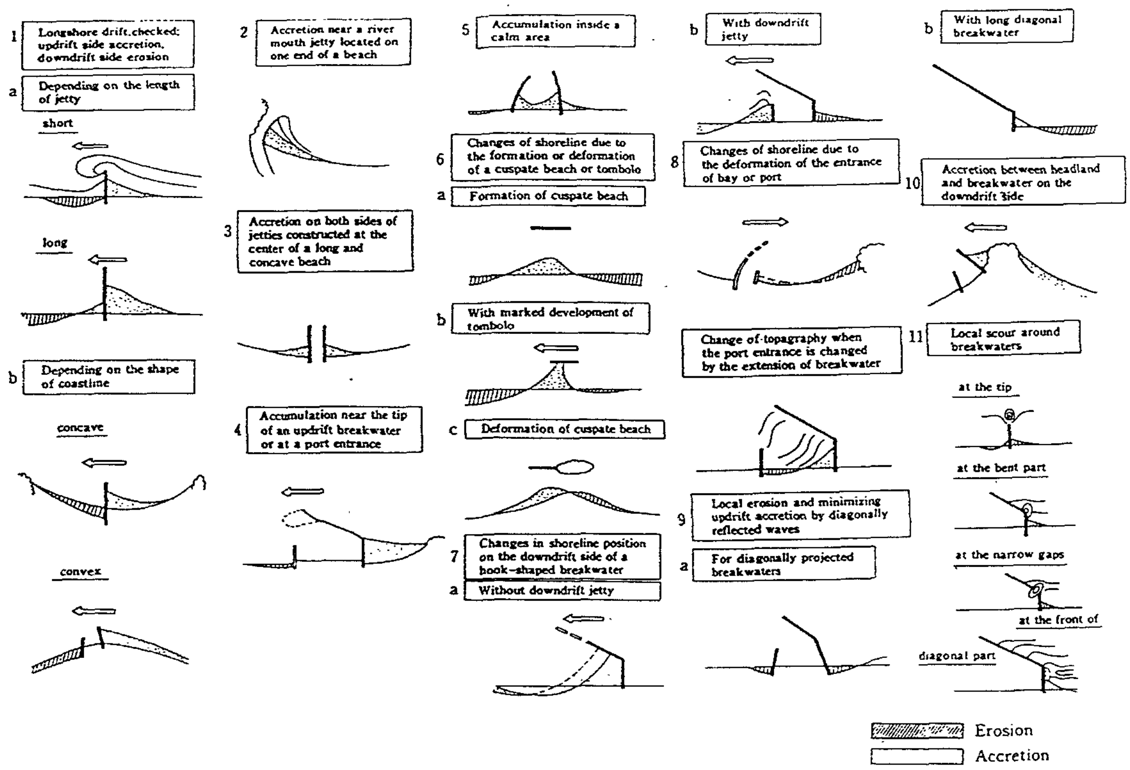


Figure 1.27. Beach Transformation Modelling, Horikawa (1988) (top)

Figure 1.28. Classification of Morphological Changes due to Presence of Stuctures, Tanaka (1983); *Arrows indicate the direction of net sediment transport* (bottom).

features of shore profile, sediment transport rate, wave diffraction etc. A certain classification of shore evolution models is given by Kraus (1983).

From among the 3-D models available one may recommend Perlin and Dean (1985). Multi-line models are well represented by Kriebel and Dean (1985). Hanson (1987) presented a synthetical numerical model referred to as Genesis, which provides a selection of potentials for modelling about structures. Boundary conditions at seawalls are discussed by Hanson and Kraus (1986). Evolution patterns, from straight shoreline at $t = 0$, due to a series of converging and diverging waves at a seawall and other structures are predicted. Steetzel (1987) proposed a model for beach changes near dune revetments. Mc Dougal et al (1987) provide simple equations for the landward encroachment of sea and the size of the eroded area on the downdrift side of a seawall. These and other models are discussed more comprehensively in Section 2.7.

Long-term shore evolution

Long-term morphological effects may be attributed to the gross variability of coastal climate and to islands, headlands, promontories and other structures, natural or artificial, which bring about large-scale variation in depositional and erosional patterns. Some natural features are illustrated in Fig.1.28.

1.4 MORE ABOUT STRUCTURES AND THEIR FUNCTIONS

Many costal structures may have some common features with regard to functional requirements, design formulae and construction methods. Therefore it is instructive to examine more closely certain types of these structures.

1.4.1 *Longshore Structures*

Seawalls and kindred structures

Seawalls (beach walls) have been constructed in a wide variety of materials and cross-sections. With respect to the cross-section the most common types of seawalls are listed below. Monolithic and sheetpile type seawalls are widely used throughout the world as rigid structures. Both concepts, rigid and flexible structures, such as flexible revetments, a modern defence concept, because of their typical wave reflecting characteristics, usually need a considerable flexible berm to reduce reflection and the hazard of scour. The following types can be distinguished:

- monolitic seawalls;
- sheet pile seawalls;
- palisade-filled seawalls
- rubble mound seawalls;

- composite-type seawalls (monolithic, on rock foundation);
- random placed sloping rock (natural or artificial), covering a core of erosive material;
- as above, with regular placed rock elements,
- bituminous systems;
- gabions, stone mattresses;
- berm type seawalls (characteristic alternative geometry for systems listed above).

An overview of these basic concepts is provided in Fig.1.29 and Fig.1.30.

Seawalls that protect interior waters against storm surges are commonly known as **barriers, sea dykes** or **storm dykes**. For economic reasons barriers can be constructed largely with natural materials. For elements with specific functions also artificial and industrial materials are applied. Examples of the latter are moving elements as turning and sliding doors, which are often made of steel. In particular, in deeper water a considerable volume of material is involved, which often makes natural (rock) materials competitive. The principal barrier concepts are the same as for seawalls. Two major categories are permeable and impermeable designs. Permeability can be controlled by means of materials chosen or by special measures.

In **breakwater** design, similar concepts may be applicable as for seawalls. Breakwaters can be **shore-connected** or **detached, emerging** or **submerged, longshore** or **oblique**. They may also appear in various combinations and systems, much as other types of structures. Typical functions and construction methods however will in general result in respective differences versus seawalls, for example in geometry and size of material elements. Breakwaters are mainly applied for beach protection and to reduce wave disturbance in harbours and entrance channels.

Depending on their specific function, breakwaters may have strongly variable alignment and cross sections. Both the alignment and the cross section of a breakwater affect, to a certain extent, the hydraulic loading of the cover layer. Moreover, the bulk volume of a breakwater is mainly determined by these geometrical characteristics. In many cases breakwaters are exposed to relatively heavy wave loadings because of their exposure. Given the strong dependence of the required armour strength on the wave height, often high demands must be made upon the armour elements, construction techniques and equipment. Depending upon the specific function of the breakwater, overtopping and wave transmission may be allowed or not, which has important consequences for the design of the structure. The above concepts can also be applied to breakwaters. For a detailed description of all aspects of breakwater design one can refer to the report of PIANC WG12.

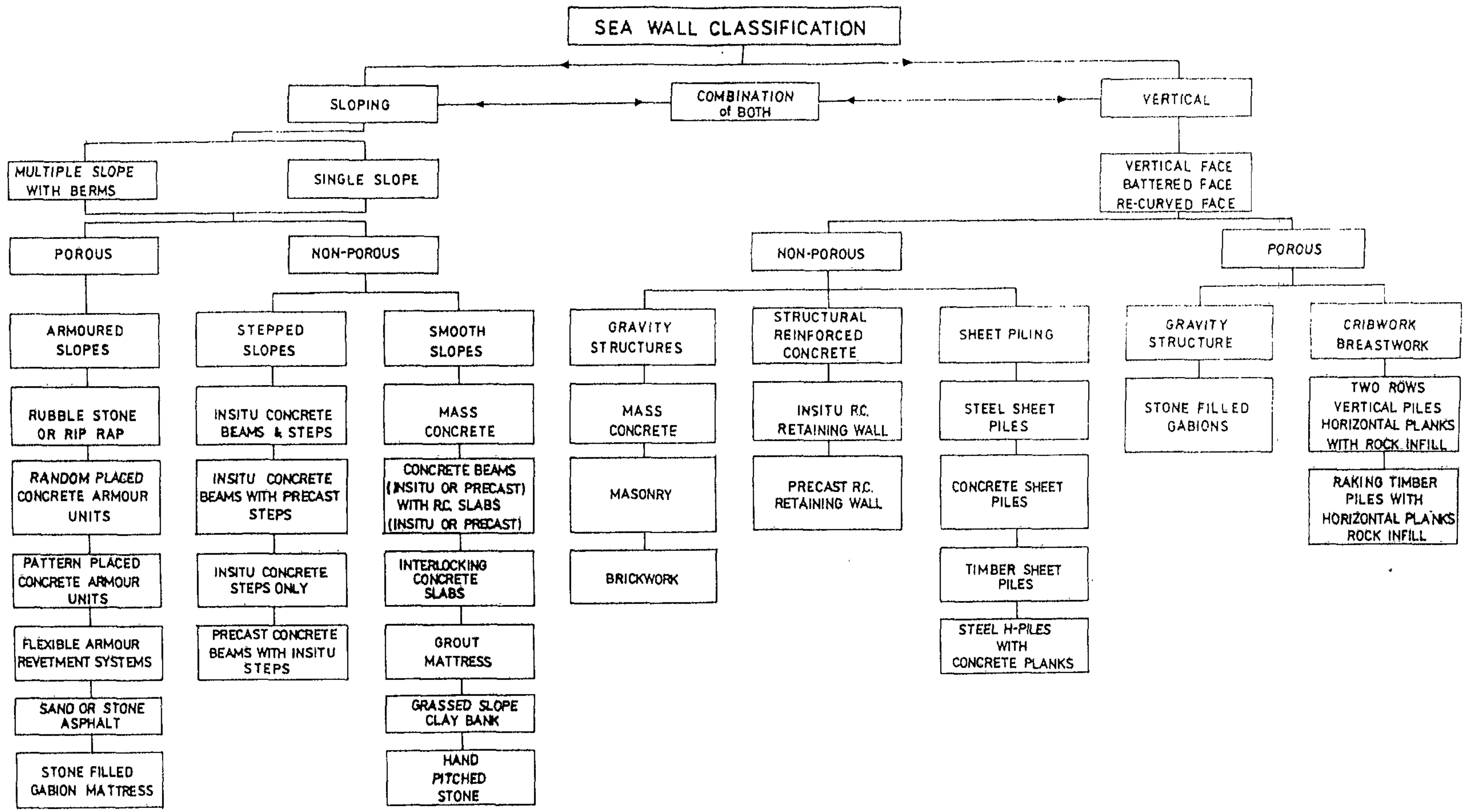


Figure 1.29. Classification of Sea Walls, Barrett et al. (1967).

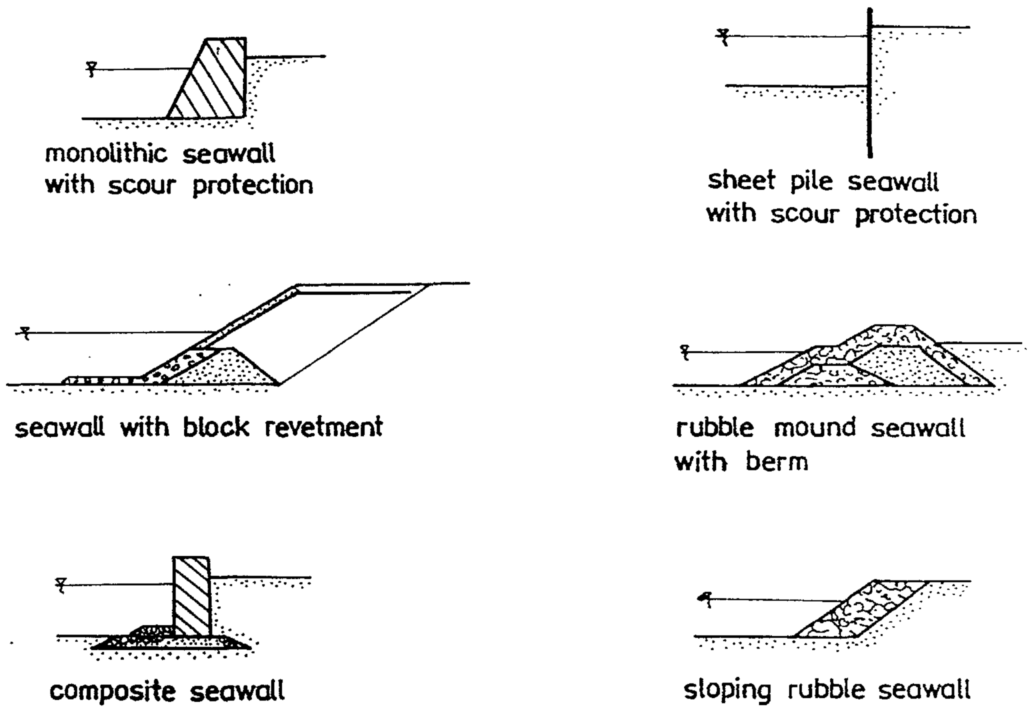


Figure 1.30. Basic Sea Wall Concepts.

Flexible revetment systems deserve special attention. A revetment system must be designed as an integrated system of cover (top) layer, filter and base (core) material. In general, a revetment system will consist of a number of layers, the principal of which are the cover layer, the filter layer(s) and a separation layer, which can be a geotextile or equivalent substitute.

Changes in one of these elements can have an important effect upon the performance of the revetment. The protective function of the revetment and its required flexibility define the range of concepts, that can be used in the design.

The cover or armour layer is the major protection of the structure and should resist external and internal loadings. Strength against external loadings can primarily be provided for by a sufficient weight of the armour elements .

The resistance against internal loadings (uplift pressures) depends to a large extent on the permeability ratio of cover- and filter layer. For a given wave loading uplift pressures can be reduced by lowering the permeability of the cover layer relative to filter permeability.

Further, the permeability of the core will also affect the stability of the cover layer. Additional stability of the cover layer can be obtained by friction-, interlocking- or tensile forces. These forces may act between the elements of the armour layer and between the armour elements and the underlayers. Most of the artificial systems have been designed deliberately to mobilize these additional forces.

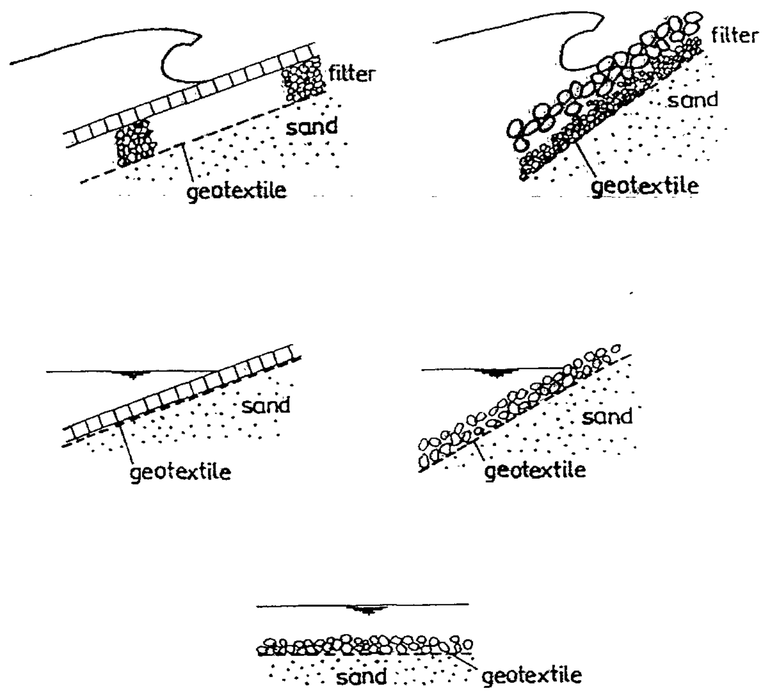


Figure 1.31. Flexible Revetment Concepts.

Revetment concepts are usually distinguished after their cover layers. The most common concepts are listed in Table 1.5.

Most of these concepts use rock as the basic material. Additional materials can be applied to provide for an increase in stability.

Dykes and their design are discussed at many places throughout this document, wherever pertaining to the subject.

Dykes are incorporated, together with jetties and other river and coastal engineering structures, in the project aimed at protecting the city of Valencia (Spain) from flooding by Rio Turia (Fig.1.32).

1.4.2 *Cross-Shore Structures*

Jetties

The principal functions of jetties are control and guidance of flow or fixation of gullies, often throughout the coastal (surf) zone. Therefore the structural differences versus breakwaters are minor. Hence, in principle, breakwater and seawall strategies can be applied for jetties.

Groynes

Groynes are used as structures oriented more or less perpendicularly to the coast-

Table 1.5.

Revetment concept	Alternative systems
Random placed natural stone	rock
	rubble
	rip-rap
	gravel
	loose, concrete blocks
Placed or cast units	units cast *in-situ*
	loosely placed blocks
	cable-connected concrete blocks
	interlocking concrete blocks
Container-type systems	gabions (wire, polymer)
	stone mattresses
	bags (jute, fabric)
	fabric mattresses
	tubes (fabric)
Bituminous system	grouted stone
	open stone asphalt
	sand asphalt
	asphalt concrete
	bituminous membranes
Vegetative cover layer	natural grass-type plants
	plant containers
Other systems	breakwater armour units
	tyre mattresses
	grid mats
	filament mats (polymer)
	pocket fabrics

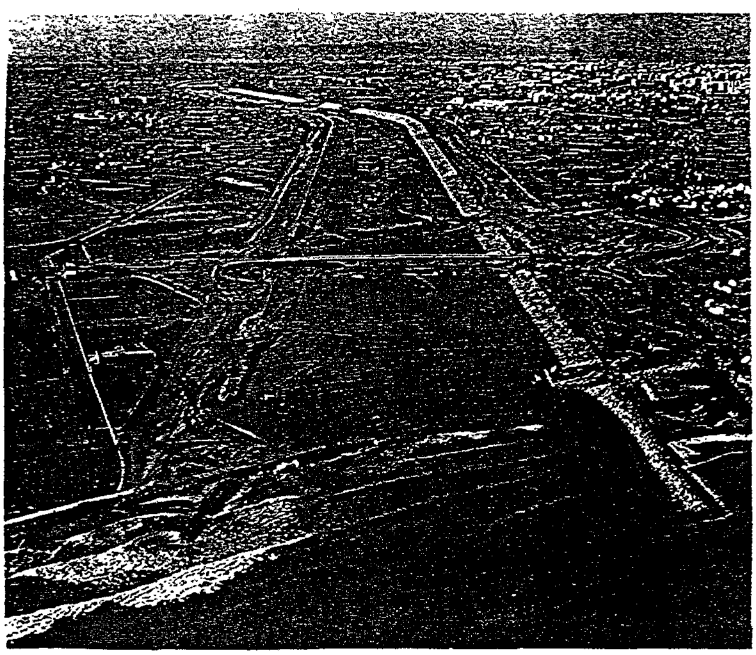


Fig.1.32. Example of Longshore and Cross-shore Structures Combining in a River & Coastal Engineering Project off Valencia; *SOGREAH*

line. Their function is coastal protection along the coasts subject to erosion in the **presence of longshore sediment transport**, which is manifested through preservation, or even generation, of a wide beach.

It is generally assumed that groynes bring about accretion of sediment in situations with rather small, obliquely incident waves, while high storm waves cause erosion despite the presence of groynes.

A single groyne has never proved successful, and therefore groynes are constructed in groups or systems. Downdrift erosion on the lee side of the groyne system is seldom avoidable.

1.4.3 *Other Systems*

Bottom or scour protection

The means intended for scour protection, usually costructed with primary coastal defence or management measures, often evolve into heavy structures themselves. They cannot be classified as strictly longshore or cross-shore.

Stabilisation of seabed may be required for several reasons. Navigation and protection of submarine cables or pipelines are some examples. Also local scour, induced by coastal and ocean structures, such as seawalls, breakwaters, groynes, etc. or by other human activities on the seabed, may require a bottom protection. Because of advantages mentioned and since often the natural geometry must be maintained, flexible concepts are employed in such cases. Scour protection is always needed if scour is expected to endanger the stability of the foundations of structures. The principal concepts for bottom protection are:

- rock-type (single- or multilayered);
- mats of artificial blocks;
- container-type mats;
- bituminous systems;
- sealing methods (these are beyond the scope of this report).

Concepts of seabed stabilization are discussed in detail in the subsequent chapters.

The variety of **combined systems** is only limited by designer's imagination. The common property of most systems is the mobilization of additional resistance forces between idividual elements. This resistance can be friction, interlocking or tensile forces.

Breakwater armour units are an example of interlocking, which is provided for by a variety of typical shapes. Because of the combination of size (mass) and shape, considerable moments are exerted on joints of these units. As a result, material strength parameters become a dominating factor for the response of these units when exposed to wave loading. Material strength and reinforcement thus become limiting factors for application. These units are described in the report of PIANC Working Group 12 (Breakwaters).

Tyre mattresses can be considered as an example of the application of waste materials. When connected and ballasted with concrete, scrap tyres provided a cheap alternative type of flexible revetment.

Many possible and attractive alternatives are discussed in Chapter 6.

Vegetation can be applied as reinforcement of a basically erosive material. This gives an increase in strength of the base material. Additionally, vegetation may reduce the loadings due to dissipation. Vegetation can be provided for by seeding or by placing of plant containers (bags, tubes, etc.). The container is filled with soil, enclosing the root system and initial stems of plants. Roots and stems can grow through the container. When conditions are favourable, both stems and roots will grow and develop the final strength of the system. The strength largely depends on the health of the plants.

Mats can be used as an intermediate layer to provide for additinal friction or soil reinforcement . Pocket fabrics, filled with soil and young vegetation, can also be used to allow for the development of a natural flexible revetment.

As already mentioned, these and other environment-friendly, low-cost, temporary, etc. coastal defence and management measures are presented in Chapter 6.

1.5 DESIGN APPROACH; DETERMINISTIC VS. PROBABILISTIC

1.5.1 *General*

A coastal structure is subject to the external effects of the environment, labelled shortly as **loading.** The response of the structure to this loading, in the form of linear and angular displacements, is usually characterised through **stability** or **deformation.** In terms of the theory of dynamic processes one has loading as an **input** and stability or deformation as an **output.** Since **strength** is also associated with states of stability and deformation, we are henceforth employing it as an output feature.

Basic functional relationships linking the stability, or strength, to loading, referred to as **transfer functions**, are used to describe the limit states in the domains of hydraulics and geotechnics. These relationships, which form the basis for the structural design are often known as the "stability criteria". Eventually acceptable (limit state) deformations and displacements should be included through a relationship between loading and deformation. After analysis of the physical process involved, loading and strength functions can be derived from the stability function.

Loading and strength are both functions of a large number of basic variables and parameters. The strength variables are determined by material properties and the geometry of the structure. The loading variables of marine structures mainly originate from the environmental boundary conditions (wind, waves, currents, ice). The general formulations for the strength and load functions, that are used in the remainder of this report are respectively:

strength function: $R = R(X_1, H_2......X_m)$;
loading function: $S = S(X_{m+1}...X_n)$;
where: $X_1, X_2....X_m$ are the strength variables and
$X_{m+1}......X_n$ are the loading variables.

The structure response is then defined as displacement, movement or deformation. This is illustrated in Fig.1.33 by the interactions (transfer functions) indicated with "I".

For the structural design of a coastal structure now the following elements should be considered in a design procedure:

- boundary conditions;
- external transfer functions (I);
- loadings (forces, pressures);
- internal transfer functions (II);
- structural system transfer functions (III);
- response of system (displacements).

When the loadings have been determined, the strength has to be derived from the functional relationships. Usually stability formuale are used to determine the limit state strength. Safety is provided for by chosing a design strength, which exceeds the limit state strength. Now the structure is stable, except for a certain probability.

Two principal options exist to limit this probability by adjusting a sufficient safety margin between strength and loads: **deterministic or probabilistic.**

In a probabilistic design, assumptions have to be made and (if possible) verified with regard to the statistical distribution functions of the design parameters (eg. wave height, stone density, filter permeability). The choice of either probabilistic or deterministic design approach has consequences for the design data that should be generated. The essential difference is that probabilistic methods need additional statistical information on the input data, for instance not only a mean wave period but also a standard deviation of wave periods.

Prior to embarking on details of the deterministic and probabilistic c design schemes one has to specify the notions of **failure, damage, fault tree** and **system approach** which will be employed henceforth.

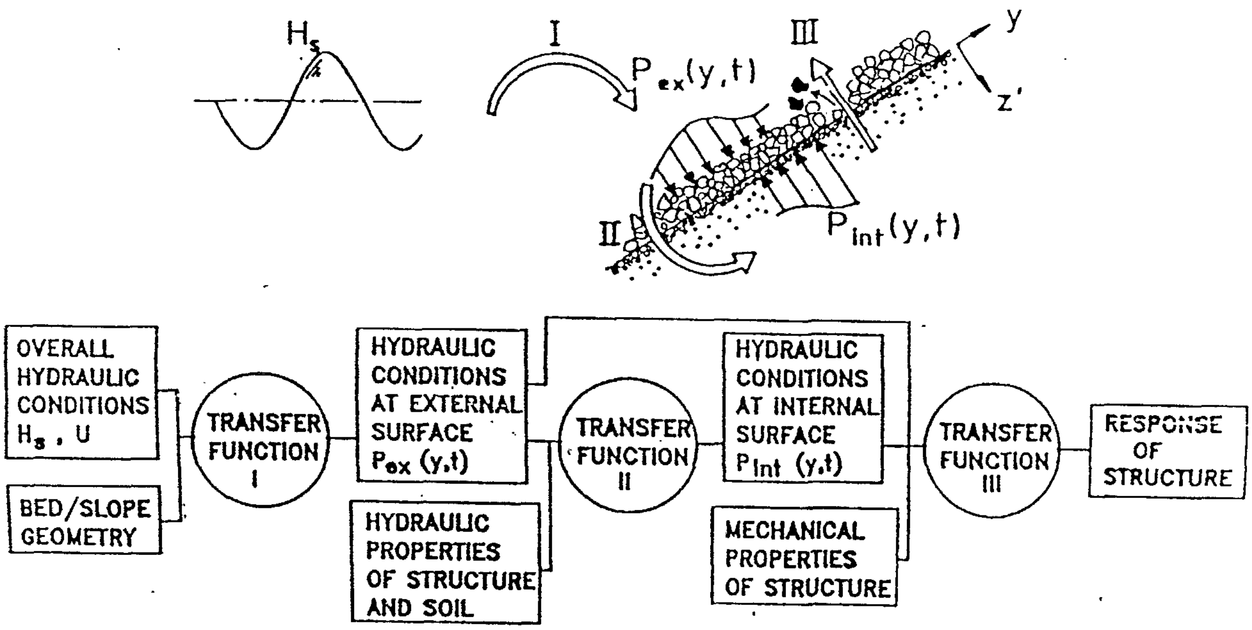


Figure 1.33. System Approach.

When, at any time within the lifetime, the principal functions can no longer be fulfilled, the structure has failed. This **failure** does not necessarily imply a total collapse or disappearance of the structure. A certain reduced level of functioning may be maintained after failure has occurred. A certain residual strength may also remain after failure.

In general, **damage** can be defined as a certain change in the state of the structure. The state of a structure is reflected by the following three characteristics of the structure; strains, movements or diplacements.

Changes of the types (1) and (2) often correspond to a certain physical loss or displacement of the material of a structure. This in turn may cause a loss of functions. Failure can thus be expressed as a certain threshold damage, and the corresponding specific loading level. When a (sudden) progressive increase of damage as a function of the loading level cannot be observed, the point of failure has, for practical reasons, to be defined at one particular degree of damage. Failure can occur to parts of a structure and to an entire structure (eg. partial failure of an armour layer and total failure of a breakwater due to liquefaction of the subsoil). Partial failure as such is in general regarded as less serious than total failure.

Some failure mechanisms can be accepted to occur repeatedly up to a certain limit (eg. the displacement of an armour stone). For other mechanisms not even a single occurrence can be accepted (eg. liquefaction of the subsoil under a breakwater).

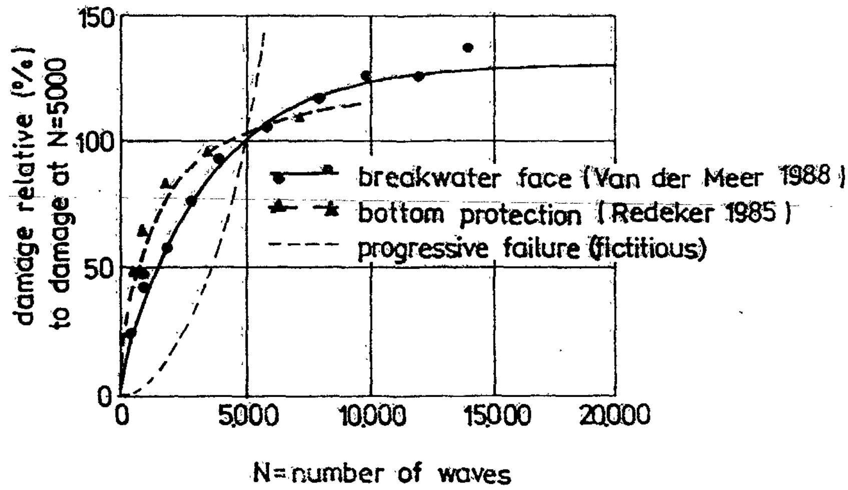


Figure 1.34. Evolution of Damage as a Function of Time.

Repeated occurrences of one mechanisms lead to increasing damage (loss of filter material in the first example given above).

The frequency of repetition determines the evolution of damage. In general, the damage will not only increase with the loading level (as shown in the given example), but also with time. An important question is whether (at a certain constant loading level) the rate of damage will decrease or increase with time. Increasing damage rate, as shown by the fictitious line (3) in Fig.1.34, is typical for coastal structures, which derive an important part of their strength from interlocking and/or friction and which are exposed to loadings beyond the design value. Examples are found amongst block revetments and breakwater armour units.

A **fault tree** is a schematic diagram, which gives an ovierview of possible failure mechanisms and their mutual relationships. A simplified example of a fault tree is given in Fig.1.35. (In fact this is part of the much more extended tree that can be made for breakwaters and seawalls). In a fault (or failure) tree of possible mechanisms a number of subsystems can usually be distinguished (eg. degradation of crest level due to wave attack or due to settlement, shown in Fig.1.35.

A major distinction must be made between parallel or series systems. In the report of PIANC Working Group 3 ("Risk considerations for Bank Protections") this has been treated in detail. **Event trees** can be used similarly to failure trees to schematize the functioning of systems rather than the failure. It should be noted that mechanisms appearing as parallel elements in a failure tree are series mechanisms in an event tree. For definition of parallel and series systems one is referred to the event tree of the system. In the example of failure tree in Fig.1.35, both a series (left) and a parallel (right) subsystem are shown. In a series system failure one needs failure of only one subsystem, where in parallel systems the failure of

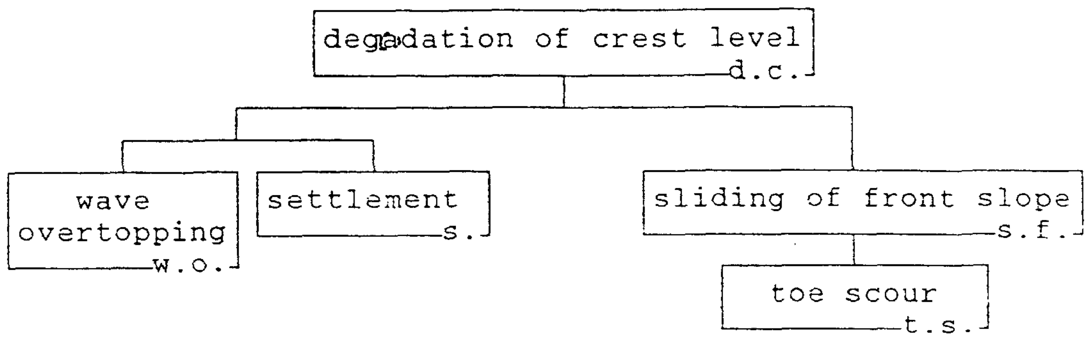


Figure 1.35. Fault Tree.

all parallel subsystems is needed to cause failure of the system. Usual rules of the theory of probability are applied accordingly to respective combinations of partial failure probabilities in series and parallel systems.

In the fault tree analysis, failure has thereby been considered as the result of the exeeedence of a limit state. In the following geometrical and structural design a structure and its constituents can be schematized to be mechanical systems. The input is the loading (either hydraulic or geotechnichnical) and the output is a response.

1.5.2 *Design Approach*

The motivation of a design process is the expected impovement, in terms of construction measures, of the risk-benefit balance compared to the existing circumstances. Implications for the design process are that a maximum level of functioning of the future structure is required against a minimum cost during the life cycle of the structure.

In general, two levels of design can be distinguished, viz. **geometrical** and **structural design**. As choices made in the geometrical design may have consequences for the structural design, an iterative process may result. The two levels are dealt with in our Chapter 4, separately under "morphological (overall) dimensioning" standing for the **geometrical design**, and "forces, stability and near-field effects" referring to the **structural design**.

In the stage of **geometrical design**, general layout and concept is developed, aiming at an optimum geometry. This optimisation is made from different points of view. The most important of these are the functional requirements, expected loadings, available materials, transportation, future use and management.

In the **structural design**, the dimensioning of various constituents of the structure takes place. In the most general sense the objective of the structural design is to provide for a detailed technical specification that will enable the construction with a certain strength/loading ratio. This ratio is the ultimate criterion that follows from the functional requirements of maintaining a prescribed state of the structure under the expected loading conditions. This will ultimately imply a strength that is not exceeded by the loadings. This implies that the strength variables ($X_1 ... X_m$, see above) assume values at which the strength R exceeds the design loading S to a prescribed degree.

Using a deterministic approach a design strength (R_d) and load (S_d) are assigned to the structure. Traditionally, the ratio R_d/S_d is called the safety factor. Using the distribution function of S one defines the probability of failure as the exeedance frequency of S_d.

Intermediate or semi-probabilistic methods assign partial safety factors to the most relevant mechanism, which are finally combined into an overall safety factor. These are the simplest probabilistic methods, in fact very close to deterministic methods. Probabilistic techniques give the designer more objective means to built into the design a certain degree of safety. The uncertainties in environmental and material parameters are thereby taken into account.

In a probabilistic approach the moderate loading conditions and the uncertainties of S and R are also included. At present deterministic and probabilistic methods are the two basic options to secure (or built into the design) a certain safety for a structure.

Possibilities to Control the Hydraulic Loading

The primary function of coastal defence measures is to protect the edge of land and water from hydraulic loading by waves, tides and currents. The determination of the hydraulic design conditions is the result of a quantification of the local conditions in combination with a certain level of safety. In this way the design conditions are defined and presented in the form of water level, wave height and wave period, usually combined with a certain accepted form of the energy density spectrum and sometimes an estimate for the duration of the selected design condition.

However, the fact that the design conditions are fixed, does not necessarily mean that the loads on the protected structure are also fixed. Within certain limits, it is however possible for the designer to influence and consequently to choose the size, the sort and the place of attack of the hydraulic loads, by a proper selection of construction materials, geometry and configuration.

A good designer will make an optimum use of these possibilities. In the following some of the different ways to control the loads are briefly illustrated for the case of flexible revetments (Leeuwestein, 1989):

- By changing the slope steepness the breaker type can be influenced (see Fig.1.16) The value of the breaker parameter

$$\xi = \tan\alpha/(\frac{H}{L_0})^{1/2}$$

is not only decisive for the type of breaker but also for the levels of runup and rundown. For a given value of the wave steepness H/L_0 the value of ξ increases with increasing slope steepness. The type of breaker in itself determines the way a breaking wave exerts loads on a slope and thus on a slope revetment. This can be with a huge wave impact or, on the opposite, with large masses of water running up and down the slope. Because the levels of wave runup and rundown are also influenced by the value of ξ, the slope steepness determines the required crest elevation and the level where the maximum wave impact takes place and the level where other damage mechanisms endager the structure's stability. It is therefore essential to realize that it is possible to choose the critical damage mechanism by manipulating the slope steepness.

Especially for rubble slopes, or other types of random placed slope protections, the influence of the combination of slope steepness and wave steepness is difficult to establish, because for these types of structures it is the combination of parallel and perpendicular flow on and in the toplayer that determine the toplayer stability

- By variation of the toplayer permeability it is possible to focus the hydraulic loads on certain parts of the revetment structure and to relieve other parts. So the design can be optimized for the locally available construction materials and techniques. For example, a less permeable toplayer leads to fairly limited pressure variations in the sublayers, also during large pressure variations on the outside of the revetment. Consequently, the internal stability can be quaranteed easily (but the stability of the toplayer is severely endangered in this situation by the resulting upward loads on the toplayer from the filterlayers at the moment of maximum wave rundown).

  With a very permeable (=open) toplayer, on the contrary, the hydraulic gradients over the toplayer will always be small, also under severe wave attack, but in this situation the loads on the sublayers are large because almost no damping occurs through the toplayer. In this situation a real hazard also exists of the erosion of filter material through the toplayer. Accordingly, the designer should give special attention to the design of filter layers and internal intefaces. In Chapter 4 the formulae for the calculation of the underlaying physical processes are presented. It appears that the ratio of the toplayer and sublayer permeability k/k', determines the value of the leakage factor Λ, which determines the pressure difference over the toplayer ϕ_w and the internal hydraulic gradients i

- The application of wide graded or fine filter material reduces the filter permeability and consequently increases the toplayer stability. However, for wide graded filters there is a danger for internal instability (suffosion) when the fine fraction is eroded from in between the coarse fraction, thus increasing the permeability of the filter material. In this situation there is also a real chance for undermining of the toplayer by erosion of (the finer) part of the filter material.

- Application of a very thin granular filter layer underneath the toplayer reduces the upward hydraulic gradients over the toplayer and consequently leads to a reduction of the required weight of the toplayer. However, the loads on the base material (sand or clay) will increase. This may lead to instability at the interface between base and filter, where the fine base material may be eroded.

The examples above, that all refer to placed block revetments, are merely presented to illustrate the fact that no strict procedure can be given for the determination of the external or internal structure's geometry covering all practical situations. During the process of designing a slope revetment, numerous, more or less subjective, choices must be made, influenced by considerations about price of construction materials, locally applicable construction techniques, technical restrictions, functional requirements and personal preferences.

Possibilities to Control the Strength

It seems somewhat illogical, but the possibilities for a designer to manipulate the strength of a structure are by far more limited than those for manipulating the loads. On the other hand it should be mentioned that often the change of a construction detail influences both, strength and loads. With respect to the sublayers the choice of the granular materials should be such that the materials itself and the internal interfaces are sufficiently stable under design conditions (see Chapter 4 for the design of granular filter layers) and will remain stable ("ageing" of the material) during the design lifetime of the structure. A geotextile sublayer should be selected for the purpose it must serve and should be (and stay) sufficiently strong for the tearing and punching forces during construction and operational use.

For the toplayer in fact only two possibilities exist to increase the strength:

- By increasing the block weight.
- By increasing the capabilities of cooperation between individual blocks in such a way that perpendicular forces and moments can be taken.

By manipulating the strength in one of these ways the designer should be aware that improvement of only the strength of a toplayer without taking the rest of the structure into consideration will usually solve only a half of the problem; another part will now be the weakest.

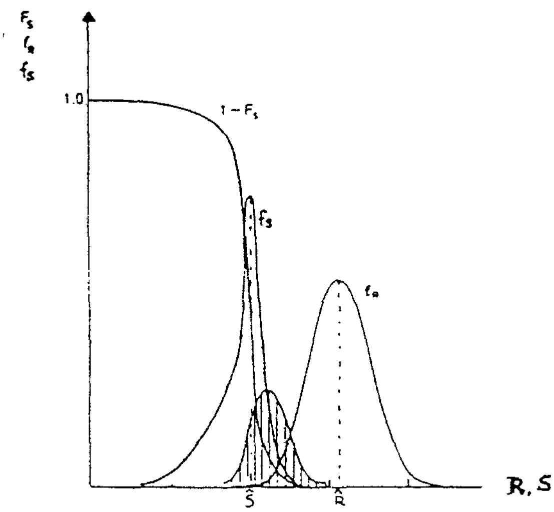


Figure 1.36. Distribution Functions and Probability of Failure.

Deterministic approach

Both loadings (S) and strength (R) are stochastic quantities, characterized by their respective statistical distributions. In general, R and S are functions of one or more basic parameters regarding to environmental or material characteristics (eg. wave height, stone diameter, material density etc.). Often these parameters are also stochastic. In the design, a deterministic approach is traditionally chosen, to provide a certain safety margin between the loads and strength. This implies that a limit state condition (ULS or SLS) is chosen with respect to the accepted loading of the structure. This limit state usually corresponds to a certain characteristic value of the strength. Exceedance of this limit state condition (which is usually called failure) is not accepted, except for a certain probability (P_F). This probability is usually based upon considerations of safety and economy. From a statistical analysis, the design loading (eg. wave height), which corresponds to the accepted probability P_F must be found.

In practice, the (small) value of P_F is usually expressed as an (average) probability per year that the structure fails. Often the somewhat confusing, reciprocal value of the latter is used: the (average) return period ($T_R = 1/P_F$). This is interpreted as the average time between two occurrences of the design loading (wave height). Because of this assumption (no damage for $S < S_D$) the deterministic approach has an important limitation: no account is taken of the fact that loading below the design value contributes to the development of damage. This is a serious shortcoming when future damage must be estimated and quantified for maintenance assessment.

The probability density functions of S and R usually have a shape comparable to the examples shown in Fig.1.36, which can be characterized by a maximum, corresponding to the "average" values S and for R and a declination with increasing horizontal distance from the "average" values. By chosing a characteristic strenght value (R_C) and a design load (S_D) so that R_C exceeds S_D sufficiently, the resulting failure probability, $P(S > R)$, is kept low. Therefore a safety factor $T(T > 1)$ can be defined as:

$$T = R_c / S_D \tag{1. 25}$$

Through the choice of T thus a certain safety margin is maintained. In fact the stochastic character of the strength and loading variables is only accounted for very roughly. The choice of T is largely based upon experience gained from existing structures. In particular when several partial safety factors are applied, a conservative design might be obtained. General standards for the choice of T are not available at present and are often adjusted in a rather subjective way. However, values recommended in standards apply to the use of specific materials in industry and civil engineering. In a quasi-probabilistic approach T is composed from a number of partial safety factors T_i (eg. for wave height, material strength, friction factor, etc). Probabilistic methods are also used to find a rational basis for the partial safety factors. The report of PIANC WG 12 provides more extensive information.

Statistical data with respect to R and S can be used fo find their respective probability density functions (f_R, f_s) and distribution functions (F_R, F_S) . Subsequently, P_F can be found as the probability $P\{R < S\}$. Probability distribution and - density functions of any parameter x are related through:

$$F_x = \int_{-\infty}^{x} f_x dx' \tag{1. 26}$$

The quantity $P\{R < S\}$, is found by (double) integration of the joint distribution. Using F_s, however, $P\{R < S\}$ can be shown to follow from single integration of the product of $1 - F_s$ and f_R:

$$P\{R < S\} = \int_{0}^{\infty} \{1 - F_s(x')\} \cdot f_R(x') dx' \tag{1. 27}$$

This probability is illustrated by the shaded area in Fig.1.36, where the density functions of S and R are also shown. When stability of rock armour is considered, for instance, S and R can read as wave height and stone weight, respectively.

Probabilistic approach

Better implementation of stochastic characteristics of design variables has been postulated above. Analysis of the behaviour of structures under influence of stochastic loading, both in prototype and laboratory, has shown that the simplified failure concept, disregarding any effect on the structure of loadings below the design level, can be improved. This improvement has led to the use of probabilistic methods. The efforts of a probabilistic approach will be payed back in saving cost.

Additionally, an improved appreciation of the stochasic elements in both loadings and strength also offers means to assess the expected damage during the lifetime of a structure.

In summary, the major advantages of the probabilistic approach compared with the traditional deterministic approach are:

- better appreciation of strength and loading statistics;
- prevention of unnecessary conservatism, leading to cost saving;
- provision of means for maintenance assessment and management data.

To a certain degree, the probabilistic methods are logical extension of the traditional method. Hereby a safety margin is obtained implicitly by taking into account all the uncertainties of the loading and strength variables. By accepting a certain probability of failure the designer adjust the safety margin in a rational process.

By international conventions, probabilistic methods can be distinguished on 3 levels. These are listed here in the order of decreasing complexity:

Level III : these methods take account of the real probability distributions of the strength and loading variables

Level II : these methods use schematised distribution functions for the strength and loading variables;

Level I : these are in fact quasi-probabilistic methods that use partial safety factors for the various variables.

The deterministic method that uses one safety factor T between characteristic strength and loading is in this order often indicated as a level-zero probabilistic method.

Cost analysis of defence measures embodies the following elements

- primary construction cost, including materials;
- cost of financing (interest);
- cost of management (maintenance, repair).

Loading and strength form the basic factors that determine the behaviour of the structure during its life cycle. As a result, the total capitalised cost, including management (maintenance and repair) can, to a certain degree, be controled during the design through the strength/loading ratio. Also, the degree to which the functional requirements are met and the total capitalized risk level are largely determined by the strength/loading ratio. In general, account should be taken of the possibility that the strength of a structure changes (decreases) with time (wear, abrasion).

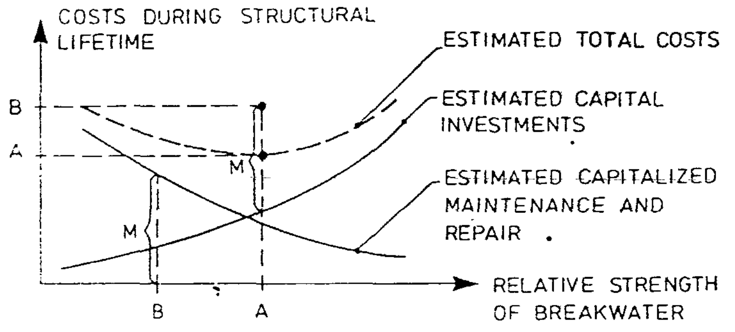


Figure 1.37. Cost Optimization.

Probabilities of failure provide an important link to the economy of design. In the design stage a fundamental choice can be made with respect to the choice of capital investment and reservations for maintenance and repair. Other considerations, for example those related to construction, might be included here as well. The strength/loading ratio adjusted during the design stage is the controling factor for both investment costs (C_I), risk (or cost of ensurance, C_R), and the cost of future management (C_M). C_I is a function of the design load, or the accepted probability of failure (p_f). C_M includes maintenance, repair and rehabilitation. The formulae for these costs, in terms of present or constant capitalised cost over N years, can be written as follows:

$$C_I = C_I(S, R)$$

In this equation R represents the real interest rate and C_m are the expected yearly costs of management (expenditures on repair and maintenance). These costs are determined by the expected damage (eg. volume of displaced stones, for given R and S) and by the cost of materials (eg. stones), labour and equipment. The total costs over the lifetime of N years, which is a function of R and S, can then be written as:

$$C = C_I + C_R + C_M \tag{1. 28}$$

In Fig.1.37 an example is given of an optimisation curve for the design of a breakwater with four optional values for the outer slope.

As already mentioned, in this document we will **not** discuss cost aspects, so that hereafter we confine ourselves to an introductory discussion presented by Rietveld and Burcharth (1987).

The final design should represent a cost minimum in Fig.1.37. Capital investments can be calculated rather accurately whereas maintenance and repair costs are much more difficult to estimate.

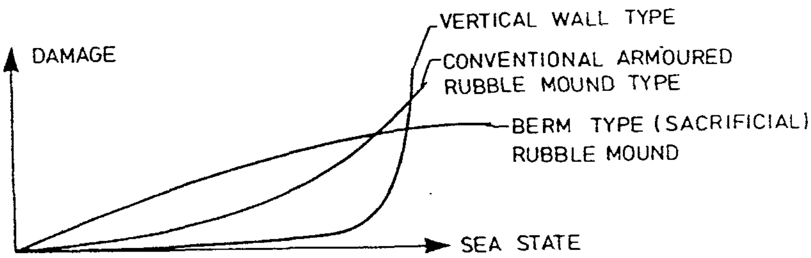


Figure 1.38. Illustration of Coparative Damage Sensitivity of Various Types of Structures, Rietveld & Burcharth (1987).

If we design for condition A we might find ourselves in B if the maintenance and repair costs are underestimated. A flat curve for maintenance and repair costs will restrict the economic risk.

For a quantification of the principles outlined above a stochastic model should preferably be used (e.g. Nielsen and Burchart, 1983).

Due to limitations of knowledge regarding wave climate and structural response there is always a chance of damage or failure. The client responsible for the maintenance of the structure should have a "user's guide" which will inform him about the vital parts of the structure and the maximum allowable damage done to these parts. He should also be told at what external conditions certain effects might be expected. Regular surveying and special surveys after extreme conditions should be prescribed ("survey to survive").

The basis for such a user's guide is the knowledge about the residual strength (resistance) of the breakwater under various levels of damage, such as displacements and breakage of armour units, sea bed scour, deterioration of rock and concrete materials, displacements of superstructures etc.

This knowledge can be provided by much more extensive model test programs than generally used today because such programs involve many stages of partial failures and also involve material aspects. In principle respective diagrams should be produced. The obvious possibility of co-existence of several damage modes makes it necessary also to produce at least some characteristic diagrams and tables for the evaluation of joint partial damages.

The damage evaluations of course depend very much on the type of a coastal structure. At least three types of structures can be identified due to significant differences in sensitivity to exceedence loads, Fig.1.38.

CHAPTER 2

Far-field (morphological) effects

2.1 GENERAL

This report has been intended as a general study on the effectiveness of coastal defence and management measures. This chapter deals with macroscopic, **far-field** effects, while Chapter 3 contains a review of **near-field** aspects, such as scour, stability and durability of coastal structures, along with possible failures. Both chapters are aimed at providing a background on coastal processes intervening in various protected zones so as to yield guidelines on selection and design of coastal schemes and enable the planner and designer to select their primary options for coastal management and defence. The material is based on a fairly broad spectrum of bibliographical entries discussing cross-shore and shore-parallel structures; beach nourishment; headland control; and low-cost, environment-friendly, emergency and temporary measures.

The findings on the **far-field** behaviour of different structures and measures in the coastal environment are presented in this chapter in the above order, together with description of the operation of various coastal schemes combining features of different protection measures. Together with conclusions on durability and stability of structures and other near-field effects in Chapter 3, these findings are harnessed in this report to serve the aforementioned purpose of guiding planners, designers, and engineers in their coastal projects.

2.2 CROSS-SHORE STRUCTURES (Groynes)

A variety of cross-shore structures, arranged normally to shoreline, are encountered in coastal engineering. They include jetties,breakwaters, causeways, piers, groynes etc. In view of the objective of this study, only the structures deemed for most immediate coastal protection, i.e. groynes, will be discussed, as typical and most common representatives of cross-shore structures.

Since quite recently it has been widely accepted that after a sufficiently long time the shore and beach topographies around single groynes and system groynes, positioned

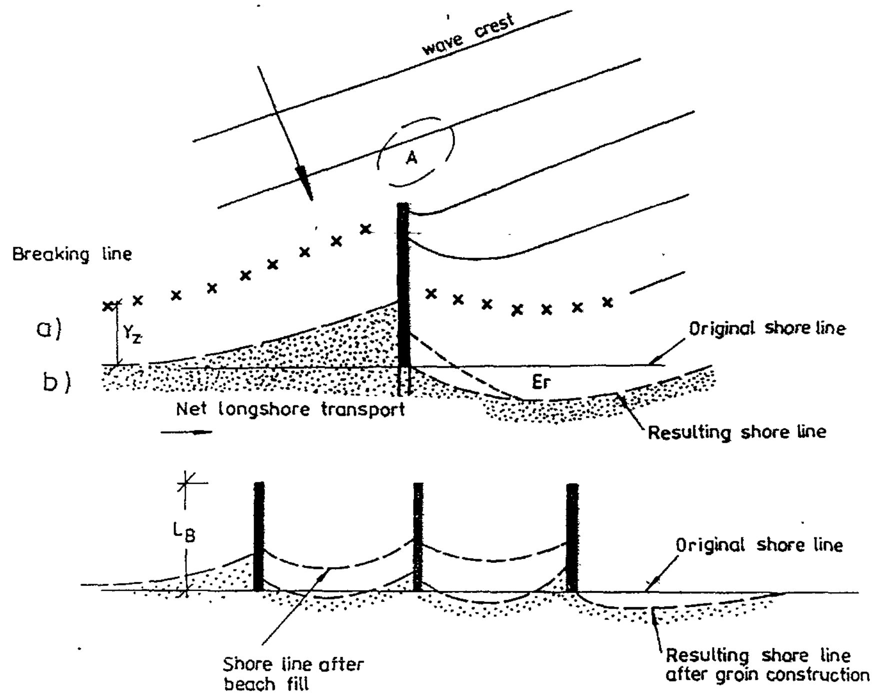


Figure 2.1. Effect of Cross-shore Structures: Single Groyne (a) and System of Groynes (b), Sorensen (1978).

normally to the mean shoreline, look like those depicted in Fig.2.1 (Sorensen, 1978). However, the recent studies on beaches protected with groynes, cf. Tomlinson (1980), Summers (1983) and United Nations Report (1982) have brought to our attention that the use of groynes has been successful only in some cases while in many other it caused either minor changes, with an insignificant slowdown of erosion, or even detrimental effects. These observations indicate that groynes may react with beaches in different ways and also that our knowledge of the relevant coastal phenomena is limited.

Groynes are usually constructed at normal or small angles to mean shoreline. Some examples on Polish coastline are illustrated in Fig.2.2. Their primary function is to protect natural and man-made beaches and stabilize shores that are subjected to severe storms or seasonal transformations. They can also be used to create new and enlarge the existing beaches, recreational or defence-oriented. The Soviet coastal engineering practice makes distinction between *sand-trapping* and *beach-preserving* groynes (Khomitskiy, 1983).

Beach supply due to groynes originates from entrapment of the longshore transport and limitation of the sediment transport out of the protected area. This process is enhanced by division of the protected area into smaller segments (often nourished artificially) and by local altering the direction of the shoreline into normal to the prevailing waves. Groynes can also serve as preventive means against sand accre-

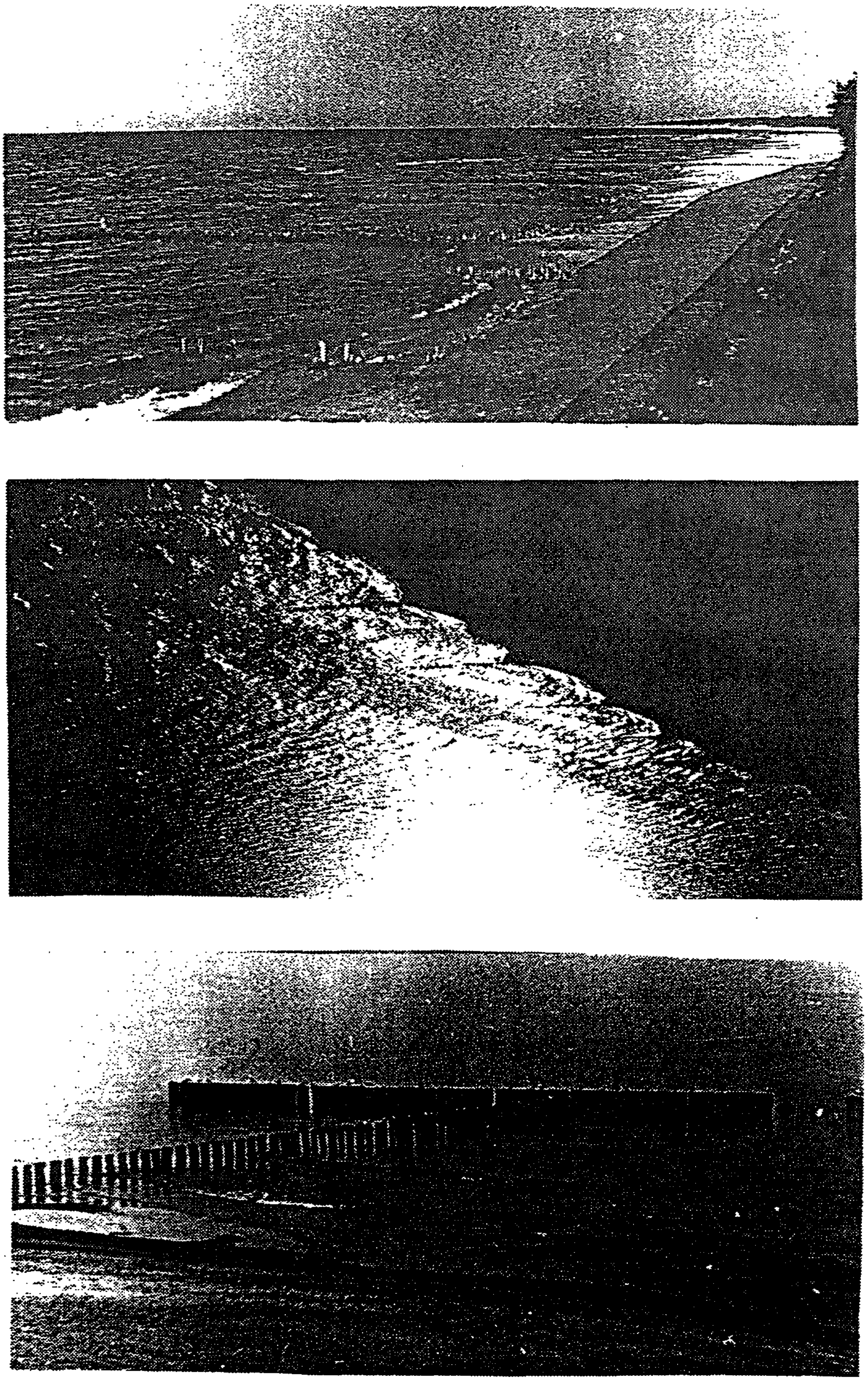


Figure 2.2. Three Polish Coastline Areas with Groyne Systems.

tion on their lee side, if the latter proves an undesirable effect in some locations.

A variety of groyne systems is noted. Groynes can be implemented as permeable or impervious, short or long, high or low, with constant or variable height (Fig.2.3). Single groynes or groups of groynes can be made from different materials, such as timber, stone, steel and concrete.

Groynes are widely used in the coastal environments with **prevailing longshore sediment transport**. They redistribute wave and current patterns, but their basic function is to slow down the rate of littoral drift. Groynes are of **no value without an adequate supply of beach material** and create material deficiency on their downdrift side. They are also used extensively to control the distribution of material along a frontage and to limit the temporary effects of drift reversal.

Suitability of groynes has been questioned on a number of occasions, and there remains a significant diversity of opinion on the subject. There are, however, very many examples where the success of groyne systems is not in doubt, and it is the authors' opinion that - if properly used - groyne systems remain a valuable tool for beach management.

Hence groynes are one of the most controversial coastal defence measures. In many instances they are successful but equally well they might be situations with detrimental environmental effects. These effects are due to local conditions of the coastal environment, design characteristics of sediment transport, and general variability of many factors.

The performance of groynes on a cobble or shingle beach is reasonably well established, but as the grading of the beach material becomes finer, adverse effects of wave-generated currents in causing scour within the groyne bays increase. The sutability of groynes on a flat sandy beach with a large tidal range is certainly open to question.

The operation of groynes under *normal wave attack* has not been explored sufficiently. The measured data indicate that circulation cells are generated in bays between groynes, whereas water flow along the structures gives rise to intensive scouring in the very proximity of groynes. The active length of a groyne increases with growing angle of wave incidence. The gaps between groynes in a coastal defence scheme should be closely related to this length. The efficiency of groynes depends on their length, i.e. their penetration into the surf zone. During severe storms, groynes can operate as relatively short structures for which erosion at groyne heads and scouring on the updrift side is commonly observed, while during periods of relatively calm weather the structures become long, and stimulate accretion and widening of beach.

The bed transformation at the structure depends on sand properties; this problem has not been explored to a sufficient degree. Some authors indicate that in the case of *fine sediments*, which are moved mostly as suspended load, erosion occurs on the

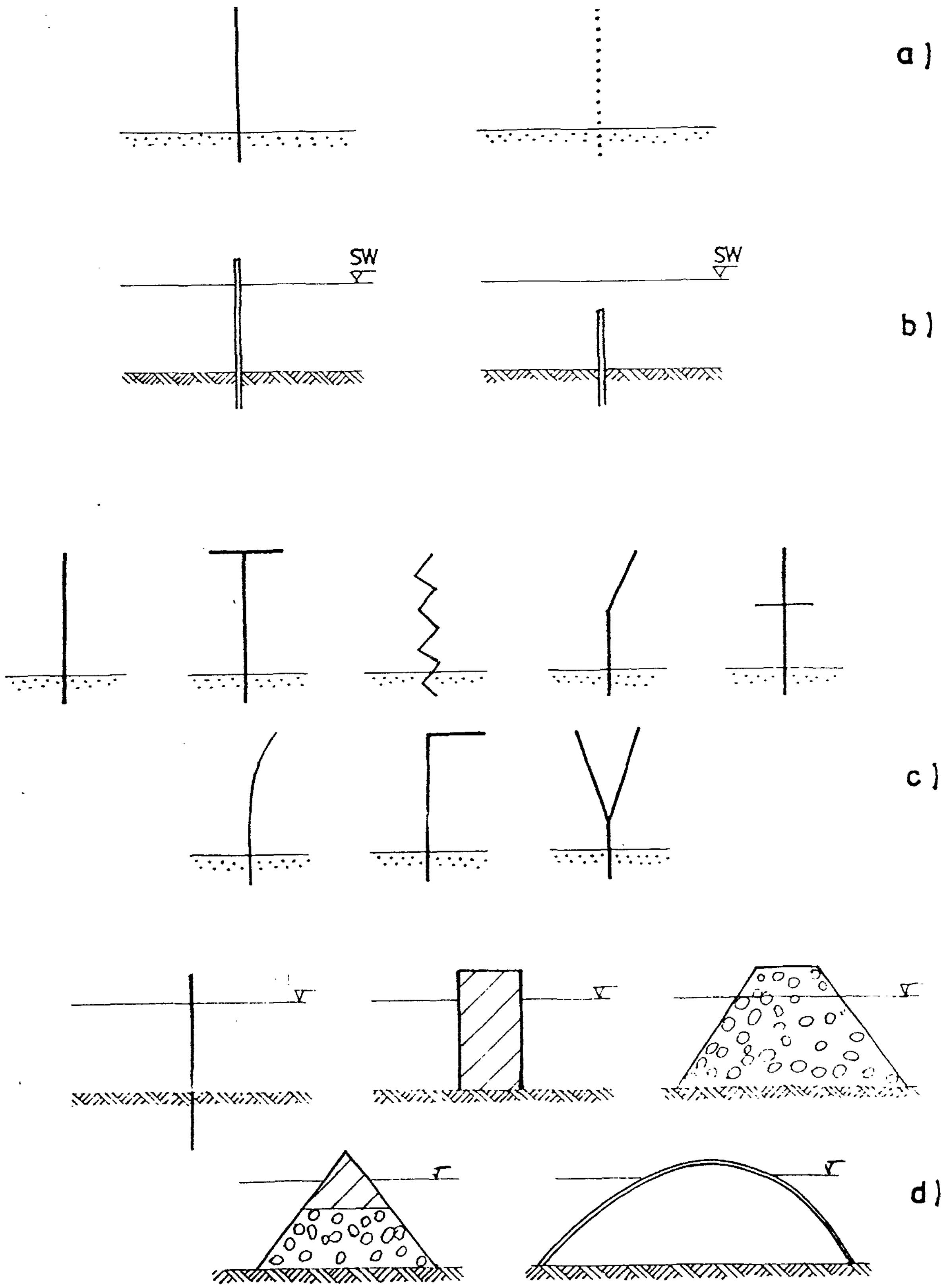


Figure 2.3. Types and Shapes of Groynes.

updrift side of a structure together with accretion on the downdrift side. Opposite effects are encountered for coarse sediment transported mostly as bedload.

Efficiency of groynes depends on the degree of their permeability. Permeable groyne or submerged groynes will entrap less sediments then impermeable high groynes. Moreover, during periods of storm surges, berms and beach foot will be subjected to more severe erosion due to wave reflection from an inclined beach. The effect of groyne permeability on the processes of sediment transport and evolution remains undetermined. Some investigators ascertain that for the clearance of 37% the longshore sediment transport is reduced by about 50% (Kolp and Otto, 1971), while some others prove that groyne permeability has no practical impact on the sediment transport rate.

It should be remembered that a local coastal defence scheme can have negative impact on the neighbouring coastal segments because of falling sediment supply and the growth of erosion pockets.

Groynes are usually applied if the long-shore sediment transport predominates. One must be aware of the erosion on the down-drift side of a group of groynes. Such erosion may require additional artificial beach nourishment at places particularly vulnerable to damage. Dramatic examples are provided by the GDR coastline where lee erosion was remedied temporarily by construction of additional groynes, those being later supplemented by next groups, etc. The final effect consists in "armouring" of the coast without visible improvement in view of permanent deficit of sediment (Khomitskiy, 1983).

Accretion can be encountered on the up-drift side of a groyne system. The temporal changes include variation of geometric characteristics of groynes, their relative length in the surf zone etc.

Field observations of Dutch groynes provide the following summary given by Rakhorst (1984)

- Erosion prevails over accretion on average coastline protected with groynes
- Erosion of shoreline after construction of groynes slows down from maximum 6...16 m/year (prior to construction) to 0...6 m/year (protected coast). It is noted that the erosion process is not arrested entirely
- Erosion intensifies at extremites of groyne systems
- The use of groyne system leaves uncertainties as to whether and how the groynes affect positively the coastline to be protected, as the protection effects depend heavily on proper dimensioning of groyne system.

Bakker et al (1984) stipulate that in the case of a groyne system in tidal seas the first check point should be an evaluation of the risk of washing out of piles by shoreward motion of tidal channels. One might face situations that even the maximum practical pile length will not be enough to guarantee constructional stability. Hence it is so important to consider near-field effects, on which we embark in Chapter 3.

From an extensive and comprehensive review of the interaction of groynes and the coastal zone it can be seen easily that the opinions on the performance and effective configurations of groynes and groyne systems are divided. This can be exemplified by the tabular excerpts from different sources presented in *Appendix 2-1*. Nevertheless, despite all controversies we have to endeavour some conclusions. The recommended parameters of geometrical design of groynes are given in Section 4.1 while the following items shed light on the general performance.

Hence the following **concluding remarks** can be ventured for groynes

- for coastal environments with prevailing long-shore sediment transport, the protective structures used most often are cross-shore structures called groynes

- the operation of groynes under normal wave attack is known insufficiently. The measured data indicates that circulation cells are generated in bays between groynes, whereas water flow along the structures gives rise to intensive scouring in

- the active length of a groyne increases with growing angle of wave incidence. The spacing of groynes in a coastal defence scheme should be closely related to this length

- the efficiency of groynes depends on their length, i.e. their penetration into the surf zone. During severe storms groynes can operate as relatively short structures for which erosion at groyne heads and scouring on the updrift side is commonly observed, while during periods of relatively calm weather the structures become long and enforce accretion and

- the bed transformation at the structure depends on sand properties. This problem has not been explored to a sufficient degree. Some authors indicate that in the case of fine sediments which are moved mostly as suspended load erosion occurs on the updrift side of a structure together with accretion on the downdrift side. Opposite effects are encountered for coarse sediment transported mostly as bedload.

- efficiency of groynes depends on the degree of their permeability. Permeable groynes or submerged groynes will entrap less sediments than impermeable high groynes. Moreover, during periods of storm surges berms and beach foot will be subjected to more severe erosion due to wave reflection from an inclined beach

- the effect of groyne permeability on the processes of sediment transport and evolution is not defined in a final and unique way. Some investigators

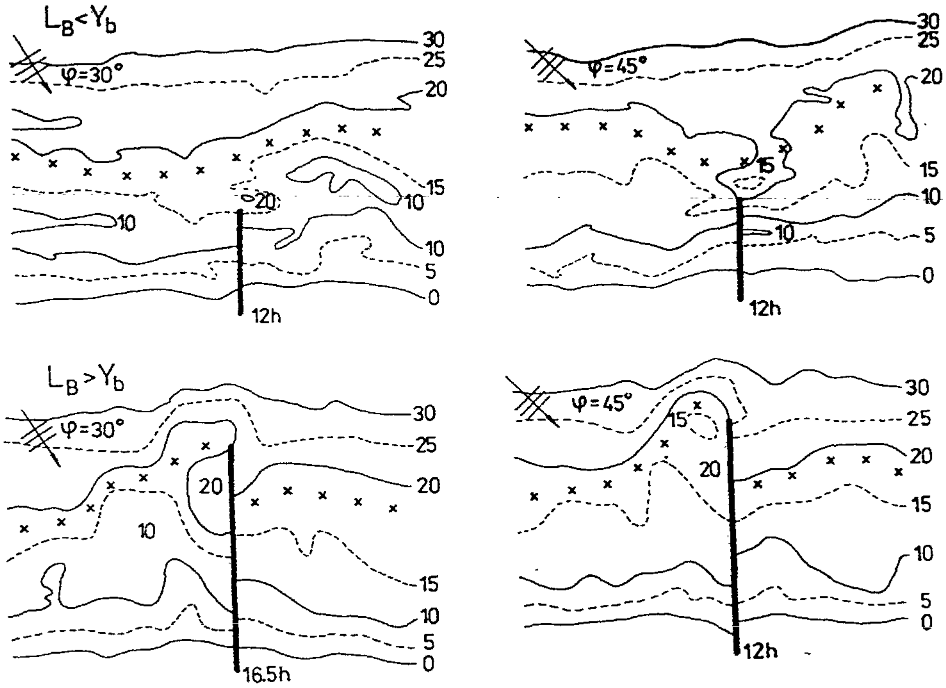


Figure 2.4. Bed Changes at a Single Groyne of Varying Length, due to Oblique Waves, Tarnowska (1985).

ascertain that for the clearance of 37% the longshore sediment transport is reduced by about 50% (Kolp and Otto 1971), while some others prove that groyne permeability has no practical impact on the sediment transport rate

- it should be remembered that a local coastal defence scheme can have negative impact on the neighbouring coastal segments because of falling sediment supply and the growth of erosion pockets.

Adequate parameters of geometrical dimensioning, or design with regard to the far-field (morphological) effects, are **recommended in Section 4.1.**

Mathematical modelling of cross-shore structures is discussed in Section 2.7. As an introduction, it can be said that in modelling coastal processes at groynes one may successfully apply the one-line theory, which despite its simplicity reproduces quite well the prototype conditions. The degree of complexity inherent in n-line models is overly high for sophisticated boundary conditions. Depending on specific coastal conditions and practical necessities one may recommend the refined model derived by Le Mehaute and Soldate (1980), together with a complex system of computational procedures GENESIS presented by Hanson (1987).

Movable-bed wave tanks can prove helpful in elucidating the shore transformation phenomena. Fig.2.4 shows examples of bed transformation measured in IBW PAN laboratory.

Figure 2.5. Jetties and Regenerated Beach at Los Urutias - Murcia (Spain).

Other cross-shore structures, such as the jetties shown in Fig.2.5, can also be successful in maintaining sand on the updrift side, and provide very spectacular results if combined with artificial beach nourishment.

An overview of the existing groyne practices is given in Appendix 2.1.

2.3 OFFSHORE BREAKWATERS

2.3.1 *General*

Offshore breakwaters, sometimes referred to as *detached breakwaters*, are shore-parallel coastal structures sited at a certain distance away from shoreline. They are preferred to other options if the cross-shore sediment transport mode prevails. Their primary destination is to dissipate wave energy and redistribute wave and current patterns so as to protect or even nourish the protected beach. Some other uses (sheltering harbours and preventing siltation in port access ways etc) are not dealt with in this report.

Distinction must be made between low and high, i.e. **submerged** and **emerging** breakwaters. Sometimes it is assumed that the former have a height h_B of less than 40% of the water depth h or the relative submersion $(h - h_B)/h_B > 0.5$. The former are often referred to as **reef breakwaters**, sometimes **sills**, while the other are common barriers protruding above still water level.

The functioning of offshore breakwaters depends on their geometrical proportions and local environmental conditions.

Low breakwaters (**submerged** or reef-type) have a very little effect on wave propagation, but can generally cause transformations of waves and currents. They produce strong nonlinear interactions between waves and mass movements. For instance, at the base of the coast-side sloping face of these obstacles, when the slope is about 1:1, an almost exclusively shoreward velocity has been observed, Cortemiglia et al.(1986). This seems to be the chief cause of erosion arising on the coast side of breakwaters, removing as it does the sediments from the obstacle. At the same time, there is an area near the bottom where sediments move only towards the coast, while they return towards the sea only if carried in suspension. For rather heavy particles the effect of this is a sand-trap, since the material passes in one direction but cannot return. Less heavy particles only show a tendency to move towards the coast, as their concentration is greater near the bottom.

High obstacles (**emerging** breakwaters) also produce a significant attenuation of the wave that generally breaks on them. The effect on the beach derives partly from attenuation of the wave, but partly certainly from the presence of this impassable barrier for particles moving on the bottom.

Offshore breakwaters are often constructed as rubble-mound breakwaters of trapezoidal cross-section, having different slope angles on the seaward and shoreward sides.

The slope angle of a breakwater depends on the forces exerted by the waves. Under breaking waves on the seaward side a mound is subjected to impact forces. Steep walls bring about rise in reflection-caused velocities and eddies at the foot of the structure, this in turn causing local scour and settlement of the structure.

Flat slopes give rise to energy dissipation on a longer section of structure.

2.3.2 *Morphological Interaction of Coast and Breakwaters*

Emerging breakwaters

In coastal engineering those structures are in most common use to:

- shelter harbours against waves
- prevent port access ways from silting
- protect beach against erosion
- nourish beaches.

The two latter aspects i.e. coastal defence with detached breakwaters will be discussed below.

The bulk of data on the performance of offshore breakwaters is still inconclusive. The site-specific nature of every project and the large number of parameters involved, as well as the fact that this is still a developing design area, makes standardised solutions inappropriate.

Detached breakwaters dissipate the energy of incoming waves by damping, reflect waves on their seaward slopes and create shadow areas in which wave diffraction is most pronounced. Due to the diffraction and associated with them local changes in wave - current fields arround the structure, the littoral sediment transport is modified, which results in sand entrapment and accretion on the shadow area. Characteristic spits or salients, are generated (Fig.2.6). Under favourable conditions these forms can join the structure to create a tombolo (Fig.2.7). On the seaward side of the structure the wave reflection, together with eddies about the structure extremities can bring about scouring and the degradation of beach profile.

An empirical attempt to determine the relationships between beach response and geometric parameters of offshore breakwaters has been undertaken by Pope and Dean (1986). The following five beach forms were distinguished:

- permanent or temporary tombolo
- pronounced salient

- weak salient
- no sinusoity.

The analysis was based on many field observations and measurements on eight protected coastlines in the USA. The geometry of offshore breakwaters has been characterized by dimensionless parameters $\frac{L_B}{G_B}$ (the ratio of breakwater length to gap width, which characterizes the capacity of the system to absorb wave energy), and $\frac{Y_B}{h_B}$ (ratio of distance from shoreline water depth, which reflects the response of the system, wave diffraction and shoaling), cf. Fig.2.8. It was assumed that well pronounced beach forms are generated if a small amount of energy is transported towards shore line, while the absence of sinusoities is characteristic for considerable wave energy reaching shore line. The demarkation lines shown in Fig.2.9 as functions of $\frac{L_B}{G_B}$ and $\frac{Y_B}{h_B}$ can provide guidelines for designers in siting of structure. One must stress that the protection systems monitored were located in regions of moderate waves.

Although offshore breakwaters have many merits in the area of morphological effects, such as favourable controllable accretional patterns, Cortemiglia et al.(1981) rightly pinpoint that some negative features are also apparent, viz.

- onset of erosion in unprotected adjacent areas, due to the trap-effect caused by the area of calm water behind the breakwater
- onset of degradation in the quality of the sand and water in the closed area near the entrance when the tombolo reaches the breakwater. This degradation is accentuated by the concentration in a small area of too many bathers in summer
- rapid sinking into the sea bottom of the breakwaves, requiring maintance work;
- degradation of the aesthetic aspect of the beach;
- erosion and creation of rip currents in the gaps between breakwaters. These may be very strong when the barriers are overtopped by the waves, and very dangerous for inexperienced swimmers.

Hence coastal defence with detached breakwaters has its advantages and disadvantages. The former include i.a. preservation of continuous longshore sediment transport while among the latter one may list relatively high investment costs, particularly due to special equipment for underwater works. Coastal engineers have shown a lot of interest in detached breakwater for the two recent decades, but still the insufficient experience gained so for might bring about a risk of erroneous design while the low number of projects in operation substantiates limited confidence in this typė of structures.

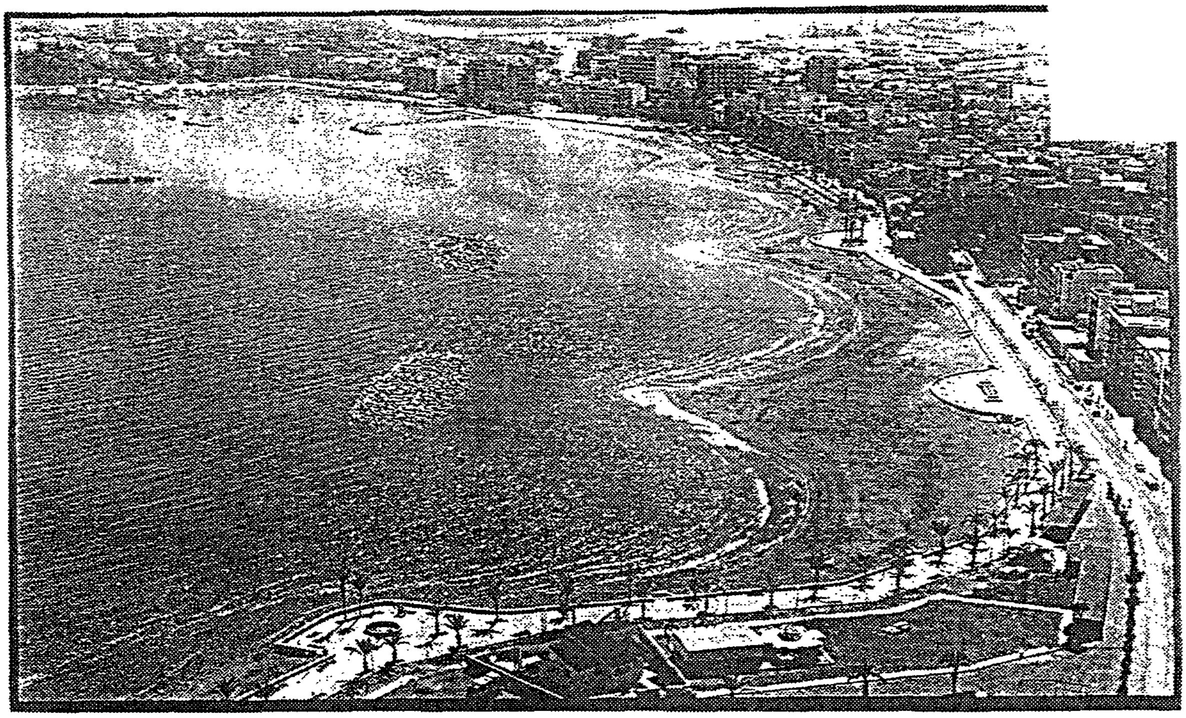


Figure 2.6. Regeneration of Rihuette Beach - Murcia (Spain); *Three detached breakwaters 90, 85 and 80 m long, plus 180,000* m^3 *of sand supplied to form the beach.*

Figure 2.7. Castell de Ferro Beach - Granada (Spain); *Three detached breakwaters, each 80 m long, connected by two submerged breakwaters, 150 m each, plus 130,000* m^3 *of sand and gravel supply.*

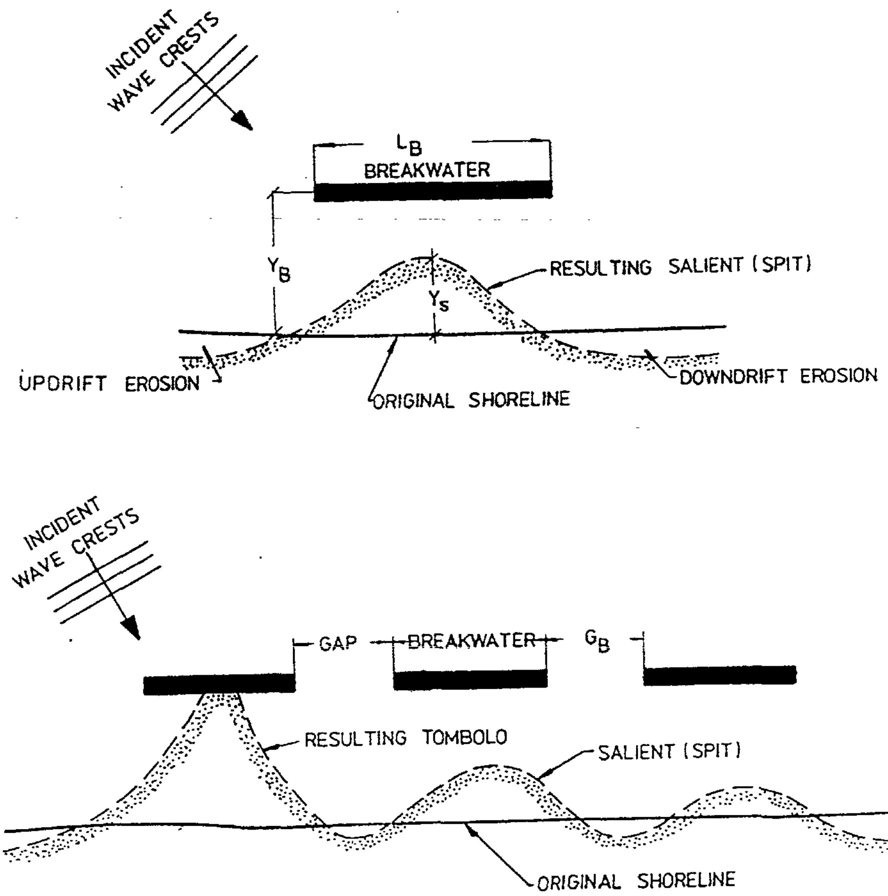


Figure 2.8. Effect of Shore-Parallel Structures: *Single Detached Breakwater (a) and Segmented Detached Breakwater (b)*, Dally and Dean (1986).

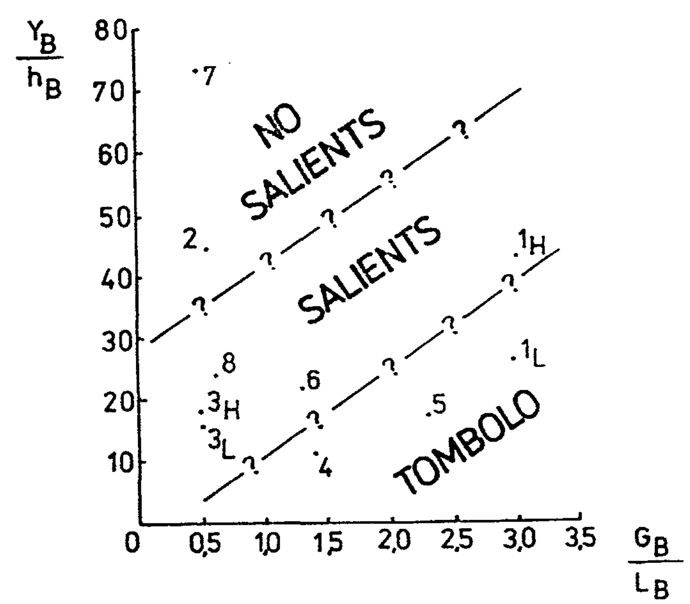


Figure 2.9. Effect of Breakwater Layout on Coastal Features, Pope and Dean (1986).

Coastal transformations in the presence of a detached breakwater are affected by wave diffraction. The latter in turn is controlled by the wave length, height and angle of incidence, and additionally by the relative spacing of segmented breakwaters. Some other parameters involved are the local variation of sea level, slope of shore profile, supply and gradation of sediments.

For segmented breakwaters, an essential role is played by the distribution of wave height in the sheltered area for it affects the relative spacing of the segments. In computation of the diffraction effects Dally and Pope (1986) recommend simple graphs (SPM, 1984).

The type of bed processes seawards of a structure will depend on the existence of breakers (of. Onoszko et al 1980). If an unbroken wave reaches a structure it may be subjected to total or partial reflection, and wave interference is typical for the area seaward of the structure. The size and character of this area depends on whether waves are incoming normally or at angle to the structure.

One notes two detrimental effects of wave reflection from structures. They are bed scouring during storms, which is associated with settling of various revetments and rubble mound and the phenomenon of bed liquefaction which can bring about displacement of weak soil by a heavy structures. Breakwaters are practically seated on various depths which means different position with regard to the breaking of the energy containing wave components.

The angle of wave incidence controls both beach equilibrium and the intensity of accretion. Oblique waves give rise to littoral drift which can dominate local circulation cells generated by a breakwater and also reduces the accretion behind the structure. The intensity of accretion in the sheltered area is controlled not only by the prevailing angle of wave incidence but also by the type of the directional spectrum. If the latter is wide then longer structures are required to reduce the effect of oblique waves.

The equilibrium of tombolos and salients depends on the magnitude of littoral drift and length of structure. If accretion is promoted on the up-drift side then asymmetry of the system is stronger and the accretional form is displaced in the downdrift direction. On the other hand, weak supply of sediment brings about an analogous displacement in the updrift direction.

The operation of shore-parallel structure is also controlled by the variation of sea level. If the latter has amplitudes above 1.5 m then accretion forms of the tombolo type are not continuous features.

The initial bed slope plays a significant role in the selection of the distance of the structure from shoreline.

Generation of tombolo depends on the ratio of the breakwater length to the distance from shoreline. Different criteria are given; Dally and Pope (1986) assume that this ratio is about 1.5.

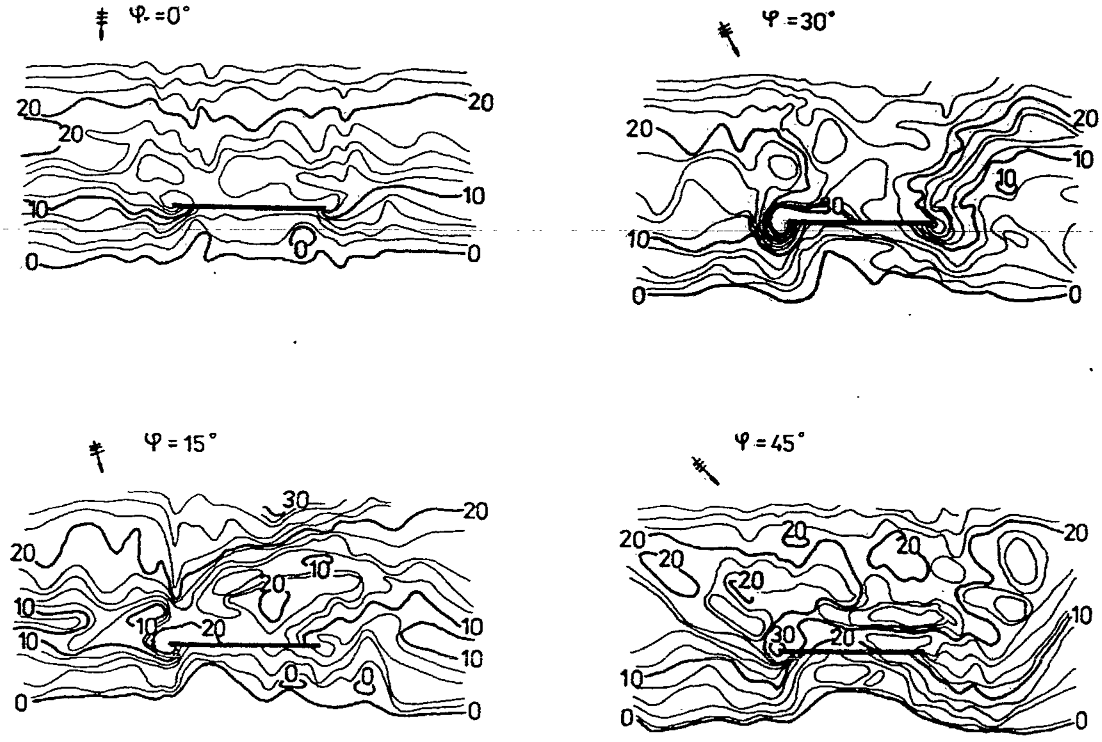


Figure 2.10. Bed Topography at Offshore Breakwater for Relative Distances $\frac{Y_B}{Y_b}$ and Wave Incidence Angles ϕ, Tarnowska et al. (1987).

Existence of stabilized coastal forms depends on sufficient sediment supply. For a weak supply accretional forms develop very slowly without reaching the equilibrium (Dally and Pope, 1986). Artificial beach nourishment should be used in such cases.

Large-scale tests in a movable-bed wave tank conducted for a few years at IBW PAN have shed light on the quality of the processes occurring about single detached breakwaters (Tarnowska, 1989). Fig.2.10 provides examples. The shore transformation has been clearly differrent for the two cases: $Y_B < Y_b$ (breakwater **inside** surf zone) and $Y_B > Y_b$ (breakwater **seawards of** breaking line. In the former case one observed a very intensive erosion at the updrift extremity of the structure and minor erosion on the downdrift side. The sediment was transported away from the zone of wave interference on the seaward side of the structure, and settled in the shadow, where a salient grew. In the case of $Y_B = Y_b$ strong erosion was repeated in front of the structure, together with asymmetric salient (depending on angle of wave incidence), while for $Y_B > Y_b$ thr phenomena were much weaker or did not appear at all.

Single and segmented detached breakwater are practiced in coastal engineering. The former, having various lengths are primarily designated to protect local shoreline while the latter stretch along sections of coastline. Equal lengths of the breakwaters and gaps between them are common practice.

Detached breakwaters are placed on various depths (Toyoshima, 1974) and therefore assume various positions with regard to wave breaking.

Submerged breakwaters

These structures are intended to attenuate waves, primarily through selective breaking of the (highest) waves, and promoting sand accretion in the sheltered area.

Submerged breakwaters are used in coastal zones with prevailing on-offshore mode of sediment transport and insignificant tidal ranges. Water exchange behind submerged breakwaters is better than that for detached breakwaters.

Submerged breakwaters can be constructed in the form of long continuous structures without gaps. The latter give rise to rip currents, bed irregularities and tombolos. The advantages include preservation of environment and relatively low capital investments. Both detached and submerged breakwaters suffer from considerable settling due to scouring at the foot, vibrations and sediment transport under the foot.

Submerged breakwaters, similarly to emerging ones, are intended for prevention of beach erosion. This function is accomplished through entrapment of suspended sediment travelling off shore with the return flow and overtopping the breakwater in the onshore phase of wave motion. One way or another, a submerged breakwater dissipates wave energy and prevents more intensive bed movement.

Aminti et al. (1983) claim that, in certain respects, submerged breakwaters are better than emerging breakwaters. The advantages of submerged breakwaters include the following features:

- they exert a selective action on the waves, stronger on the largest ones, thus assuring better water exchange during normal wave condition
- they may be forseen as long continuous structures, thus avoiding gaps and drawbacks connected with them
- they do not spoil the aesthetic aspect of the beach
- they may result more economical than emerging structures.

A comparison between continuous and discontinuous submerged barriers in terms of average agitation behind them (Cortemiglia et al.,1981) is made for barriers of the same volume at 2.5 m depth; the emerging barrier is 3 m high and has openings along 30 % of its length. Instead, the submerged barrier is 1.5 m high, the slope of the rubble mounds is 2/3 in both cases, and the width of the tops is 2.5 m and 7.1 m respectively. Comparison is extended to the case in which water depth becomes, for example, 3 m because of the tide.

The highest waves, breaking at a 2.5 (3.0) m water depth and being 2.0 (2.4) m high there, cause a mean wave agitation of 1.28 (1.58) m behind the emerging barrier and 0.92 (1.34) m behind the submerged one. For lower waves the submerged barrier is much more permeable, and thus does not need openings for the necrssary continuous exchange of water to and from the internal area. Barrier continuity prevents the formation of tombolos and other hindrances to the longshore current.

The results shown in the literature, with graphs and formulae, give fairly reliable information as regards design, but in most cases they only deal with effects connected with wave motion: transmission, reflection and breaking.

2.3.3 *Some Details; Studies and Predictions*

Detached breakwaters can be constructed as inclined wall structures made of rubble or as vertical wall structures, mostly on steel sheet piling or piles. The rubble structures are primarily designed for considerable depths with strong waves. The vertical-wall breakwaters usually apply for areas with relatively weak such as Great Lakes, gulfs etc. (cf. S.P.M. 1977).

Because of a relatively small bulk of data available for the operation of detached breakwaters, the prototype observations in USA, Israel and Japan will be utilized in this report.

Emerging Breakwaters

The first detached breakwaters in Israel were constructed at the turn of the sixtees (Nir, 1982). The breakwaters were usually sited two hundred meters from the original shoreline, on depths from 3 to 4 m on bare rock. The selection of a site was dictated by economic factors, mostly foundation. The lengths of breakwater segments were equal to the distance from shoreline. Accretion forms of the tombolo type were generated between the structures and the shoreline. Majority of these forms reached their final shape after one or two years, while in next three years one saw the mature form. The area of the tombolo was 40% to 50% of the rectangular side behind the structure. Upon the maturity only seasonal changes were observed, i.e. growth in the summer and weakening of the form in the winter.

Interesting observations were done after two years from completion of the structure. The amount of the accumulated sand decreased clearly during that time which was attributed to a weaker supply from the neighbouring regions.

It was also observed that there was a relationship between the length L_B of a structure and its distance from shoreline. This is usually expressed by the ratio Y_B/L_B. The higher the ratio Y_B/L_B the lower the amount of sand accumulated behind a structure. Accretion is very small or does not occur at all if the ratio Y_B/L_B is equal to or greater than 2 (Fig.2.11). This critical value is slightly smaller than that given by Imman and Frautschy (1966) who proposed 3 to 6, basing on their studies in California. The Israeli structures have not that high figures of Y_B/L_B.

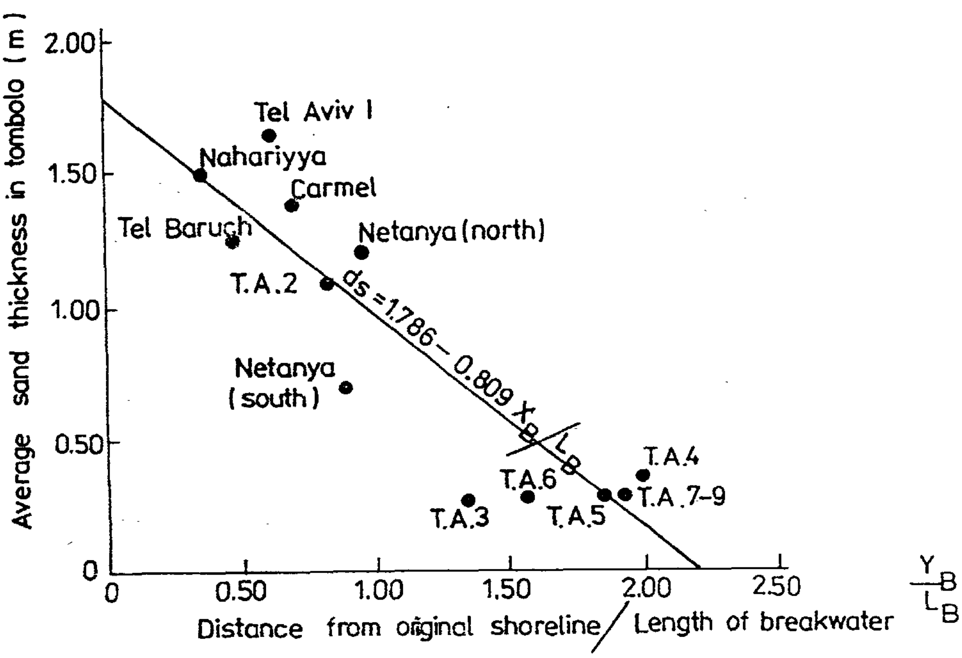


Figure 2.11. Sediment Quantity Entrapped Behind Breakwater as Function of $\frac{Y_B}{L_B}$, Nir (1982).

Nir (1982) indicates that the ratio Y_B/L_B also determines the thickness of the sand layer accumulated in the tombolo. This relationship can be given by the following equation

$$d_t = 1.786 - 0.809\frac{Y_B}{L_B} \tag{2. 1}$$

in which:

d_t = mean thickness of sand layer

Y_B = distance of breakwater from shoreline

L_B = length of breakwater

Nir has also assessed the quantities of sand accumulated most early in the dry part of the accretional form. These quantities are given by the Hall formula:

$$v = \frac{d \cdot c(2a + b)}{3} \tag{2. 2}$$

in which

$2a$ = length of the section along which tombolo is connected to the structure

b = half of tombolo base menasured along shoreline

c = distance of tombolo apex from shoreline

d = depth of water measured at tombolo apex.

Nir sugests that similar quantities of sand were accumulated under water.

The first experimental detached breakwater were built in Japan in 1966 (Toyoshima, 1982). The success of this project gave rise to a system of detached breakwaters proposed to protect Kaiko coast. The first breakwater having length of 150 m was constructed there on a depth of about 5 m a distance of 110 m from shoreline. The structure was implemented from rubble mound and tetrapodes. The breakwater has proved very effective. The thickness of the sand accumulated about the apex of the salient was about 4 m, versus about 2 m at the shoreline in front of the sea wall. Again, the sucess of this project stimulated the construction of ten other breakwaters spaced equally every 50 m. As a result, sand was accumulated not only on the lesside of the structures but also seawards of the breakwaters, thus decreasing the bed slope and smoothing out the isobaths. The latter tendency could not have been explained.

Rosen and Vajda (1982) summarised their own and other data to produce Fig.2.12 and Fig.2.13.

Relatively little experience has been gained in the USA for the operation of segmented detached breakwaters. Single detached breakwaters have been in use since long both as submerged breakwaters localized close to shoreline and high structures emerging above still water level, situated in deep water, and connected to other structures.

The detached breakwaters in use in the United States are straight structures, shore parallel, mostly in the form of rubble mound. The design lengths reach 600 m, and the distances from shoreline vary from 46 m to 600 m. The top of a structure varies from + 0.4 m to + 5.5 m above mean water level. These structures are founded on depth from 0.3 m to 7.6 m. Dally and Pope (1986) provided characteristic parameters for single and segmented detached breakwaters. From the data it follows that a tombolo was formed if the ratio was close to one. In addition, the authors have given a theoretical relationship for this ratio in the case of perpendicular wave incidence and constant depth on the landward side of a structure. According to these computations, a tombolo can be generated both behind a single breakwater and between segments of group breakwaters and shoreline provided $L_b \geq 2Y_B$. It must be noted that a number of factors, such as depth changes, wave breaking, effect of wave frequency and nonlinear effects have been neglected. These circumstances, together with the observations on tombolo for $L_B = Y_B$ forced Dally and Pope to suggest that the design criterion of the tombolo should be taken as $L_B \geq 1.5Y_B$.

Segmented breakwaters are usually constructed if the length of a protected coastline is at least 5 times greater than the design distance of breakwater from shoreline. If complete tombolo is desirable then each of the segments should be up to two times longer than its distance from shoreline. These parameters should be tailored to the anticipated form of shoreline. Dally and Pope say that the gaps between segments should be two mean wave lengths for a period of a relatively

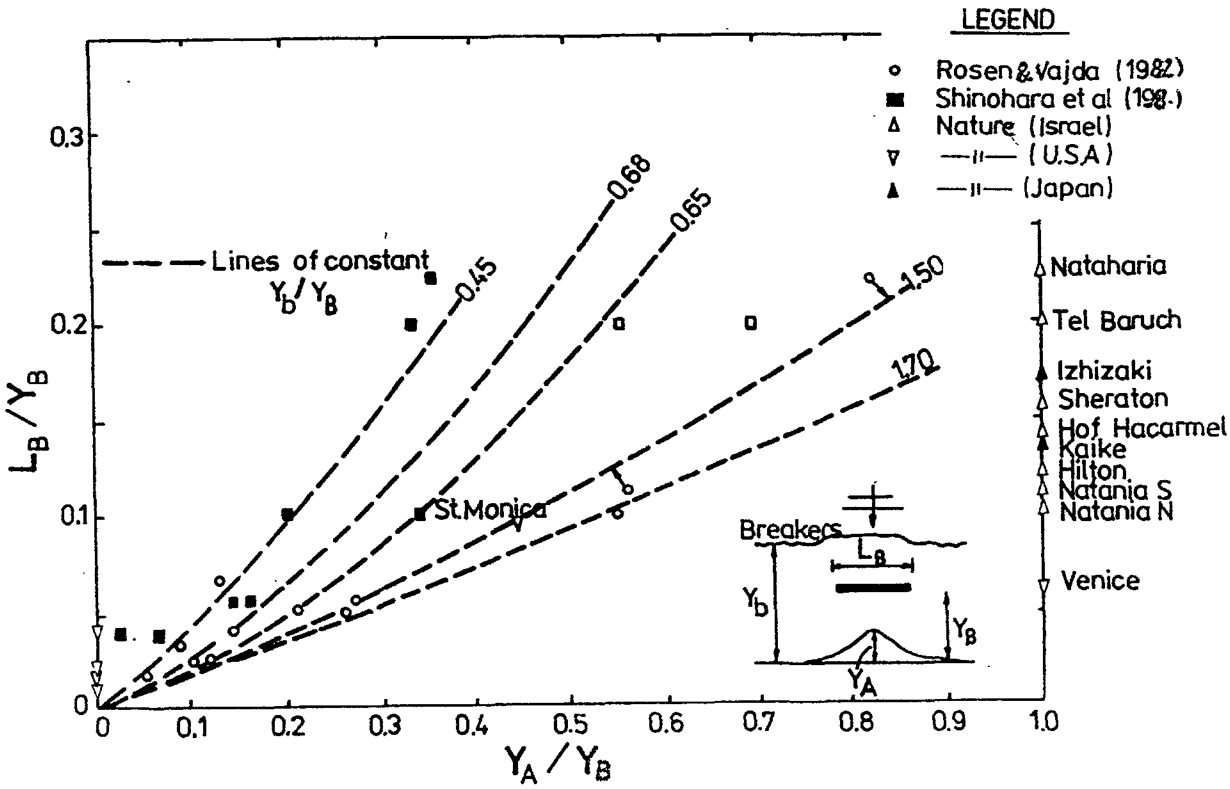


Figure 2.12. Equilibrium Relationships for Salient Parameters, Rosen and Vajda (1982).

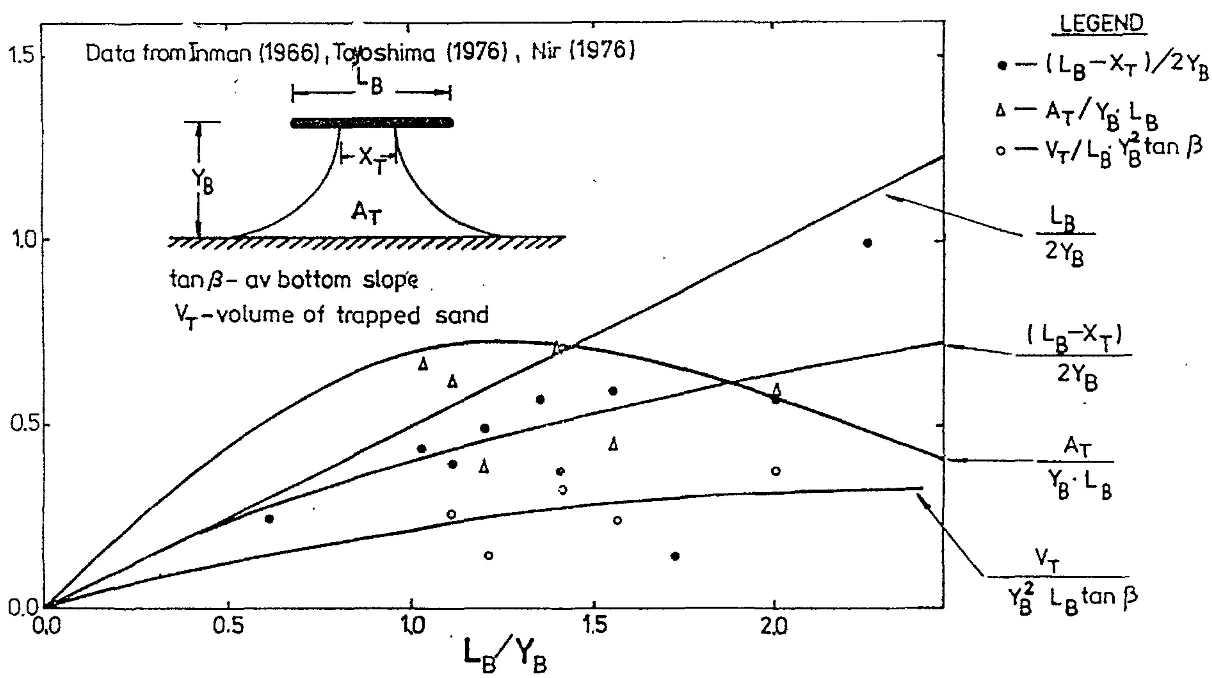


Figure 2.13. Equilibrium Relationships for Tombolo Parameters, Rosen and Vajda (1982).

calm weather. Long gaps can bring about sinusoidal shoreline of high amplitude, while small length of segments and small gaps can promote a smoother shoreline.

Detached breakwaters have not yet been tried in Poland. Slomianko (1971) mentioned the operation of shore-parallel amusement pier in Kolobrzeg, which can be identified as a longshore structure. Shortly after the shore-parallel segment was completed clear-cut shallowing at shoreline was observed which subsequently increased its area.

The observations by various authors cited do not yet substantiate any generalisation. The investigators emphasize that an accurate prediction of shore transformation cannot be reached at the present state of the art.

Laboratory tests of Harris and Herbich (1986) provided the following relationship of the accumulated sediment as a function of relative length of structure:

$$\frac{Q}{Y_b L_B h_B} = exp\{31481 - 192187(Y_B/L_B)\} \tag{2. 3}$$

This relationship was verified by field data from the USA, Japan and Israel and laboratory data by Shinohara.

Seiji, Uda and Tanaka (1987) have endeavoured a relationship of the accretion behind a breakwater and relative depth h_B/h_b and relative breakwater length $\frac{L_B}{Y_B}$. In addition, the effect of shoreline at gaps between segmented breakwaters was also given in terms of $\frac{G_B}{L_B}, \frac{G_B}{Y_B}, \frac{L_B}{Y_B}$.

Laboratory tests have been conducted by Suh and Dalrymple (1987). The results refer to a single breakwater as well as to a system of segmented breakwaters. The tests were carried out in a spiral wave tank and concentric topography. The results have been compared with field investigations of other authors and are presented graphically. It has been found that the relative length of the structure L_B/Y_B is an important factor affecting the quantity of entrapped sediment. This is an important factor affecting the quantity of entrapped sediment. The spit amplitude Y_s also increasses with increasing length of the structure.

Depending on the location of the structure relative to wave breaking the spit amplitude is respectively (Fig.2.14):

- for $Y_b/Y_B \leq 0.5$ i.e. for a remotely situated structure, seaward of surf zone; $Y_s/Y_B = 0.156$
- for $0.5 \leq Y_b/Y_B \leq 1.0$ ie. for a structure in the outer nearshore zone closer to break point $Y_s/Y_B = 0.377 \cdot L_B/Y_B$.

It may be concluded that for the same relative length of breakwater the structure located more closely to shoreline is more effective.

To determine the efficiency of group breakwaters Suh and Dalrymple have proposed a dimensionless parameter which relates the gaps to the length of the system segments.

The following formula has been presented for the efficiency of a system of breakwaters expressed in terms of the spit amplitude Y_B

$$\frac{Y_S}{Y_B} = 14.8 G_B \frac{Y_B}{L_B^2} exp\{-\frac{2.83(G_B Y_B)^{1/2}}{L_B}\} \tag{2. 4}$$

From this equation it follows that the relative spit amplitude increases with increasing parameter $\frac{G_B Y_B}{L_B}$ (Fig.2.15). This amplitude becomes maximum $Y_S/Y_B = 1$ for the parameter $\frac{G_B Y_B}{L_B} = 0.5$. Once this critical value is exceeded the spit amplitude falls dramatically.

It has been emphasized that the efficiency of detached breakwaters in terms of the relative spit amplitude is better in field than in laboratory.

A graphical method for the assessment of the effectiveness of segmented breakwaters constructed of artifical armour units has been presented by Spataru (1986). The operation and effectiveness of a structure may be determined for a given relative height of structure and depth of water $\frac{h_B}{h}$ (Fig.2.16); wave dissipation and reflection and permeability of a structure are also taken into account. Spataru's method refers to submerged breakwaters, and is also acceptable for emerging breakwaters.

Submerged breakwaters

Shore evolution is closely related to transmission of wave energy towards beach. Upon prediction of wave flux one can indirectly model the eventual bed changes. Using this type of control one may enforce erosion or accretion. The review of studies on shore parallel submerged structures and their effect on coastal evolution is given by Onoszko, Tarnowska and Zeidler (1980). This type of structures is most common in Japan and the USSR. The review embodies the studies up to 1980 and includes primarily the effect of structures on changes in regular wave height. Mei and Black (1969) provide theoretical background basing on small amplitude waves and intended for submerged structures having various widths at their top.

A study by Aftanas (1973) should be mentioned among the Polish laboratory investigations on the dynamics about a submerged breakwater of triangular cross-section and relative height (0.2...0.3) h. Significant eddies about the structure were exposed, associated with a medium size scour on the lee side of the breakwater and a shallower but broader scour on the seaward side. The depth of scour was increasing for higher wave steepnesses. The maximum scour parameters corresponded to wave breaking on the breakwater.

Homma and Horikawa (1961) have concluded from their laboratory and field tests that the efficiency of a submerged breakwater is highest if its relative height is 0.7..:0.8.

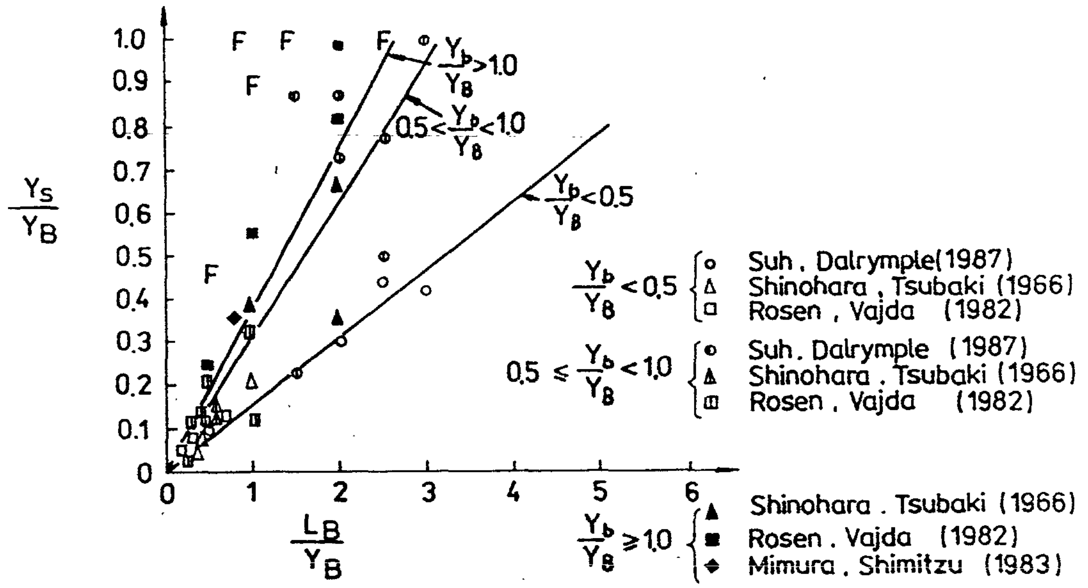


Figure 2.14. Salient Amplitude, Suh and Dalrymple (1987).

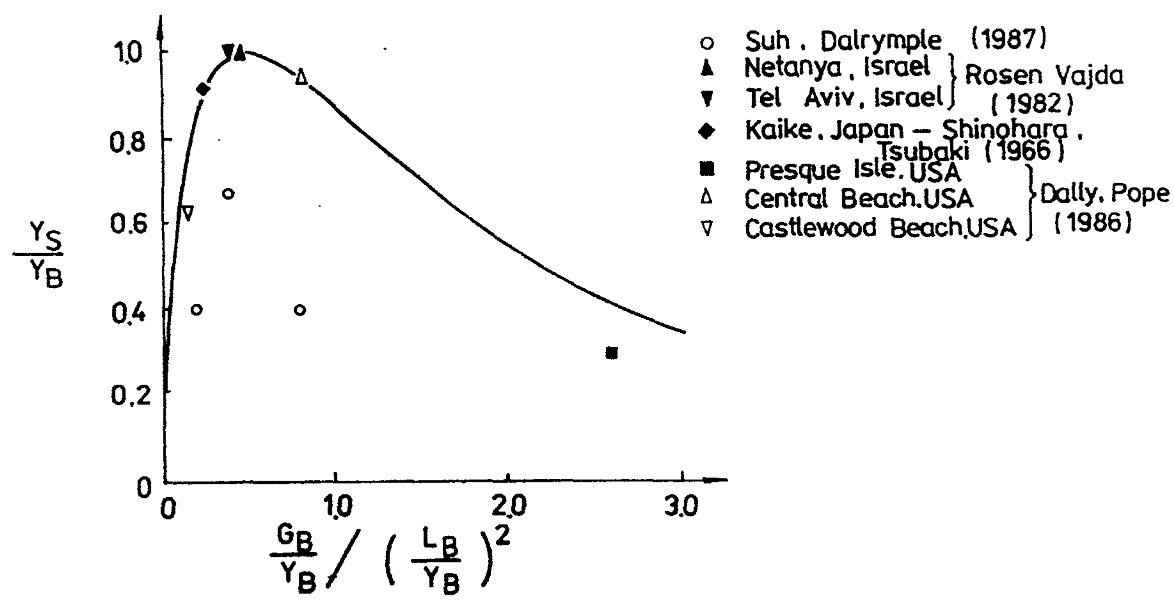


Figure 2.15. Salient Amplitude, Suh and Dalrymple (1987).

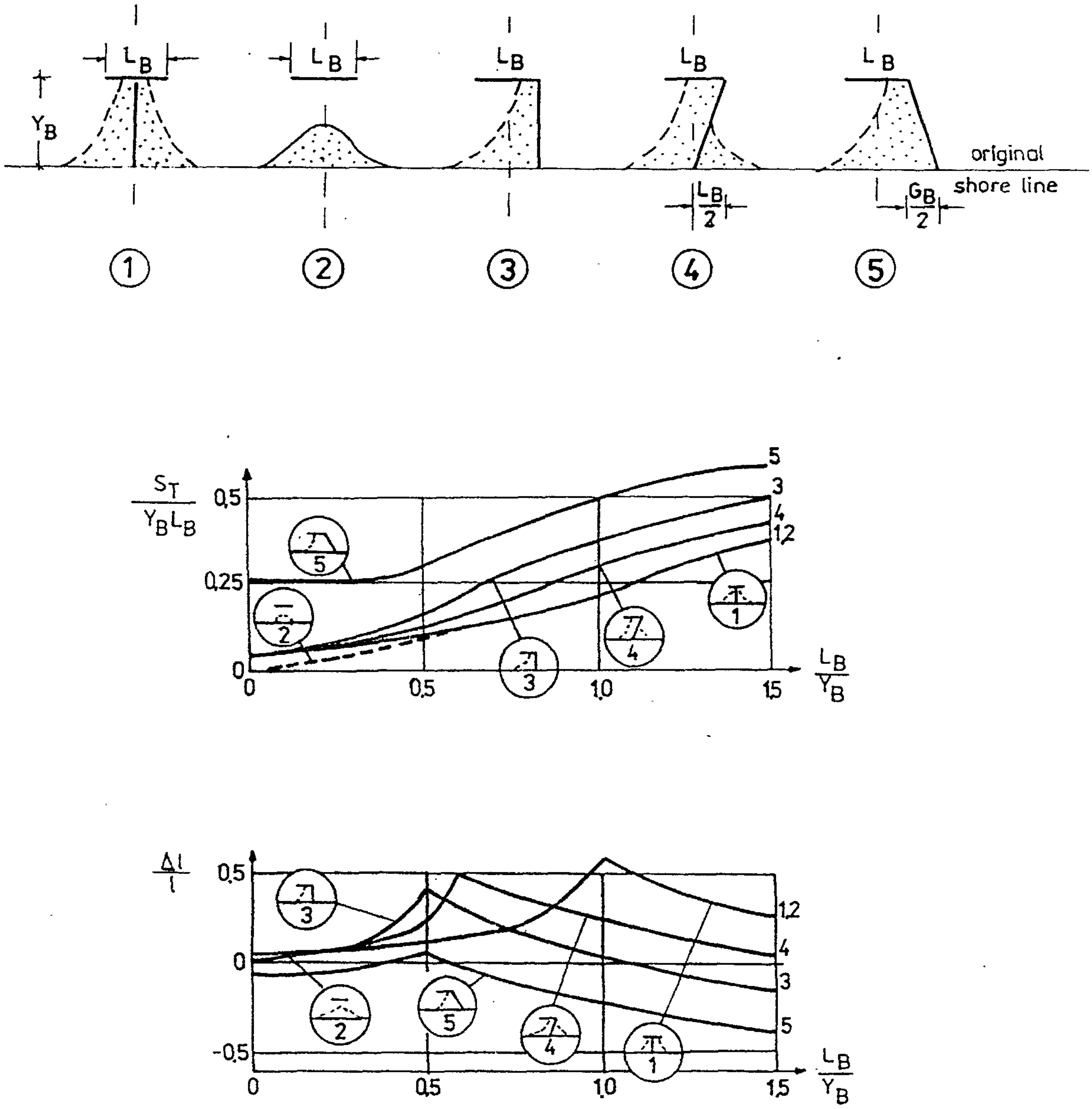


Figure 2.16. Effect of Offshore Breakwater Layout on Accretion Patterns, Spataru (1986).

One of the most recent studies was published by Baba (1986). It summarises the existing computational methods for wave attenuation and coastal defence with a shore-parallel submerged breakwater. This analysis basing on the laboratory tests has shown that Goda's theory is the best among the methods assessed (Lizov, 1962, Goda, 1967 and 1969, and Seeling 1980). The analytical background is provided by the following data on wave transmission

$$K_T = \frac{H_t}{H_0} = 0.5[1 - \sin\frac{\pi}{2\chi}(\frac{h_p}{H_0} + \beta)] \tag{2. 5}$$

in which

H_0, H_t = initial and transmitted wave height, respectively

χ and β = empirical factors being

$$\left.\begin{aligned}\chi &= 2.2\\ \beta &= 0.0...0.8\end{aligned}\right\}\quad \text{for vertical wall} \tag{2. 6}$$

$\beta = 0.1 - 0.35$ for complex shape of the front side of the breakwater

$h_p = h - h_B$ = submergence of breakwater top.

The method is applicable for

$$0.8 \leq B/h \leq 1.1 \tag{2. 7}$$

$$0.41 \leq h/L \leq 0.5 \tag{2. 8}$$

in which

B = breakwater width at top.

Results of the computations of wave decay due to a shore parallel submerged breakwater should be assessed with care. This stems from the fact that the available forecast methods are based on laboratory investigations for regular waves, Mei and Black (1969), Baba (1986). The applicability and clearness of these models also depend on a number of empirical lump factors which are usually selected quite arbitrarily.

2.4 SEA WALLS (BULKHEADS) AND REVETMENTS

The terms **bulkhead** and **sea wall** are often used interchangeably, although, strictly speaking, bulkheads are retaining walls constructed to prevent sliding of soil and moderate protection while primary purpose of seawalls is to protect the backshore from heavy wave action. A revetment is placed on a slope to protect it and adjacent uplands from erosion and to dissipate wave energy. Both sea walls (bulkheads) and revetments serve as the last resort in the combat of waves at the

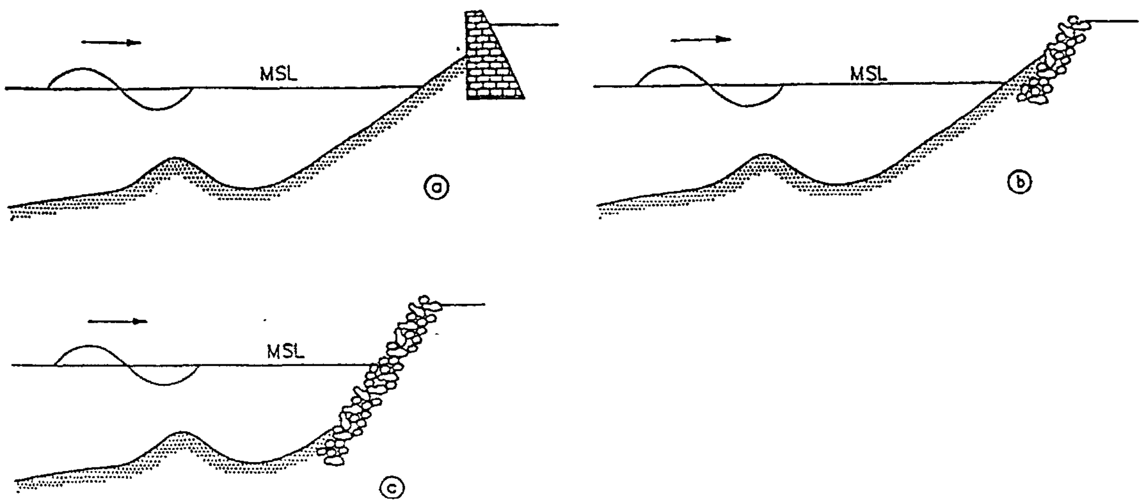


Figure 2.17. Shore-Parallel Structures Analysed by Graaff & Bijker (1988).

sea-land interface.

Sea walls and revetments are shore-parallel structures meant to protect the hinterland against flooding and/or erosion. They are situated about such features as bluffs, scarps, embankments, cliffs, dunes and promenades.

Sea walls on sandy beaches are analysed by Graaff and Bijker (1988). Fig.2.17 shows three typical cross-sections with a shore-parallel structures. Cases (a) and (b) are considered by Graaff and Bijker (presence of sandy beach in front of the structure) while absence of beach is not discussed.

Case (a) is typical for well-developed seaside resorts. On the benefit of a prosperous development often a clear distinction between 'sea' and 'land' is desired. Case (b) is typical for important investments which are apparently at stake. The encroaching sea obviously causes erosion of the beaches and the dunes. With the shore-parallel construction the attack of the sea is intended to be beaten off.

All over the world, however, damaged and even entirely demolished seawalls can be found. In many cases it is felt that in fact design errors are the main cause of the damage; in some other near-field effects are to blame. The latter, primarily local scour, are dealt with in Chapter 3, while this section is devoted to some far-field effects.

Erosion problems about sea walls

Seawalls are often built to overcome the most pronounced problems of our coasts. In principle two basic erosion problems with sandy coasts do exist, viz.

- erosion/recession during a single storm (surge) event
- gradual long term erosion

Fig.2.18 shows in plan view a stretch of a coast at a certain moment in time (under 'usual' sea conditions). The problems (a) and (b) can be clearly illustrated if the behaviour of cross-section A-A of Fig.2.18 is considered as a function of a long time. For reasons of simplicity the behaviour of the position with respect to the reference line of the so-called dune-foot is considered to be representative for the behaviour of the entire cross-section (other characteristic profile features could also be selected). The dune-foot is the intersection line between the gentle beach slope and the steep slope of the dune front.

Fig.2.19 shows three typical possibilities of the behaviour of a sandy coast.

Fig.2.19 a represents an essentially stable coastline. However, storm (surge) events cause sudden recessions of the position of the dune-foot. Since the cross-section in consideration is, seen over a longer period, stable, a recovery of the dunes will take place in the years after the storm event. Depending on the seriousness of the storm event the magnitude of the recession can vary considerably. Fig.2.19 a represents in fact case (a) indicated in this section.

Fig.2.19 b shows a gradually eroding coast (with surge events superimposed on that). The recovery after a surge is not entire. Fig.2.19 b illustrates case (b).

For the sake of completeness shown in Fig.2.19 c is an accreting coast (also with storm events), although such cases do not pose problems to the coastal manager.

Erosion of dunes and the upper part of the beaches can occur during a severe storm. The rate of recession during that event depends on the seriousness of the storm involved. During the storm not only the wave attack is greater than during usual conditions, but also the water level (surge level) increases to levels (far) higher than usual. Along coasts bordering oceans the increase in water level during the passage of a storm is often moderate; along coasts bordering funnel- -shaped seas the increase of the water level may mount several meters.

Fig.2.20 shows schematically what happens during a severe surge. Material of the dunes is eroded and (mostly) settled again on the foreshore. Since the shape of the profile becomes less steep, the erosion process (the rate of erosion) slows down with time. After the surge a retreat distance RD can be observed.

Dean (1986) made an attempt to conduct a rational assessment of the potential adverse effects of coastal armouring on adjacent shorelines and to propose methodology for mitigation, where appropriate. Specific attention is directed toward claims that armouring causes: profile steepening, increased longshore sediment transport, intensified local scour, transport of sand to substantial offshore distances, etc.

Coastal armouring in the form of seawalls or revetments is usually designed to be located along sandy shorelines which are either experiencing an erosional trend or which are subject to substantial seasonal swings and/or storm-induced fluctuations that could conceivably endanger upland structures.

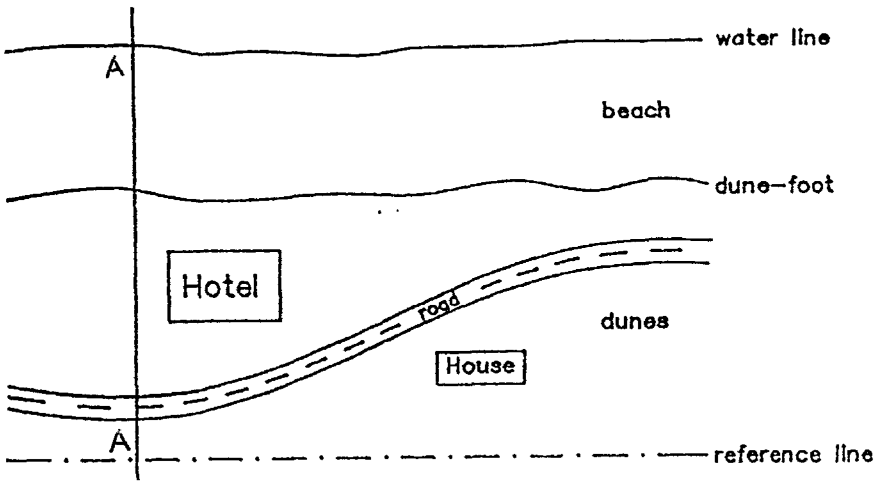


Figure 2.18. Plan View of Coast, Graaff & Bijker, 1988.

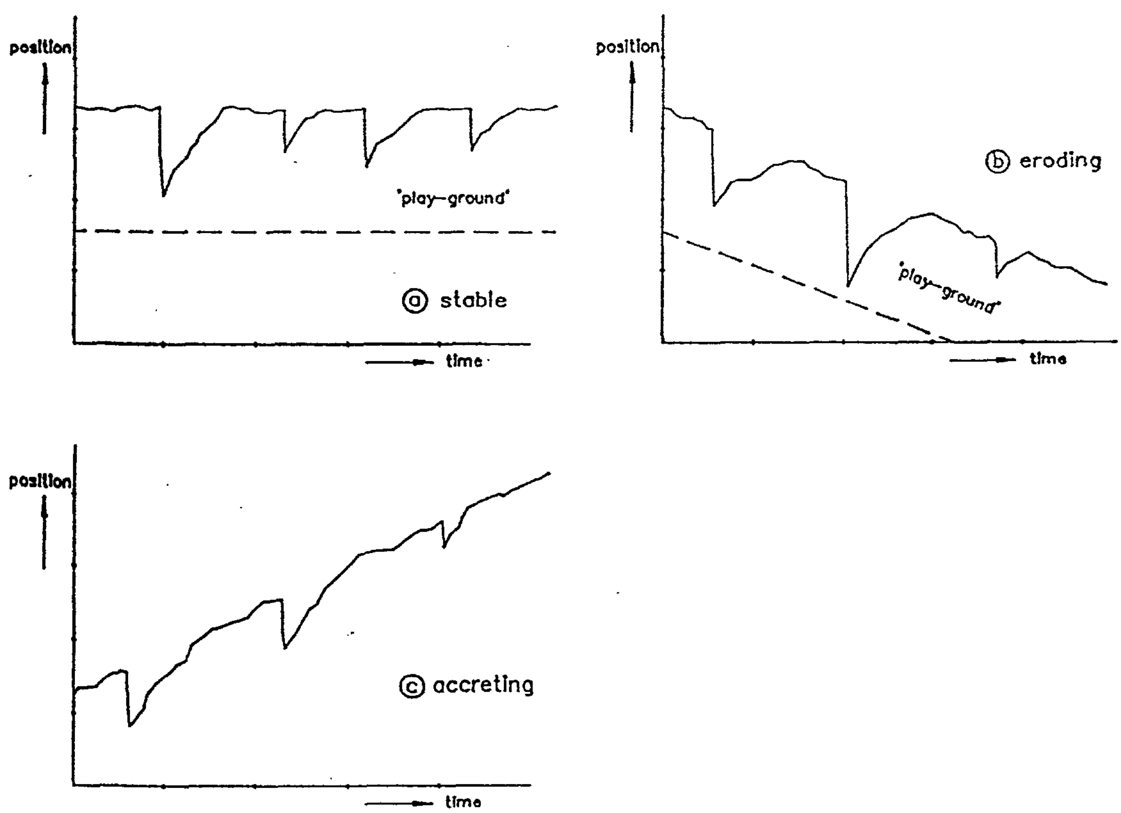


Figure 2.19. Three Types of Coast Behaviour, Graaff & Bijker, 1988

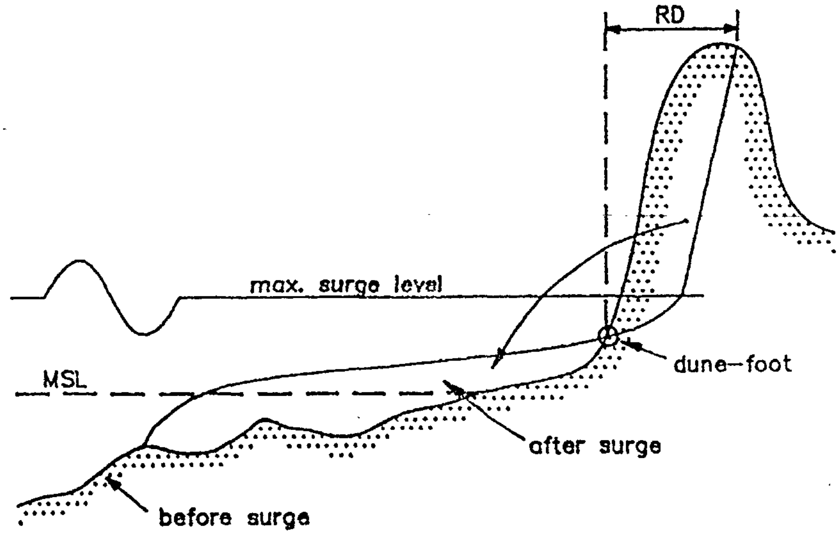


Figure 2.20. Dune Erosion Process.

Coastal armouring, once regarded as a relatively desirable means of achieving shoreline stabilization, has recently been the focus of much attention including various concerns over possible adverse effects to the adjacent beach system, see for example Kana and Svetlichny (1982). An analysis of the available literature demonstrates that definitive laboratory or field data are sparse, thereby providing a fertile ground for conjecture and misinformation. Placed along a shoreline with an erosional trend, armouring can perform its intended function of upland stabilization while the adjacent shoreline segments continue to erode. The resulting offset between stabilized and unstabilized segments may be interpreted incorrectly that the armouring has caused the adjacent erosion.

Coastal amouring does have the potential of causing adverse effects to the adjacent shoreline. In situations where coastal armouring is contemplated, it is important to recognize, and where possible to quantity and perhaps mitigate for these adverse effects. Specific alleged adverse effects of coastal armouring include: (1) offshore profile steepening, (2) intensified local scour, (3) transport of sand to a substantial distance offshore, (4) adverse downdrift erosion, and (5) delayed post-storm recovery. We shall herewith Follow Dean(1986) in his arguments on the above issues.

Offshore Profile Steepening

Increased wave reflection can clearly occur as a result of coastal armouring; however, there does not appear to be a mechanism for an associated offshore transport to significant depths nor is there evidence to support such transport.

Storm-Induced Intensified Local Scour

During storms, it is expected that increased scour would occur at the toe of the vertical seawall. This is consistent with data presented by Kriebel et al. (1986) for post-hurricane Elena profiles in which, relative to natural profiles, accentuated toe scour occured at the base of seawalls.

In three dimensions, since immediately in front of armouring there is insufficient sand to satisfy the "demand" of the offshore bar, the shore parallel downward slope to the area immediately seaward of the unarmoured segment, combined with the mobilizing effects of breaking waves causes sediment to flow from the region offshore of the natural shoreline to that offshore of the armoured segment. The effect of this interaction is to cause an additional "erosional stress" adjacent to the armouring with the magnitude of this stress increasing with the length of the armouring. This interaction is illustrated in Fig.2.21 and is consistent with post-hurricane Eloise measurements as reported by Walton and Sensabaugh (1979), see Fig.2.22.

Projection of Armouring into the Active Surf Zone

If an isolated armoured segment is constructed on an eroding shoreline where a substantial longshore sediment transport exists, the armouring will in time project into the surf zone and will act as a groin to block the net longshore sediment transport. The annual deficit of sediment downdrift of the armouring will be the sum of that blocked by the projecting armouring and that not yielded by the upland protected by the armouring. The downdrift annual deficit will thus increase with: (a) the length of the armouring, and (b) time as a result of increasing projection into the surf zone thereby blocking a greater and greater fraction of the longshore sediment transport.

Effect of Wave Reflection on Longshore Sediment Transport

Dean (1986) argues that the effect of wave reflection is to **reduce** the longshore sediment transport.

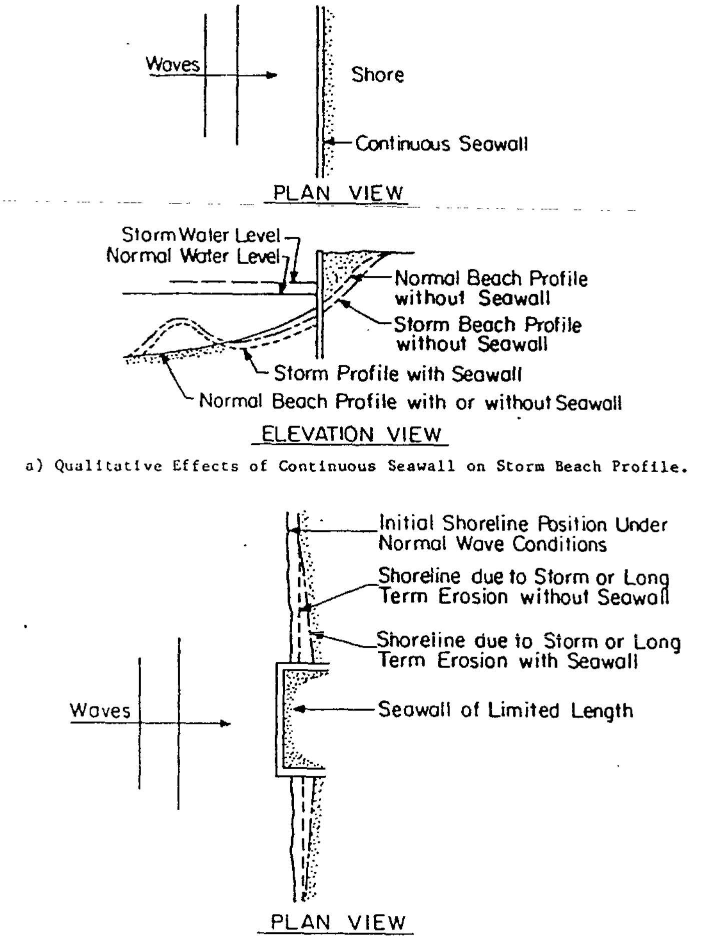
Interference with Post-Storm Recovery

Wave reflection from coastal armouring could be the cause of a delayed post-storm recovery. Data presented by Kriebel, et. al. (1986) from Hurricane Elena supports an equally rapid or nearly equally rapid recovery adjacent to coastal armouring. Moreover, observations by Mr. Ralph Clark immediately after Hurricane Elena (September, 1985) and approximately eight months later (May, 1986) indicate that recovery had occurred to at least the pre-storm condition.

Dean (1986) concludes that

1. There are no factual data to support claims that armouring causes: profile steepening, increased longshore transport, transport of sand to a substantial distance offshore, or delayed post-storm recovery

2. The interaction of an armoured segment of shoreline with the littoral systems depends on the amount of sand in the system vis-a-vis the equilibrium beach profile for the prevailing tide and wave conditions



a) Qualitative Effects of Continuous Seawall on Storm Beach Profile.

b) Effect of Seawall of Limited Length on Storm or Long-Term Beach Planform.

Figure 2.21. Two- and Three-Dimensional Effects of a Sea Wall on Beach System During Storm; Dean (1986).

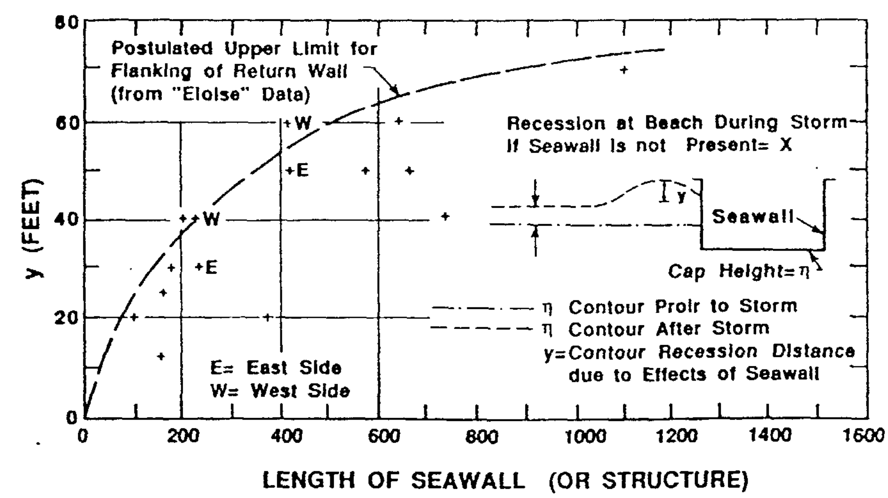


Figure 2.22. Additional Bluff Recession due to Sea Wall; Based on Walton & Sensabaugh (1979).

3. Armoring can cause localized additional storm scour, both in front of and at the ends of the armouring. A simple sediment supply&demand argument is proposed to explain the scour.

The arguments presented by Kraus (1989) and others in Journal of Coastal Research No 4 (1989) seem more conclusive and substantiated by empirical evidence, at least in the near-field domain, cf. Chapter Three.

Another set of arguments is provided by Dette (1987) who examines the history and experience of the sea wall constructed on the Island of Sylt, where the armouring effects discussed above have surfaced at length.

More than 100 years ago men started on the Island of Sylt (North Sea) to interfere with a natural long-term coastal recession of sandy beaches and dunes. Groyne systems did not prove the expected success. In 1907 a 70-m long seawall was built by a private owner in order to prevent further recession in front of an endangered hotel. Already in 1912 an extension became necessary because of the downdrift erosion. Nowadays the chain of coastal structures has reached a length of more than 3 km in front of the city of Westerland.

Dette (1987) presents a chronological review since 1865 of all types of coastal structures, with special emphasis on the seawall.

The 40-km long west coast of the Island of Sylt has suffered since centuries from continuous coastal recession and has been strongly endangered during severe storm surges. The coastline can be divided in two characteristic sections:

[1.] Approx. 22 m long central part of the island with a chain of cliffs and dunes (up to 25 m high).

The typical offshore profile consists of the dune, the beach and in the foreshore of a trough and a longshore bar (also referred to as "ridge and runnel", approx. 400 m distant from the shoreline.

[2.] The northern (8 km) and the southern (7 km) parts of the island can be considered large sand spits which originate from alternating longshore sediment transport along the central part of the island.

Already in 1867 three heavy stone groynes were built in front of Westerland. Until 1900 another 44 of such main works at distances of 500 m with 2 intermediate lighter groynes in between were constructed for protection of the central part (22 km) of the island. These crossing works only could retard the recession process. Where, however, recession should be stopped, special measures had to be undertaken. So in 1907, after another heavy storm surge, a 70 m long seawall in Westerland was constructed by a private owner in order to protect his hotel "Miramar" which then was left only 12 m apart from the dune's edge. After further storm surges and lee-erosions the seawall was lengthened northward in 1912, southward in 1923 and once more northward in 1924 (Fig.2.23). At this time it had become necessary already to strengthen the earlier seawall construction by means of toe protections.

The years between 1924 and 1936 were fairly inactive. In winter 1936/37 two heavy storm surges occurred and caused dune recession in between 10 m and 12 m northward of the seawall. After a 20-m wide beach fill in front of the dune as immediate remedial measure in elongation of the seawall a 1 to 4 revetment consisting of basalt blocks was built in 1937/38 over a length of 510 m. Those days the revetment was expected a reduced wave impact on the beach in front of the seawall. Fig.2.24 depicts the dune erosion since 1888 in this area, the dune losses in 1936, the seaward shifted location of the revetment and the development of the beach profile in front of the revetment until 1967.

In 1946 the northern end of the seawall was replaced over a length of 3,140 m by a revetment consisting of concrete slabs. This construction followed destruction of a 60-m long section of the seawall during a storm surge. In 1950 it became necessary to enlarge the toe protection in front of the seawall by additional width of 5 m; this was in the wake of continuous undermining of the beach. This process is illustrated in Fig.2.25 where the dune recession since 1891 is plotted together with development of the beach profile in front of the seawall between 1920 and 1967.

Due to considerable lee-erosion at the northern end of the revetment it was necessary to extent the revetment in 1954 by 200 m. In December 1954 already another heavy storm surge occurred and again heavy lee-erosion was caused, this time at the end of the elongated revetment.

After 1954 the length of coastal works had reached 1585 m (710 m long seawall and 875 m long revetment). Furthermore concrete pile groynes and 2 flat groynes were constructed in order to prevent further beach erosion. In 1961 a critical part of the seawall, a 100 m long part at the northern end, had to be strengthened. It was decided to place 6-ton tetrapods with a slope of 1 to 1.33 in front of the seawall. From this measure it was expected that the incoming and reflected waves would be attenuated so much that the seawall would not be endangered any longer and further erosion of the beach could be prevented. In order to protect the sheet-pile toe protection which then was already 14 m distant from the seawall this toe protection was extended by an additional width of 10 m. By this means a "3rd generation of toe protection" was created (!).

In the year 1962 the German North Sea coastline was hit by a disastrous storm surge which was classified as a centennial event, (followed already in 1976 and 1981 by even higher storm surges). The 1962 storm surge caused at the northern end of the revetment dune recession in between 10 m and 12 m. In order to prevent here a breach in the narrow dunes and a possible flooding of the city of Westerland, a 420 m long "tetrapod dam" was built as extension of the revetment (Fig.2.26). Also the southern end of the seawall had to be extended by a 300 m long "tetrapod dam" due to lee-erosion.

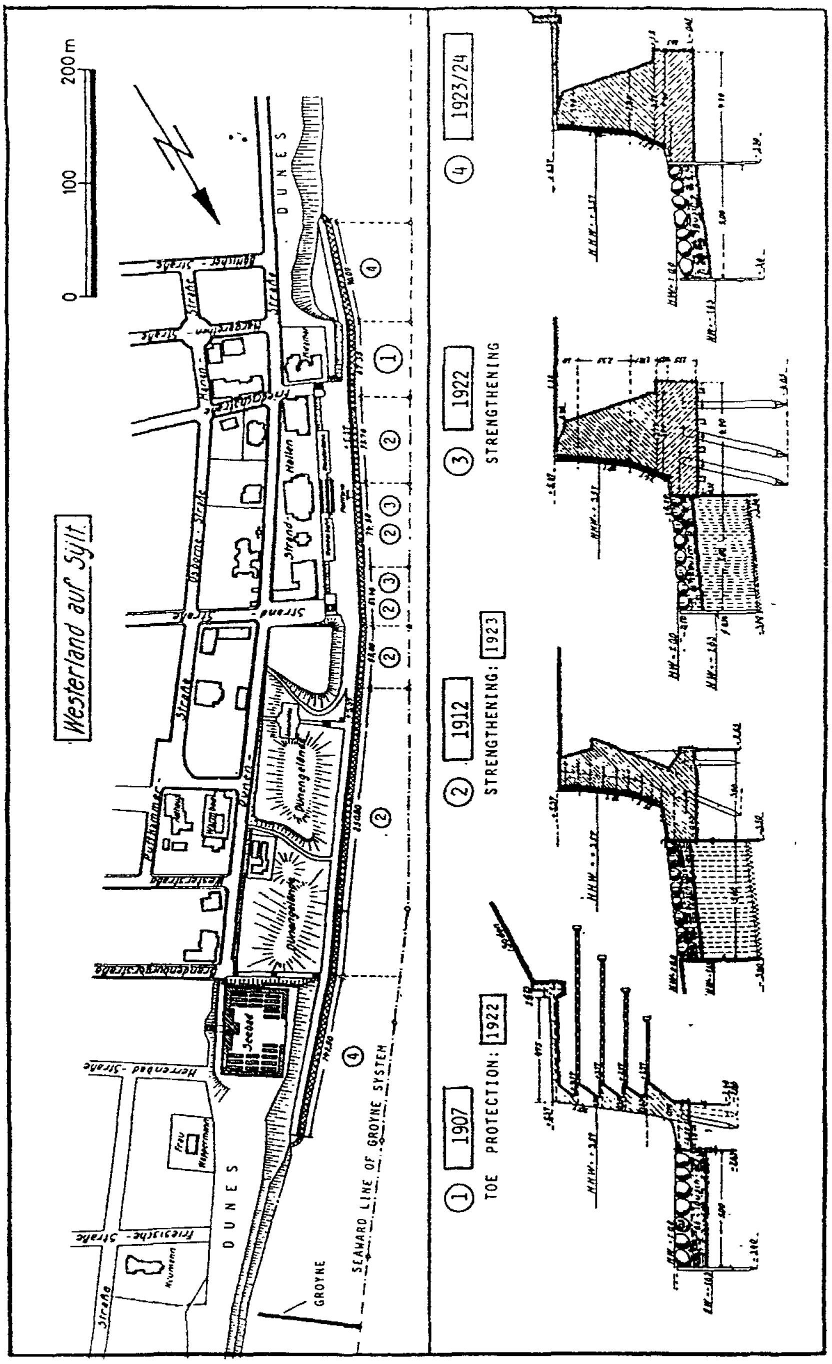


Figure 2.23. Westerland Sea Wall Between 1907 and 1924, Dette (1987).

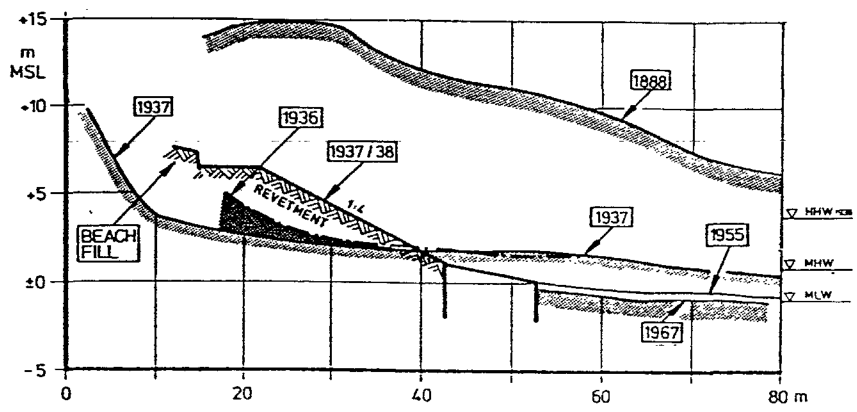


Figure Fig.2.24. Dune Erosion North of the Westerland Sea Wall, Dette (1987).

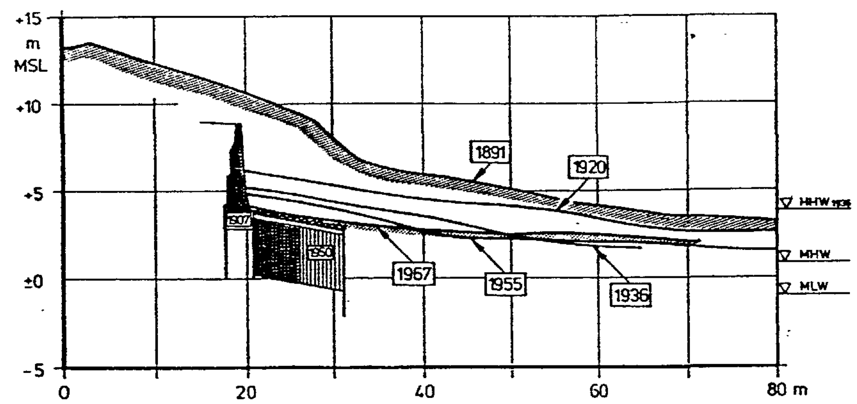


Figure 2.25. Dune Erosion in the Vicinity of Westerland Sea Wall.

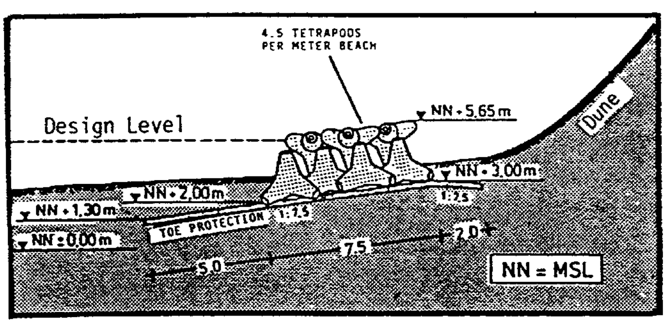


Figure 2.26. Design of Tetrapod Mound at Westerland.

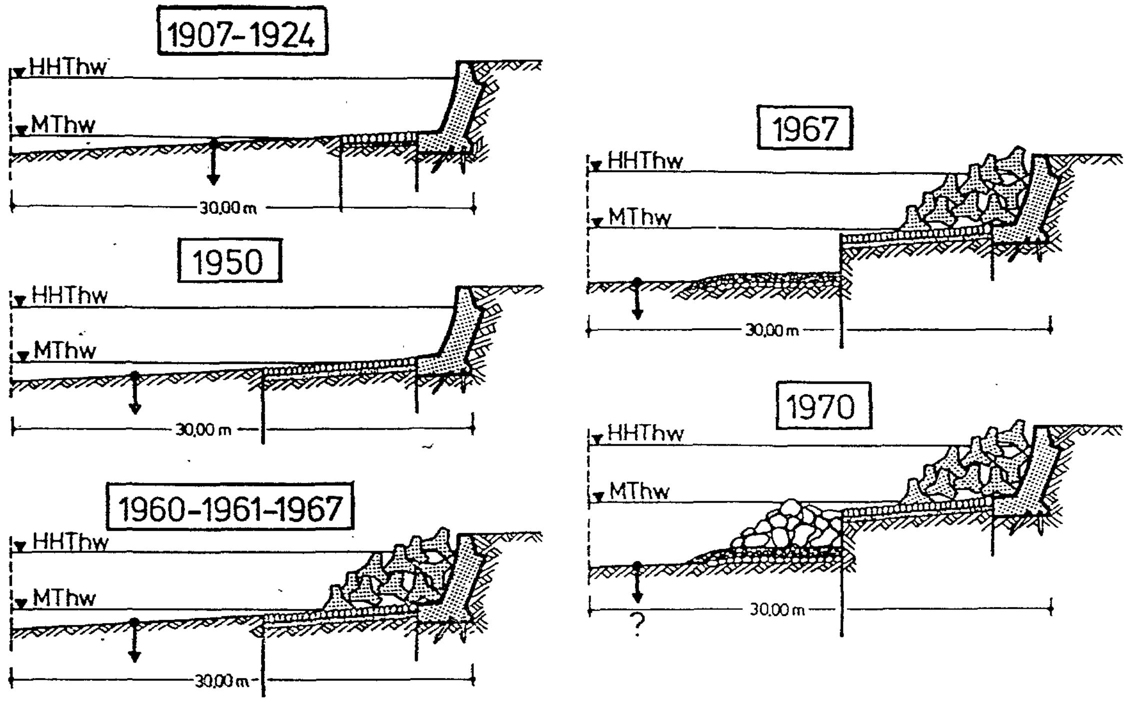


Figure 2.27. History of Westerland Sea Wall Since 1907.

However, the tetrapods placed in front of the seawall did not help to break decisively the strength of incoming waves and prevent further beach recession. This demonstrates a typical example showing the "armouring" of a sandy coastline and the futile efforts against the erosion of the beach in front of a structure.

After this condition had occured a final attempt was undertaken and in 1970 heavy riprap stones were placed in front of the toe protection. This is shown on Fig.2.27 together with a grouping of all man-made measures since 1907 in order to guarantee the stability of the seawall against heavy storm surges. Besides, this drawing illustrates the time history of a seawall over nearly 70 years. In 1972 the necessity of permanent maintenance works on the coastal structures protecting the city of Westerland as the only feasible method was abandoned and for the first time an additional beach-deposit nourishment was carried out in order to make up for the sand deficit at the structures and, besides, to shift the surf zone seaward away from the structures. Meanwhile until 1987 two more beach nourishments (1978 and 1984) were carried out.

Dette (1987) concludes that

- The characteristic topography of the foreshore consisting of longshore bars and troughs has not been "destroyed" due to the presence of coastal structures by which the long-term process of natural recession was stopped abruptly. The longshore bar and the trough still existed in 1967 (and in 1987, too), more than 60 years after first seawall was built

- The seawall in front of Westerland and the necessity of continuous elongation of the coastal structures due to unavoidable downdrift erosion and the continuing futile efforts against the beach erosion in front of the structures, consisting in the construction of "generations" of toe protection may be considered a typical and classical example of **armouring** (German *"Verfelsung"*) of the coastline at the expense of beach.

Shore evolution at sea walls can be predicted with a satisfactory accuracy only in a few cases under strictly observed conditions, Hanson and Krauss (1985). Most forecast methods are based on the one-line theory, mentioned above for other types of structures and summarised in Section 2.7. These methods can be applied if the boundary conditions during the storm surge are known (wave height; wave period; maximum surge level; storm duration; practicle size diameter; initial profile),cf. Sec.2.7. Many of these boundary conditions have a highly stochastic nature. Among the restricting boundary conditions one should require that the shoreline at a sea wall must not retreat landwards behind the sea wall frontline. The general forecast procedures developed for detached breakwaters may also prove helpful.

The maximum shoreline retreat at the very proximity of a sea wall, in the form of an erosional embayment is assessed as 10% of the sea wall length in the landward direction; the bay stretches downdrift to about 70% of the sea wall length.

These and other details may reappear in Chapter Three as the far- and near-field phenomena combine and intertwine in the vicinity of sea walls.

2.5 OTHER DEFENCE & MANAGEMENT MEASURES

2.5.1 *Perched Beach*

This section deals with some less conventional, and more recent designs and concepts of coastal defence and management. They include *inter alia* perched beach, dykes and artificial beach nourishment.

Perched beach combines a low breakwater, or sill, and a beach fill perched, or elevated above the normal level (Fig.2.28). This alternative provides a broad buffer against wave action while offering a potentially recreational site. The sill can be constructed of various materials, but it must be impermeable to the passage of the retained beach sand by using, for intance, a filter cloth behind and beneath the structure. The cloth prevents the fill from escaping through any large voids in the sill and also stabilizes the structure against settlement. While a graded stone core could also be used in a rock sill in place of filter cloth, the limited height of such sills generally precludes use of multilayered structures of this kind. The figure also shows a splash apron which is provided to prevent scour and erosion of the beach fill from overtopping waves.

Perched beaches can be provided where offshore slopes are mild enough to permit

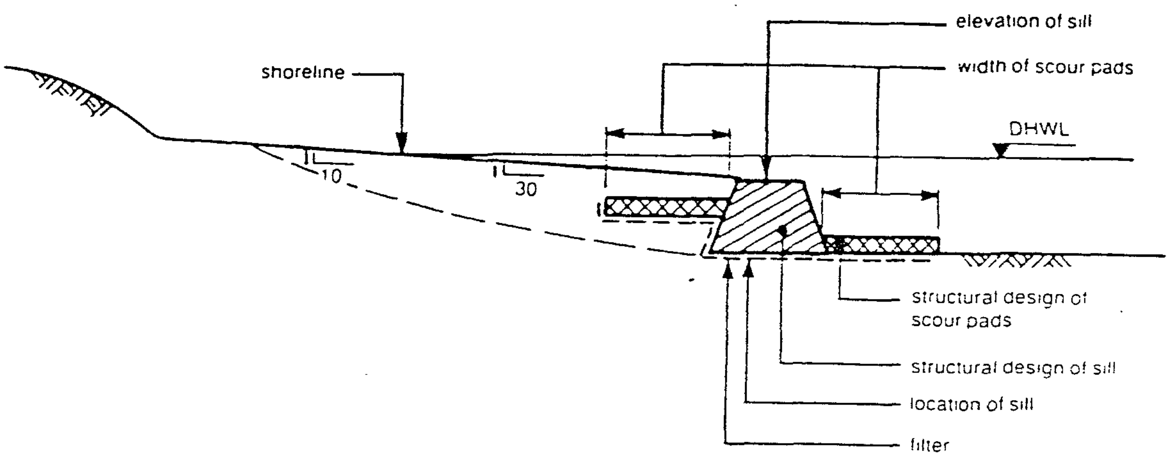


Figure 2.28. Perched Beach Design.

the use of a sill in shallow water at a reasonable distance from shore.

In addition to perched beaches, **fills** can also be incorporated in groin systems and with breakwaters. In fact, auxiliary fills are almost mandatory in most cases, otherwise serious erosion problems can occur downdrift.

Beach fills are discussed in this section and are referred to in Section 2.6.

2.5.2 *Sea Dykes*

The history of the protection of sandy coasts is relatively short compared with that of dyke construction. About a thousand years ago people in the north-western Europe had already started to construct primitive dykes, initially to gain or protect valuable farm land or small communities. However it was not until the 19th century that the first examples of protection of sandy coasts in the same area were recorded. Initially, closed or open timber works were used, but because of the poor durability of the wood these were soon followed by more solid structures.

Dykes play many roles but their primary destination is **flood prevention**. Accordingly, they are **not** expected to produce any particular, controllable far-field effects, in the sense of shaping the coast morphology. Therefore we will not dwell on them in this chapter despite their paramount importance in preservation of the coastal environment. It seems more appropriate to focus attention on dykes in the context of Chapter 3, where different failure modes can pose challenge to dykes in their near field. A common constituent of dyke design, the revetment, is touched upon in the following section.

Layout of dyke depends primarily on the configuration of the area protected but also on the bearing capacity of soil foundation, etc. For a complex shape of dyke, in plan view, the angles and intensity of wave attack can be fairly diversified, and the far- and near-field effects will vary accordingly, cf. Pilarczyk (1987) and Pilarczyk (1990).

2.5.3 *Artificial Beach Nourishment*

Artificial beach nourishment is a coastal defence scheme in which sediment is borrowed from a certain area and is transported to a beach to be filled with this sediment in order to protect the coastline from erosion, to widen the beach or transform it with any other intention.

With developments in dredging technology, beach nourishment and beach suppletion have become an alternative type of protection works. However, the first beach suppletion works, using a dredging spread, were not carried out until the 1950s.

In fact, beach suppletion only compensates for a deficit in the natural supply of sand at a certain section of the coast. This deficit results from either natural causes or human interference.

Beach suppletion, used as a beach protection measure, has gained much support owing to its cost-effectiveness and flexibility. Since the work is done at intervals, the actual planning can always be turned to previous experience gained. This latter aspect is extremely important, since in spite of progress in our basic understanding of the morphological phenomena involved, coastal engineering still has elements of an art rather than a science.

The following major conclusions may be drawn from our' review within this study.

Artificial nourishment is undertaken as a coastal protection measure and/or as a measure to enlarge the recreational posibilities of the beach. The decision to apply artificial nourishment instead of rigid protective measures (such as groynes, seawalls, shoreline revetments etc.), is generally made because of the following major advantages of beach nourishment: environmental friendliness, flexibility and economy.

Unlike rigid structures, artificial beach fill does not interfere with the natural character of a coast. From the recreational point of wiev, an artificial fill is particularly attractive, because the widened beach has a greater touristic potential. During the operation of the nourishment, the temporary limitation in the use of the beach is generally insignificant for proprietors and managers of beach facilities.

Very often the performance or side effects of rigid protection measures are not acceptable or predictable. The advantage of an artificial fill is that possible adverse effects are temporary. Increasing erosion along adjacent (downdrift) coastlines, as often occurs with rigid structures, will generally be less accentuated. On the contrary, the adjacent coast might benefit from nourishment as well.

An artificial beach fill is usually cheaper than a rigid protection structure, although the maintenance cost will be higher as the lifetime of a fill is shorter. A coastal zone which is in dynamic equilibrium prior to the nourishment will generally not tend to erode systematically after placement of the fill. However, profile adaptation directly after placement and deviating sand characteristics do affect the performance of the filled beach. One must not expect a beach fill to be stable when placed on a continuously eroding coast. The rate of retreat might be influenced by the design of the fill. An integral design method for artificial beach nourishment, including all relevant parameters, is not yet available. However, for separate aspects of the design of the beach nourishment, various design tools are available, such as models for the prediction of the morphological bahaviour of the fill and methods for the determination of the effect of different grain size characteristics. With some transportation methods, having losses of the fines size classes, and consequent coarsening of the borrow material can be significant. As a result the performance of the fill is sometimes better than predicted. The development of models which quantify the modification of material properties during the process between dredging and dumping is recommended. A review of the available data still indicates the lack of adequate monitoring before, during and after execution of nourishment schemes. Moreover, there is an evident lack of standardization in monitoring.

The demand for offshore borrow areas grows. The locations of these borrow areas should be selected such that negative influences on the beach profile are avoided and the transportation distance is minimized. More research on this subject is needed. If possible and economically attractive, material from capital and maintenance dredging should be used for beach nourishment. However, the sediment may contain an excess of fines and, especially in the case of maintenance dredging, it may be polluted. Therefore, close examination of the dredged material is a necessity, in order to determine whether it is suitable for beach nourishment.

From a biological point of view, the best time for beach nourishment and dredging is during the winter, because the spawning season for most nearshore and beach fauna occurs between spring and autumm, so that larval recuitment is not severely affected. It should be noted that this may be in conflict with the feasibility and/or efficiency of execution.

Future studies should include such objectives as optimum location of borrow areas, improvements in mathematical modelling of shore transformation, sediment losses during transportation, grain-size factors, and system analysis of all complex phenomena intervening in artificial beach nourishment schemes.

In addition to the morphological models and empirical rules, coastline computations are still carried out according to the Pelnard-Considere model. Thanks to modern computer applications, coastline development can now be computed in great detail, both in time and place, provided the basic sand-transport characteristics are known.

Frequently, the phenomena involved are so complex and so random in nature that clear-cut designs are impossible and some trial and error is indispensable in the formulation of an economic design. Although it is difficult to place a financial value on the natural visual appearance and recreational benefits obtained from using beach suppletion rather than those associated with fixed structures, these are still important aspects.

Apart from flexibility and environmental considerations, beach nourishment may be very cost-effective by comparison with "fixed" structures. For an "average" situation, beach nourishment appears to be most economic if the sediment deficit is of the order of 500,000 m^3/yr or less and the length of beach to protected 5 km or more. The cost may be reduced by a factor of two or three for lower deficits or longer beaches.

For more information on nourishment the Reader is addressed to Manual on Artificial Beach Nourishment (CUR 1987).

2.6 MORE ON STRUCTURES & COASTAL SEDIMENTATION

2.6.1 *General*

Two main causes can control a gradual erosion of a part of a coast:

- Continuous offshore sediment transport from the upper parts of the beach profile (beach and dunes) to the deeper parts of the profile
- Gradient in the longshore sediment transport.

The following analysis is extracted from Graaff and Bijker (1988)

Case (a): Continuous offshore transport

Cross-shore directed sediment transports always take place in an actual cross-section of a beach profile. Depending on the wave conditions, water level and the shape of the initial profile, onshore or offshore transports take place through an arbitrary vertical cross-section. Seen over a relatively long period (from year to year for example) the shape of the beach profile often doesn't change so much (dynamic equilibrium). In a constant situation with respect to the boundary conditions (constant mean sea level and constant yearly wave climate), there is in usual cases no reason that a continuous offshore directed transport will be the reason for a gradual erosion of the coast. If a continuous offshore transport from the upper parts of the beach profile to the lower parts of the profile occurred, the beach profile would be flatter with time. Next it can be argued that the offshore transport rates will then slow down till an equilibrium has been reached.

It is only if the supplied material to the deeper part of the profile is removed again (e.g. by a gradient in the longshore transport in that region) that a continuous cross-shore transport might be maintained. The real reason for the erosion of the coastline is then, however, not the offshore transport, but the mentioned gradient in the longshore transport (case b) in the deeper parts of the beach profile. The

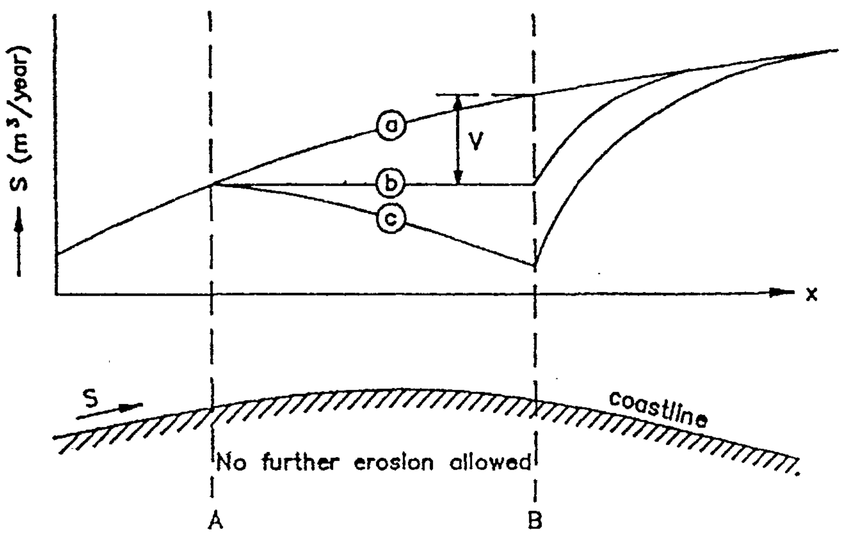


Figure 2.29. Variation of Sediment Transport along Coast.

offshore sediment transport is only an intermediary mode of transport.

If the boundary conditions change with time, a more or less continuous offshore sediment transport might occur. This transport can be seen as a prerequisite necessary to adjust the shape of the beach profile so that it fits again the 'new' boundary conditions. The global sea-level rise and an abrupt change in the wave climate are examples of changes in the boundary conditions.

Hence it is clear that if changes in the boundary conditions occur, a simple seawall cannot solve the induced erosion problem. Under usual conditions the seawall does not interfere in the underlying transport processes. (Only if the seawall is directly attacked, a certain interference might occur.) The erosion of the upper parts of the beach will continue. The seawall will be attacked more frequently and more intensively.

If one intends to control actually the (offshore) sediment transport one has to think of (submerged) offshore breakwaters. Such a measure might eventually affect (reduce) the action of waves as they approach the coast and consequently might reduce the offshore sediment transport.

Case (b): Gradient in the longshore sediment transport

In many cases a gradient in the longshore sediment transport is the main reason of the erosion problems of sandy coasts. Examples are the lee-side erosion near breakwaters or jetties and the erosion of convex coastlines.

Fig.2.29 shows schematically the magnitude of the longshore sediment transport (e.g. in m^3/year) as a function of the position along an eroding coast. The increasing sediment transport with x is the reason of the erosion problem. The magnitude of the gradient dS/dx is a measure of the actual erosion problem. If one wants to stop the recession of the coast on the stretch A-B in Fig.2.29, one has to nourish volume V (see Fig.2.29) along the section A-B on a yearly basis or one has to 'change' in one way or another the sediment transport along this stretch A-B. Instead of a distribution of the sediment transport according to line (a) in Fig.2.29, a distribution according to line (b) would in principle fulfill our requirements. The erosion in secton A-B has stopped indeed (dS/dx = 0), but downstream of the stretch A-B an increased erosion can be expexted. (Increased dS/dx values in comparison with the initial situation.).

Reaching the line (b) in reality is not a simple task. This is by no means reached with a seawall. The basic problem was a gradient in the total longshore sediment transport, the latter taking usually place on the foreshore and the shoreface. Since a seawall does not interfere with this type of sediment transport, no direct reducing effect can be expected. It is only under storm conditions, when waves hit the shore- -parallel structure directly, that probably a certain reduction of the longshore sediment transports might be expected. In that case, however, it is also conceivable that, due to wave reflection, an increase of the longshore sediment transport will occur.

Since a shore-parallel construction near the dunes does not intervene generally in the sediment transport, the erosion continues, the shoreface, the foreshore and the beaches become deeper and at the end the attack of the sea on the seawall will intensify. Damage occurs; reinforcements of the seawall will be necessary. Nearly all seawalls built to restrict the further erosion on the lee-side of harbour breakwaters or jetties suffer from the stated problems.

Hence it appears it is impossible with a seawall to move the sediment transport line (a) in Fig.2.29 to line (b). With the construction of a groyne (cross-shore structure) or a (submerged) detached breakwater (shore-parallel structure), the desired change from line (a) to line (b) is possible indeed. How to design these defence measures actually, is another, quite difficult, coastal engineering problem, but the above physical explanation can help understand the background of erosion problems.

Fig.2.30 illustrates the difficulty. Case I (line a) shows the same coastal erosion as Case II (line c). The same gradients dS/dx occur, but yet the magnitudes of the longshore sediment transport differ considerably. The lines (b) and (d) represent the ultimately desired sediment transport lines for Case I and Case II respectively. It is beyond doubt that the operation to achieve line (b) from line (a) calls for quite different countermeasures than to reach line (d) from line (c). One has consequently to know quite precisely what the actual magnitudes of the sediment transports are, before a proper design of countermeasures can be arranged.

Changing the sediment transports along stretch A-B in Fig.2.29 from line (a) to line (b) is just necessary and sufficient to stop the erosion in that section. If line (a)

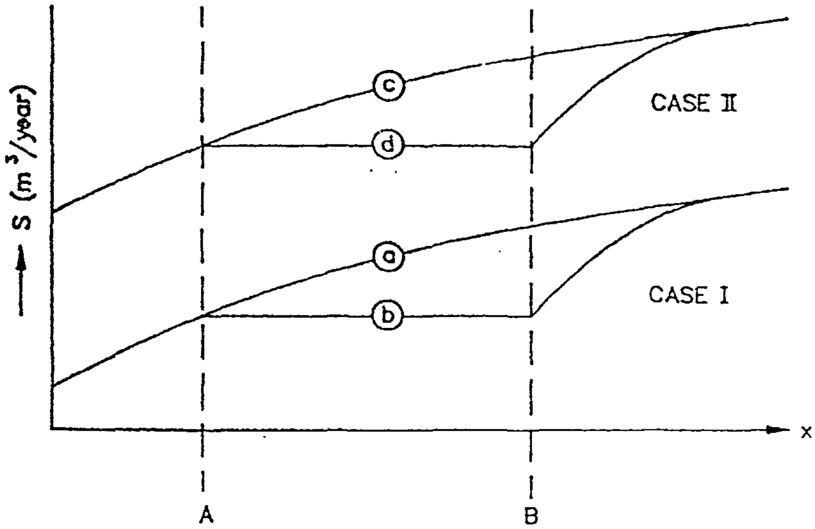


Figure 2.30. Effect of Sediment Transport Rate.

transforms into line (c), accretion of the section A-B can be expected. The erosion problems on the lee-side of section A-B will, however, consequently increase.

Since so many examples of adverse behaviour of seawalls are available, one might wonder if seawalls enhance erosion problems on the coasts they intend to protect. The physical explanation given above provides no argument for this thesis.

Seawalls do not affect sediment transport, hence they will not aggrevate the erosion problems. This holds at least till some sediment still exists in front of the seawall under usual conditions. If the beach disappears (the case in Fig.2.17 c), a quite different situation is reached. Increased as well as decreased sediment transport is possible, depending on actual external boundary conditions.

2.6.2 *Example of Reef Type Breakwaters vs. Other Measures and their Coastal Effects*

The function of reef type breakwaters as a shoreline stabilization measure and the response of the shoreline to these structures are discussed by Fulford (1985). It appears useful to examine these findings as they help understand the primary morphological functions of coastal measures.

The application and effectiveness of reef type breakwaters for shorelines typical of the Chesapeake Bay are compared to conventional shoreline stabilization measures. The functional design and typical construction costs of reef type breakwaters are discussed. From these discussions it is concluded that reef type breakwaters are a viable solution to a variety of shore erosion problems.

Introduction

Reef type breakwater refers to a low-crested rubble mound breakwater located parallel to the shoreline and constructed without the traditional multilayer cross section. This type of breakwater cross section is basiclly composed of homogeneous stones with individual stone weights sufficient to resist wave attack.

As discussed by Ahrens (1984), a number of low-crested breakwaters have been constructed in recent years to protect beaches or shoreline areas. The percentage of wave attenuation required to provide the desired degree of beach or shoreline protection is primarily a function of the crest height of the breakwater. Since the cost of a rubble mound breakwater increasses rapidly with the height of the crest, the cost differential between a low-crested structure and a traditional breakwater that is infrequently overtopped can be significant.

Function of Reef Breakwater

A shore-parallel breakwater constructed offshore provides protection by reducing the amount of wave energy reaching the water and shore area in its lee. The breakwater structure reflects or dissipates the incident waves impacting directly on the structure and transmits wave energy primarily by means of diffraction into the barrier's shadow zone.

Shoreline Type 1

Fig.2.31(top) shows the modification and typical response of a shoreline consisting of a low bank fronted by a narrow beach as a result of construction of a reef type breakwater and other stabilization measures. These responses are discussed in the following paragraphs.

Effect of Reef Breakwater

As shown in Fig.2.31(top) a, the typical response of the shoreline is the formation of a beach platform on the shoreward side of the breakwater. The formation of the beach planform is a result of the diffracted waves acting on the sediment in the lee of the breakwater placed during the construction of the breakwater, the sediment trapped from the longshore transport and the sediment derived from reduced erosion of the bank. As the planform evolves to become relatively parallel with the diffracted wave crests, longshore transport losses are significantly reduced. In addition, the presence of the breakwater significantly reduces the loss of sediment in the offshore direction from the newly formed beach and the shoreline area. This is particularly important in areas where the shoreline sediment is fine-grained and subject to offshore transport. The resulting planform becomes a stable feature and acts as a protective beach area, which in combination with the protection provided by the breakwater, effectively stabilizes the shoreline area.

The low-crested design of the reef type breakwater results in periodic overtopping of the structure by incident waves. Wave overtopping limits the growth of the beach planform towards the breakwater. This effect is desirable in that the planform remains unattached to the brakwater. Thus, the interruption to the longshore transport system which could adversely effect downdrift shoreline is minimized.

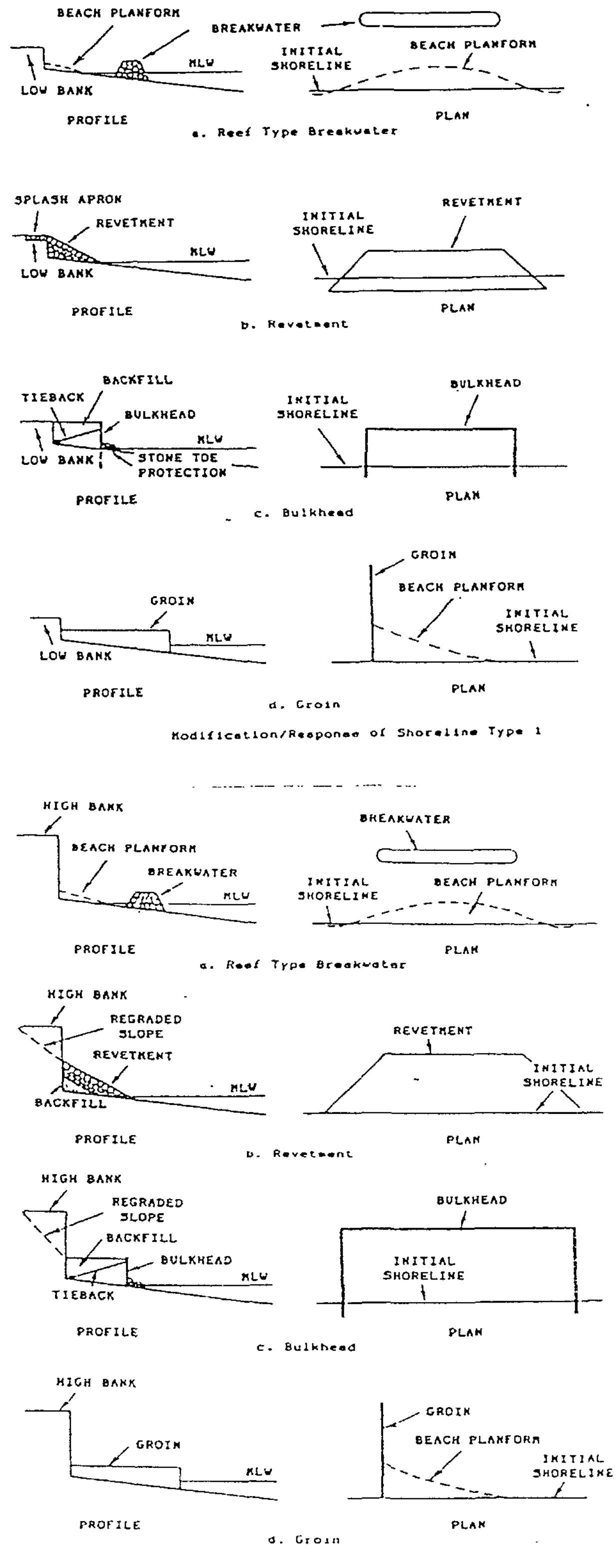


Figure 2.31. Morphological Interaction of Coast and Structures, shoreline types 3 and 4, Fulford (1985).

The ability of a reef type breakwater to dissipate incident wave energy and diffract waves into the lee of the structure results in the formation of a stable beach platform as discussed. This ability results in the suitability of reef breakwater construction in areas which are essentially sediment starved. In these areas, the source of sediment for the beach platform can be provided as part of the construction process or through continued erosion of the shoreline bank area (at a reduced rate) as a result of overtopping wave events. The reduced wave energy in the lee of the breakwater would allow the formation on the beach planform with fine-grained sediment. Following formation of the protective beach planform, the erosion of the bank area would be stopped.

Effect of Stone Revetment

Fig.2.31(top) b shows the modification and response of the low bank shoreline following the construction of a stone revetment. Essentially, the revetment stabilizes the shoreline by dissipating the wave energy before it reaches the shoreline. Since the dissipation of the wave energy occurs on the revetment slope which is in close proximity to the shoreline, the revetment slope, crest height and crest width must be sufficient to prevent significant wave runup on the structure which could erode the shoreline. A splash apron or shoreward extension of the crest of the revetment may be required to prevent this erosion. Shoreward extension of the ends of the revetment is recommended to prevent flanking of the structure. Incorporation of the recommended layers of bedding, underlayer and armour stone in revetments along shoreline areas with bank heights less than 3-4 feet may be difficult to achieve as well as the desired degree of runup reduction.

Construction of a properly designed revetment results in a positive stabilization of the shoreline area. Due to the construction of the revetment, access to the beach area is limited. Turbulence during the wave dissipation process and wave run down on the structure tends to reduce the beach area in front of the structure.

Effect of Bulkhead

Fig.2.31(top) c depicts the response of the shoreline following the construction of a vertical faced bulkhead. Typically, the bulkhead is located a distance bayward of the eroding shoreline, anchored shoreward and backfilled . Similar to a revetment, the function of a bulkhead is the dissipation of wave energy on the structure before it reaches the shoreline area. The crest height of the bulkhead must be sufficient to prevent wave overtopping which could result in erosion of the backfilled area as well as the original shoreline area. A splash apron shoreward of the bulkhead is an alternative to prevent shoreward erosion by overtopping. The incorporation of stone toe protection to prevent scour at the toe of the bulkhead is highly recommended as part of all bulkhead construction. This toe scour is a result of the downward deflection of wave energy incident on the structure and could lead to undermining of the structure. Extension of the bulkhead shoreward at each and or connection to adjoining structures is required to prevent flanking of the structure.

Construction of a properly designed bulkhead along a shoreline area results in positive protection of the shoreline. However, due to the deflection of wave energy on the structure, beach area in front of the bulkhead at the time of construction is generally lost. If the degree of wave and scour action significantly deepens the area offshore of the bulkhead, this area can become a sink for sediment from adjacent shorelines. The result of this action can be accelerated erosion of these areas as sediment from these areas begins to fill the sink area in a effort to regain an equilibrium profile.

Effect of Groynes

Fig.2.31(top) d shows the modification of the shoreline as a result of the construction of a groin. In an area where there is a predominant direction of longshore sediment transport, sediment is trapped on the updrift side of the groin cresting a beach planform. Until this beach planform reaches an equilibrium size, at which time the longshore transport resumes part the groin, downdrift erosion is likely. Filling the updrift side of the groin with sand as part of the initial construction can significantly reduce downdrift erosion effects. In areas subject to longshore sediment transport in both directions, downdrift erosion effects are not as likely to result as the groin entraps sediment on both sides.

Construction of a groin along a shoreline and the resulting formation of a beach planform provides a buffer between the incoming waves and the original shoreline area. As a result, protection is provided during moderate weather conditions. However, only limited protection is provided against storm- driven waves. Storm waves characterized by short periods and relatively high steepness (height to length ratio) can still directly attack the shoreline area. These waves are conducive to offshore sediment transport and tend to erode sediment from the shoreline area and the beach planform and transport it to offshore areas. In some cases the groin can train the longshore currents in the offshore direction which results in an increase in offshore sediment transport. This sediment is essentially lost from the littoral system since longer period, low steepness swell type waves conducive to onshore sediment transport are not characteristic of the Chesapeake Bay area.

Loss of a significant portion of the updrift beach planform further reduces the protection provided to the shoreline area. If the formation of the beach planform is dependent on the trapping of sediment from the longshore transport system, reduced protection of the shoreline area will exist until the planform is reformed. Downdrift erosion may also be experienced until the longshore transport of sediment past the groin resumes. These effects may be evoided by artificially filling the updrift side of the groin.

Shoreline Type 2

Fig.2.31(bottom) compares the modification and response of a shoreline area consisting of a high bank fronted by a narrow beach as a result of the construction of a reef type breakwater and alternative stabilization measures.

Effect of Reef Breakwater

Fig.2.31(bottom) a shows the response as a result of the construction of a reef type breakwater. The overall shoreline response is the same as for the low bank shoreline. The primary difference in this situation is the response of the high bank versus the low bank. Reduced wave energy in the lee of the breakwater will result in the formation of the beach planform from sediment derived from the longshore transport system, erosion of the bank and/or sandfill during construction. Depending on the wave energy reduction by the breakwater (a function of the breakwater design), erosion of the bank will continue at a reduced rate during wave overtopping events. Sediment derived from this erosion will aid in the formation of the platform. Due to the reduced wave energy in the lee of the breakwater, eroded fine-grained sediment that would normally be transported from the area in the longshore and offshore directions would tend to remain. This is an important feature since a significant number of high banks are composed of primairly fine-grained sediment.

As the beach planform reaches an equilibrium state, the protection provided by the planform in combination with the breakwater would significantly reduce the erosion of the bank.

Stone toe protection could be placed at the toe of the bank as added protection. The addition of drainage control to increase the stability of the bank material would increase the overall stability of the shoreline area.

Effect of Stone Revetment

Fig.2.31(bottom) b shows the modification and the response of the shoreline as a result of the construction of a stone revetment. In this situation, significant regrading of the bank to a more stable slope and the plunging of soil stabilizing vegetation would be necessary. Without this regrading, continued erosion of the bank as a result of wave runup on the revetment could lead to failure of the bank and possible destruction of the revetment. To reduce this possibility, the crest height, crest width and slope of the revetment would have to be designed to prevent wave runup from reaching the toe of the bank. Failure of the bank due to instability of the bank material would also threaten the stability of the revetment.

Construction of a revetment in combination with slope regrading, vegetation and drainage control would result in a stable shoreline area. However, the acceptability of regrading of the bank would have to be evaluated in view of the structures along the bank and the desires of the property owner. Shoreward extension of the ends of the revetment would be necessary to prevent flanking.

Effect of Bulkhead

Fig.2.31(bottom) c shows the response of the shoreline as a result of the construction of a bulkhead. Similar to bulkhead construction along a low bank shoreline, the bulkhead is located a distance bayward of the shoreline to allow for shoreward anchoring of the bulkhead and backfilling. Along a high bank shoreline, this distance in combination with the height of bulkhead above the water level must be sufficient to prevent waves from overtopping the bulkhead and eroding the toe of the bank. Erosion of the backfilled area by overtopping waves must also be considered in the selection of the design height of the bulkhead. A sufficient bayward distance is also required to prevent the destruction of the bulkhead if failure of the bank occurs as a result of instability of the bank material. Shoreward extension of the ends of the bulkhead to prevent flanking is also required.

Regrading of the bank to a more stable slope and vegetative planting would allow the bayward distance of the bulkhead anchoring of the bulkhead such as batter piles on the bayside would be possible (assuming regrading of the slope is accomplished) which would also reduce the bayward construction distance of the bulkhead. Construction of a bulkhead in combination with regrading of the bank and vegetative planting would stabilize the shoreline. However, the proximity of structures along the bank and the desires of the property owner must be considered when evaluating bank regrading. Without bank regrading, the required bayward location for bulkhead may require significant backfilling of the nearshore area as well as a shoreline protuberance that could interfere with longshore sediment transport.

Effect of Groyne

Fig.2.31(bottom) d shows the results of the construction of a groin. Due to the lack of protection of the toe of the bank area from storm waves, and the potential loss of the updrift beach planform during storm events, groin construction alone is not considered to be a viable solution.

Shoreline Type 3

Fig.2.32 compares the modification and response of a shoreline consisting of a narrow beach as a result of the construction of a reef type breakwater and alternative stabilization measures.

Effect of Reef Breakwater

Fig.2.32(top) a shows the modification and response of the shoreline resulting from the construction of a reef type breakwater. Similar to the other shoreline types, the typical response is the formation of a beach planform in the lee of the breakwater. This planform is formed from sediment from the longshore transport system, erosion of the shoreline and/or sand placed as part of the initial construction.

The construction of the reef type breakwater and the formation of the beach planform provides positive protection for the shoreline area without changing the nature of the area. Following construction, increased beach area is available for recreational use.

Effect of Revetment

As shown in Fig.2.32(top) b, the lack of a bank section along this type of shoreline

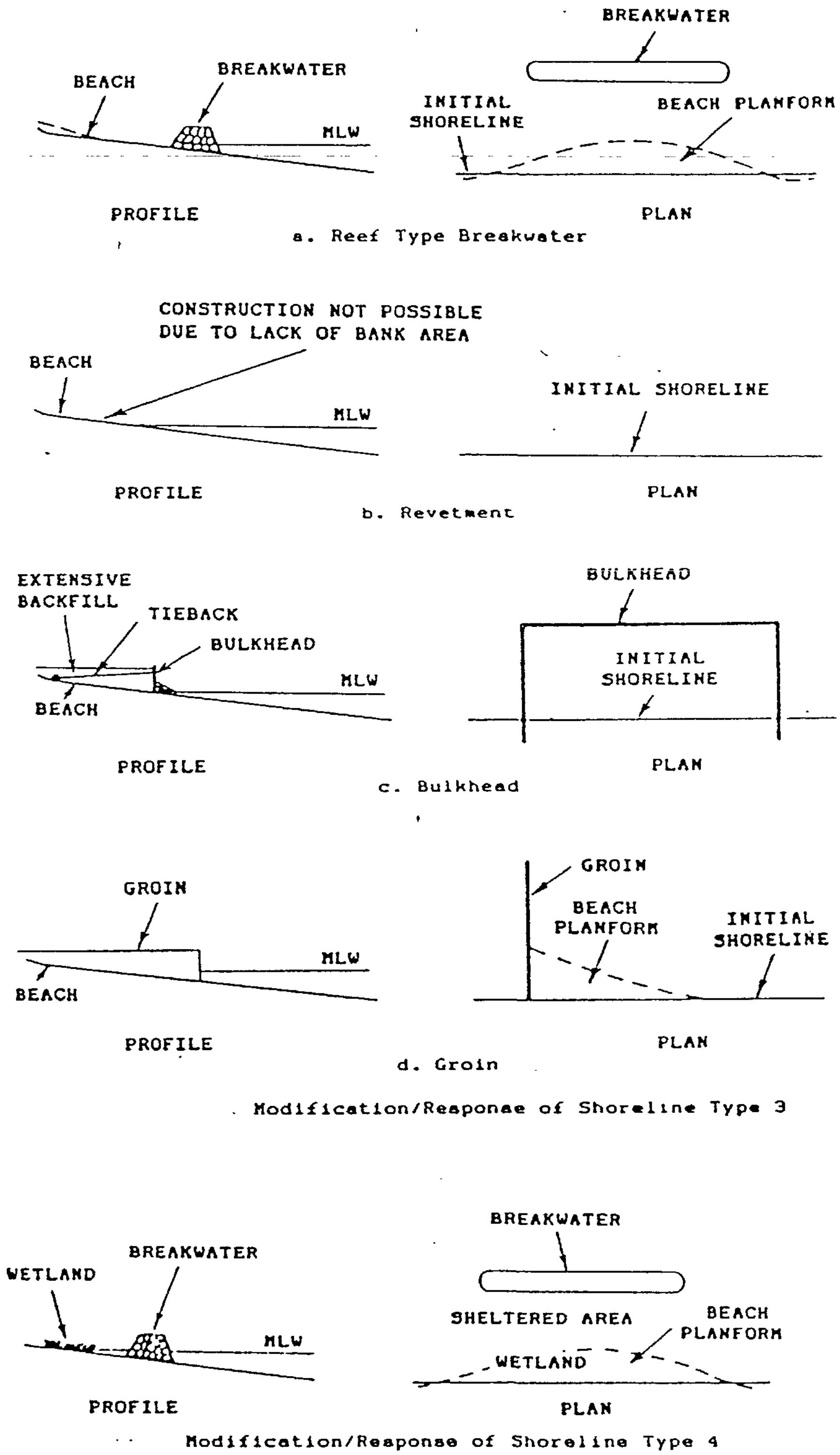


Figure 2.32. Morphological Interaction of Coast and Structures, Fulford (1985).

to support the sloped revetment makes revetment construction impossible.

Effect of Bulkhead

Construction of a bulkhead along a shoreline without a nearshore bank section is not practical. As shown in Fig.2.32(top) c, extensive backfilling of the area shoreward of the bulkhead would be required to achieve the required stability of the structure.

Effect of Groyne

Fig.2.32(top) d shows the modification and response of the shoreline as a result of the construction of a groin. Sand trapped on the updrift side of the groin and/or placed during the initial construction would form a beach planform as shown. As long as this planform remains, some protection is provided to the shoreline area. However, as discussed previously, storm- driven waves would still be able to attack this area and would tend to erode and transport sand from the shoreline area and the planform into the offshore area.

Shoreline Type 4

Reef Breakwater

Fig.2.32(bottom) shows the modification and response of the shoreline as a result of the construction of a reef type breakwater. Similar to the other applications, a beach planform would be expected to form. The wave energy reduction as a result of the breakwater and the buffer zone provided by this planform would result in positive protection of the wetland area. Continued propagaton of the wetland plants would be expected. In fact, the sheltered zone in the lee of the breakwater would be suitable to the creation of additional wetland areas.

Revetment

As discussed for Shoreline Type 3, revetment construction along this type of shoreline is not possible.

Bulkhead

Bulkhead construction is not practical along this type of shoreline. Backfilling of the area shoreward of the bulkhead would eliminate the wetland area if the bulkhead is located bayward of the wetland. Locating the bulkhead shoreward of the wetland would result in the loss of the wetland as a result of reflected wave energy and scouring in front of the bulkhead.

Groyne

As a result of the lack of storm protection provided by a groin and the updrift beach planform, groin construction to protect the wetland area is not practical.

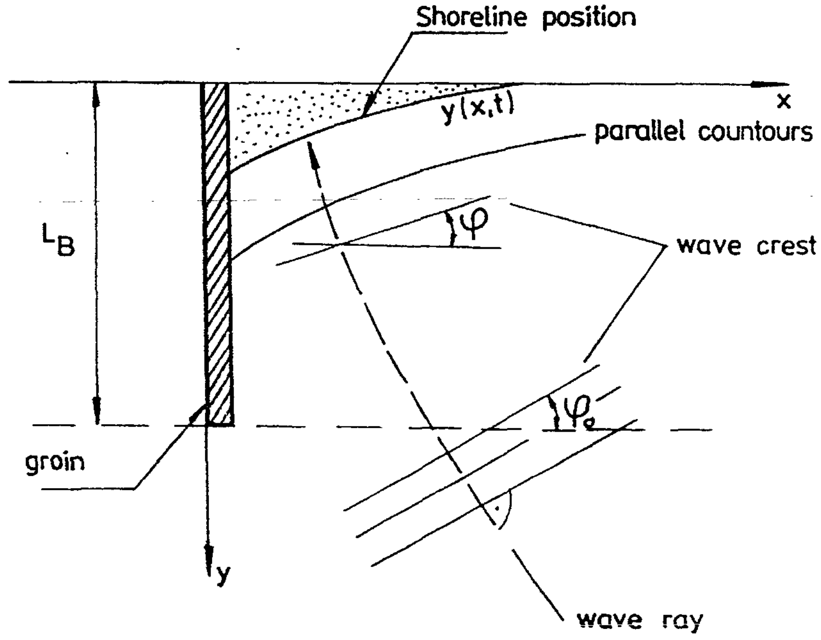


Figure 2.33. Schematic Representation of Shoreline Evolution in Pelnard-Considere's Model.

2.7 MATHEMATICAL MODELLING of SHORE TRANSFORMATION at STRUCTURES

2.7.1 *Cross-Shore Structures*

The first theoretical model to describe shoreline changes at an artificial cross-shore structure was constructed by Pelnard- -Considere (1956). This study was quite fundamental for the analytical description of the interraction between a structure and beach, and therefore has been used in most later models. In the following we examine this model.

The primary assumption in Pelnard-Considere's theory are:

- constant angle of wave incidence, not greater than $\varphi < 20^{\circ}$,
- linear relationship between longshore sediment transport Q and the angle of wave incidence, i.e. $Q = Q(\varphi)$
- mutually parallel isobaths and constant equilibrium shape of the beach profile, thus reducing the problem to the so-called one line.

The basic equation derived by Pelnard-Considere is of the diffusion type

$$K\frac{\partial^2 y}{\partial x^2} = \frac{\partial y}{\partial t} \qquad (2.\ 9)$$

in which the system of coordinates x, y is as shown in Fig.2.33. The variable y = y (x,t) is a temporal function of beach changes. The parameter K in Eq.2.9 is given by

$$K = \frac{1}{h}\frac{Q}{\varphi}\,|_{\varphi=\varphi_o} \tag{2. 10}$$

The quantity h is the deep water depth on which indipient motion of sediments takes place.

Eq.2. 10 indicates that erosion and accretion $\frac{\partial y}{\partial t}$ is linearly dependent on the shore line curvature, longshore sediment transport derivative with regard to the angle of φ, $\frac{dQ}{d\varphi}$, and inversely proportional to h.

With adequate boundary and initial conditions Pelnard-Considere's model can be used practically for every cross-shore structure (breakwater, groyne, causeway, pier, etc.). Because of the assumptions and simplifications the method must not be considered a universal tool for prediction of diversified beach profiles, but careful application of it can produce a number of valuable data for long-term beach transformations about structures.

Pelnard-Considere has applied his theory to the case of a long impermeable cross-shore structure. For appropriate boundary conditions his solution of Eq.2.9 reads:

$$y(x,t) = \frac{\tan\varphi_o}{\sqrt{\pi}}[\sqrt{4Ky}exp(-u^2) - x\sqrt{\pi}E(u)] \tag{2. 11}$$

in which $u = \frac{x}{4Kt}$

Eq.2. 11 describes shore line transformations in time t and holds true for times $t < t_1$ when the entire longshore transport is being arrested by the cross-shore structures.

The characteristic time t_1 necessary for complete filling of the area on the upward side of the structure is given by the formula

$$t_1 = \frac{L_B^2\pi}{4K\tan^2\varphi_0} \tag{2. 12}$$

in which L_B = length of cross-shore structure.

For times $t > t_1$ the up-wave side of a structure is completely filled with sediments and then a part of the sediment transported along shore will move seaward of a structure and therefore further changes of shoreline will be given by

$$y = L_B.E(u) \tag{2. 13}$$

In this original version Pelnard-Considere's method neglects a number of factors due to its simplifications.

The most important ones include negligence of wave refraction and diffraction, on-off shore sediment motion, and the assumption of constant bed slope with mutually parallel isobaths. These simplifications have given rise to many disparties between prototype and mathematical models, Le Mehaute and Soldate (1977).

A next step in mathematical modelling of shoreline transformation was done by

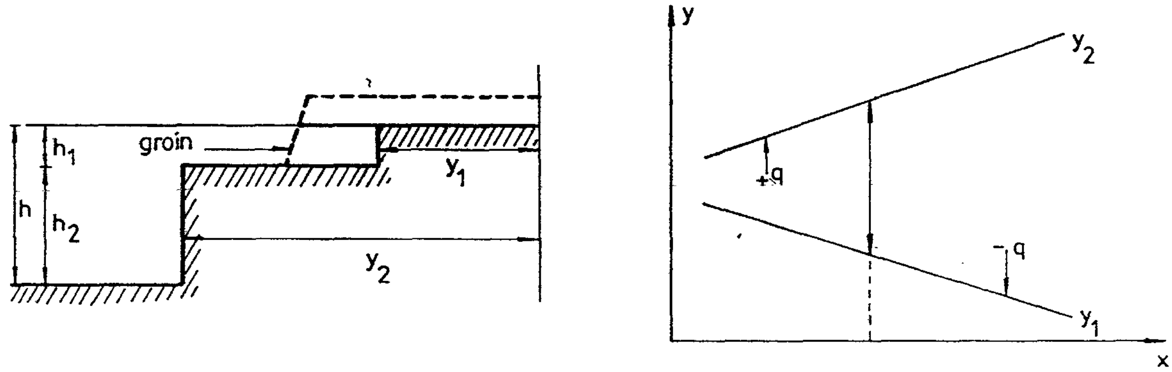


Figure 2.34. Representation of Two-Line Theory.

Bakker (1968), Bakker et al. (1970). He presented a more sophisticated model for beach changes around a single groyne or a group of structures. Having accepted Pelnard-Considere general reliationship, Bakker takes into account wave diffraction and on-offshore sediment transport linked to cross-shore bed transformation. The coastal zone in Bakker's model is represented by two lines, which in general must not be parallel. There are two parts of the beach profile given by two lines (Fig.2.34), and therefore Bakker's model is referred to as the two-line theory.

The bed transformation in Bakker's method $y_1(x,t)$ is computed as the resultant of the stationary effect of wave diffraction $y_o(x)$, and an unsteady effect $y_1'(x,t)$

$$y_1(x,t) = y_o(x) + y_1'(x,t) \tag{2. 14}$$

The quantity $y_o(x)$ to mark the steady shoreline is given by the differential euqation:

$$\frac{dy_o}{dx} = \varphi_x - \frac{\varphi_\infty}{H^2(x)} \tag{2. 15}$$

in which $\varphi_x, \varphi_\infty$ = angles of wave incidence at point x and infinitely far away from the structure, respectively;
$H(x)$ = ratio of wave height at any point x to wave height far away from structure.

The unsteady factor due to the presence of a structure and cross-shore sediment transport, $y_1(x,t)$ can be found from the equation

$$\frac{\partial y_1'}{\partial t} = \frac{\partial q_1^*}{H_1(x)\partial x}\frac{\partial^2 y_1'}{\partial x^2} + \frac{1}{H_1(x)}\frac{dq_1^*}{dx}\frac{\partial y_1'}{\partial x} - \frac{q_y}{H_1(x)}(y_1 - y_2) \tag{2. 16}$$

in which
$q_1^* = A.H_1(x)$
q_y, A, q_2 = constants.

The depth changes below the structure head y_2 are given by the differential equation

$$\frac{\partial y_2}{\partial t} = \frac{\partial q_2}{H_2(x)\partial x}\frac{\partial^2 y_2}{\partial x^2} + \frac{q_y}{H_2(x)}(y_1 - y_2) \tag{2. 17}$$

The lines $y_1(x,t)$ and $y_2(x,t)$ can be found by numerical solutions of Equations 2.16 and 2.17 with given boudary conditions. More accurate discussion of the model and its application to a system of groynes is given in a number of studies, Bakker (1968) and Bakker et al (1970).
It should be added that despite the great potentials of the two-lines theory its practical application is complicated due to the necessity of satisfying boundary conditions, which is one of the drawbacks of the model. In particular, this reservation is extended to the quantitinve analysis of pertinent phenomena. It must be noted that Bakker's theory neglects the following factors: - the effect of rip currents on the cross-shore transport at groynes
- the effect of wave diffraction on shoreline transformation on the leeside of groynes
- the effect of wave refraction on changes in sediment transport.

These effects in some cases can be of crucial importance to the accuracy of prediction.

Bakker's theory of two-lines has been elaborated in more studies, intended mostly towards verification of the model. Some results are presented by Hulsbergen, von Bochove and Bakker (1976). Among other things it is shown that Bakker's model yields an approximate description of the phenomena observed in experiments.

It is appropriate to mention a study by Perlin and Dean (1978), in which a number of computational examples are given for the action of cross-shore structures on beach. Various methods are compared (Pelnard-Considere and Bakker), along with implicit as well as explicit numerical schemes. Long impermeable cross- - shore structures were treated. Perlin and Dean have found that a multiple m-line model as an extension of Bakker's theory would be most appropriate. The m-line model would hopefully simulate bed topographies around structures in the best feasible manner. Two additional studies appeared at the end of the seventies of Le Mehaut'e and Soldate (1977, 1980). Both have been based on the Pelnard-Considere theory.

In Le Mehaute' and Soldate (1977) the quantity of sediments bypassed after $t > t_1$ seaward of the groyne head have been analysed The equation:

$$Q(t) = Q_0[1 - \frac{0.638}{(t/t_1 - 0.38)^{1/2}}] \tag{2. 18}$$

in which Q = longshore sediment transport rate for undisturbed conditions provides a tool to follow perturbations in longshore sediment transport due to the presence of a cross-shore structure.

The graphical illustration of Eq.2.18 is given in Fig.2.35. Assuming that the newly formed shoreline reaches the head of a cross-shore structure due to the presence of the latter, after time $t = 5t_1$ it is about 75% of the sediment transport Q_o

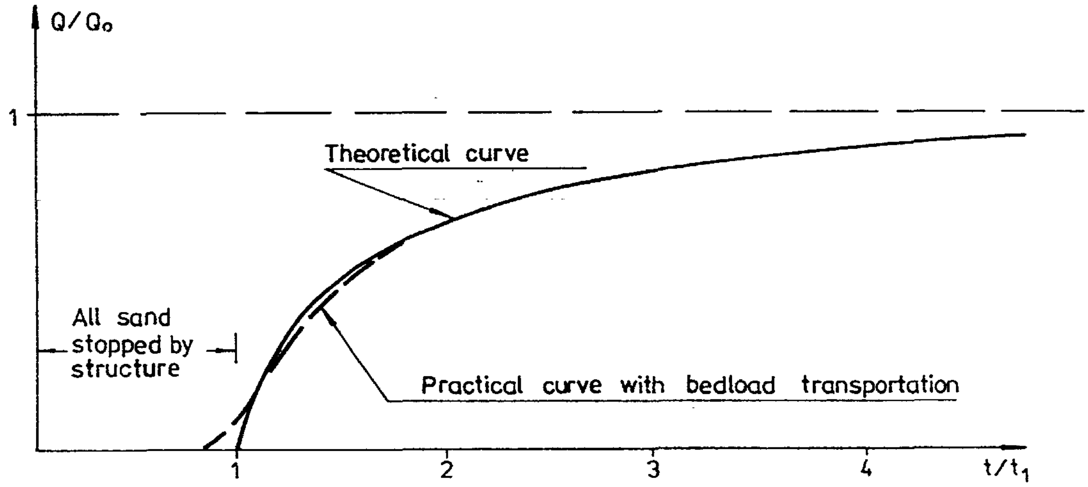


Figure 2.35. Temporal Variation of Sediment Transport at Impermeable Obstacle, Le Mehaute & Brebner (1977).

which is passed by seaward of a groyne head while the remaining part is still being entrapped updrift of the groyne. This illustrates the fact that even after a long time since a relatively stable shoreline was created there is no return to the shore configuration for natural conditions without a structure.

Results of the analyses presented by Le Mehaute' and Soldate (1977) have been utilized by Pruszak and Tarnowska (1982). Basing on their own scale model tests on the effect of a cross-shore structure on sediment transport distributions and shore evolution the latter authors have assessed the applicability of Le Mehaute and Soldate's method. Inter alia, they have found that for an impermeable cross-shore structure having a length equal to the surf zone, width it is from the very beginning ($t < t_1$) that a slight portion of sediment is moved behind the groyne head (about 5% of the sediment transport rate for undisturbed conditions). This quantity increases subsequently to about 20% of the initial value as the time approaches $t_1 (t = t_1)$. In another study by Le Mehaute and Soldate (1980) the theory by Pelnard-Considere is extended for a new single-line numerical forecast of shoreline evolution in the vicinity of a cross-shore structure. Compared with Pelnard-Considere's model the latter study includes i.a. such effects as wave refraction and diffraction, and changes in bed slope.

The basic differential equation for shoreline evolutions reads

$$\frac{\partial y}{\partial t} = \frac{B_0 + h}{B + h} F(\varphi_0)[1 + (\frac{\partial y}{\partial x})^2]^{-1} \frac{\partial^2 y}{\partial x^2} + R(x, y, t) \tag{2. 19}$$

in which

B=elevation of the active berm above sea-water level

$$F(\varphi_0) = \cos\varphi_0 - \cos\varphi_b \frac{\partial f}{\partial \varphi_0} - \sin\varphi_0 \sin\varphi_b \tag{2. 20}$$

$$R(x,y,t) = \frac{B_0+h}{B+h}F(\varphi_0)\frac{\partial\varphi}{\partial x} - (y_c - y_d)\frac{dD}{dt} + \frac{1}{2}\frac{h}{B_0+h}d\Delta y dt +$$

$$+2K_D\frac{\partial K_D}{\partial x}\cos\varphi_0\sin\varphi_b \tag{2. 21}$$

in which
$\frac{dD}{dt}$ = temporal variation of sea-water level
$f(\varphi_o) = \varphi_o(0.25 + 5.5\frac{H_o}{L_o})$
K_D = diffraction coefficient.

Precise meaning of all symbols can be found in the study by Le Mehaute' and Soldate (1980).

Eq.2. 19 in its general form is unsolvable analytically. Numerical solution can however be found for given boundary conditions although the task is rather formidable. Therefore simpler cases are usually analysed for which solutions are easier.

For a constant bed slope and elevation of berm, upon a number of additional assumptions, cf. Le Mehaute' and Soldate (1980), one may obtain the following simplified version of Eq.2. 19

$$\frac{\partial y}{\partial t} = \frac{\partial}{\partial x}K_D^2\cos\varphi_0\sin\varphi_b - \frac{1}{m}\frac{dD}{dt} \tag{2. 22}$$

Eq.2. 22 embodies both diffraction and refraction and at the same time is simple in numerical treatment. In the case of refraction effects alone the following diffusion equation holds

$$\frac{\partial y}{\partial t} = L(x,y)\frac{\partial^2 y}{\partial x^2} = (z\cos\varphi_0\cos\varphi_b - \sin\varphi_0\sin\varphi_b)\frac{1}{1+(\frac{\partial y}{\partial x})^2}\frac{\partial^2 y}{\partial x^2} \tag{2. 23}$$

in which $z = 0.25 + 5.5\frac{H_o}{L_o}$. Taking boundary conditions for a standard case of an impermeable cross-shore structure one may solve Eq.2. 23 analytically by analogy to Pelnard-Considere's solution

$$y(x,t) = 2\tan\varphi\sqrt{\frac{kt}{\pi}}exp(-\frac{x^2}{4kt} - tan\varphi E(u) \tag{2. 24}$$

or otherwise

$$y(x,t) = \cos\varphi\sin\varphi_b\left[\sqrt{\frac{2}{\pi k}}t^{1/2}exp(-\frac{-x^2}{4kt} - \frac{x}{\alpha}E(u)\right] \tag{2. 25}$$

Solutions in Eq.2. 24 and Eq.2. 25 differ from that given by Pelnard- -Considere in that the parameter K is variable.

Le Mehaute and Soldate have applied their model to Port Holland in State

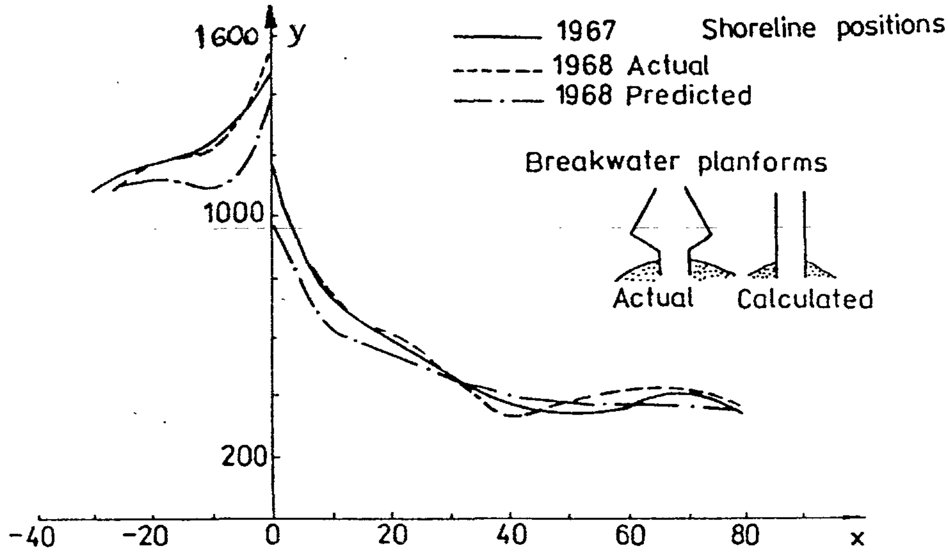


Figure 2.36. Le Mehaute and Soldate's (1980) Results.

Michigan, USA. The case considered was a system of two neighbouring breakwaters taken as normal to shoreline. Results of the computation by Eq.2. 22 with adequate boundary conditions and measurements corresponding to them are shown in Fig.2.36.
Le Mehaute and Soldate (1980) have determined that a considerable effect on shoreline evolution is due to wave refraction. It has also been found that despite its simplifications (no cross-shore transport or rip currents, and long-shore transport as a function of the variable y only) the model can be used for various cases of shore discontinuities in the normal sense.

Another example of the application of Le Mehaute and Soldate's model can be found in a study by Borah and Bolloffet (1983). Computations were carried out for long-term (about 30 years) forecast of shoreline evolution due to a reconstruction of a long quay (more than 1 km) protruding into the sea normally to the shoreline in the neighbourhood of existing piers, and thus about a whole system of structures.

A considerable number of publications have appeared in the eighties for numerical models of shore-line evolution. Almost all of them are based on the simple and well-known Pelnard-Considere's theory of single line. These studies do not introduce revolutional concepts as far as the mathematical modelling of shore-line evolution at structure is considered. They do provide however simpler or more sophisticated numerical algorithms for various situations and boundary conditions. Examples are provided in the studies by Hanson and Kraus (1980), Kraus, Hanson and Horikai (1984), Matsuoka and Ozawa (1983) or Khomitskiy (1983) and Hanson (1987).

The University of Lund has offered an interesting and relatively simple numerical model of shoreline evoltuions, including the effect of groyne permeability, Hanson and Kraus (1980). The model makes use of Pelnard Considere's one-line theory upon the assumption that the long-shore sediment transport rate is proportional to the angle of wave incidence with no currents, a small angle of wave incidence, etc.

The differential equation in this model of shore-line evolution is the diffusion type equation (Eq.2. 9) with a slightly different parameter K given in terms of the factor of proportionality k between long-shore sediment transport rate and longshore energy flux at breaking point:

$$K = \frac{2k\sqrt{gH_b/h_b}}{16h(\rho_s/\rho - 1)(1-p)}\left(\frac{H_b}{1.416}\right)^{2.5} \qquad (2.\ 26)$$

in which

p=sediment porosity.

Unlike the earlier models Hanson and Kraus' proposal includes characteristics of sediments and not only wave parameters. For simple boundary and some constant parameters of Eq.2. 26 the latter can be solved analytically. However if one seeks a better approximation to prototype, including wave diffraction, variations of bed slope and temporal variability of the pertinent factors, then an exact analytical solution must be approximated with numerical algorithms, similarly as was done by Le Mehaute and Soldate.

One of the boundary conditions applied in Hanson and Kraus' solution included the assumption on the permeability of cross-shore structures. Theoretical testing of the effect of structures having permeability varying from 0% to 60 % on shore-line evolutions is depicted in Fig.2.37.

The numerical model proposed by Hanson and Kraus is characterized by a typical sequence of computations which are also present in other studies. They include - computations and input of initial and boundary data (original shoreline, bed slope, depth, wave field, location of groyne, groyne permeability etc.)

- determination of the temporal and spacial increments
- determination of the angle, location and height of breaker
- determination of long-shore transport rate and its modification due to boundary conditions
- determination of consecutive shorelines depending on given boundary conditions.

The details pertaining to the solution of such problems can be found in the study by Hanson and Kraus (1980).

Khomitskiy (1983) and Selezov et al (1983) have presented a new method for computations of shoreline evolution at a cross-shore structure being now in use in the USSR. This is a next version of Pelnard-Considere's one-line theory, very close to the one proposed by Le Mehaute' and Soldate in 1980, Eq.2. 23.

In Khomicki's formulation (1983) the basic differential equation reads

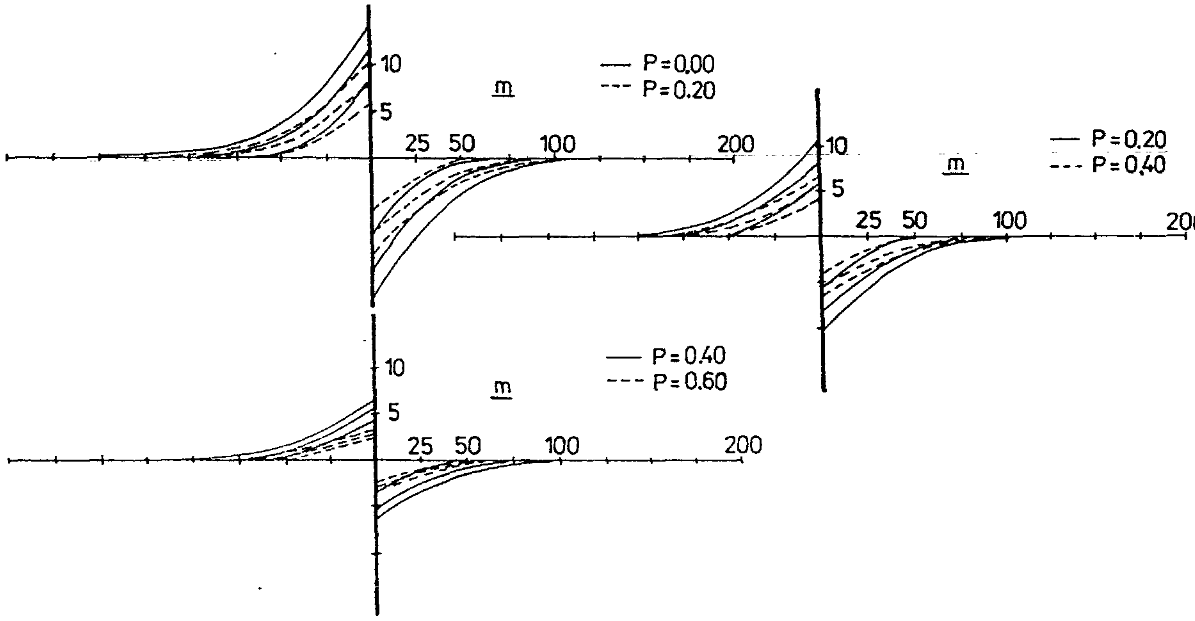


Figure 2.37. Effect of Groyne Permeability on Shoreline Changes. Computations by Hanson & Kraus (1980) for H=0.7 m, φ=20°, t= 1 day.

$$z(y)\frac{\partial y(x,t)}{\partial t} = k_0\frac{\partial^2 y(x,t)}{\partial x^2} \tag{2. 27}$$

This equation can be transformed to

$$\frac{\partial y(x,t)}{\partial t} = \frac{k_0}{z(y)}\frac{\partial^2 y(x,t)}{\partial x^2} = K\frac{\partial^2 y(x,t)}{\partial x^2} \tag{2. 28}$$

in which K_o = constant linked to wave and sediment properties.

Eq.2. 28 can be solved numerically upon adequate assumptions for boundary conditions, yielding the curve $y(x,t)$ for shoreline. An analytical solution can not be found due to the variability of $z(y)$; (in Pelnard-Considere's model this quantity was constant $z(y) = h$, Taking $z(y) = h$ one obtains a solution to Eq.2. 28 as a function $y(x,t)$ analogous to formula 3.3 or 3.5 depending on which time interval $t \lesseqgtr t_1$ is considered.

Recent years have been numerous numerical solutions for shoreline evolution derived in Japan. The study by Matsuoka and Ozawa (1983) is a single-line model consisting of two parts. The first part describes the wave field together with wave diffraction and refraction while the second part computes shore-line evolutions. The continuity equation

$$\frac{\partial y}{\partial t} + \frac{1}{h}\frac{\partial Q}{\partial x} = 0 \tag{2. 29}$$

is applied by analogy to the diffusion equation assumed by some other investigators. The model was used for various types of structures, among others for a system of two piers having different length normal to shoreline. The results of computations obtained were only partly in agreement with the figures measured in the field, Matsuoka and Ozawa (1983).
One of the most recent studies has been provided by Hanson (1987). It presents a system of numerical models referred to as GENESIS, which was derived for shoreline evolution at structures. All versions of the model are based on the one-line theory and on the assumption that the shore profile remains constant. It is only the shoreline position which varies.

Unlike the other models derived from the same concepts, GENESIS represents complex algorithms which can deal with systems of structures having different locations and shapes. Wave refraction and diffrafion is included together with sediment flow about groyne and sources and sinks of sediment, e.q. in estuarial areas.

Aside from the models for direct forecast of shoreline evolution at structures one should also mention the methods for computations of waves and currents alone. For given waves and currents one can subsequently assess the patterns of shore cross erosion and accretion. This problem is not however dealt with within this report. The readers interested in the interaction of waves and currents with structures may be addressed to the two-dimensional model elaborated by Liu and Mei (1976), which has been implemented on IBM PC at IBW PAN. The wave section of that model is based on the linear approximation and assumes isobaths parallel to shoreline. The current patterns about structures are computed from the Navier Stokes' equation averaged over depth and time, including three components of radiation stree, bed friction and gravity forces.

Liu and Mei's model was tested against the data measured in a wave tank, Skaja and Szmytkiewicz (1982). Satisfactory agreement of the computations with the hydraulic data has been noted.

In summary, at should be stated that despite numerous studies done hitherto,from an engineering point of view the most effective and useful models are based on the one-line theory. This is due to a variety of reasons. First of all, our knowledge of cross- -shore transport in an n-line forecast of shoreline at structures is insufficient. Secondly, the degree of complexity involved in n-line models is overly high for sophisticated boundary conditions, even in the numerical implementation.

Next, the single line theory is so successful because despite its simplicity it represents prototype conditions quite well.

Field observations have shown that dramatic changes in beach profile due to a severe storm are smoothed out after a relatively short time of the order of a few days, or weeks at most. The shore profile returns to its pre-storm shape, although its origin (shoreline) can be displaced. As a result, the shore transformation is a periodic phenomenon.

Observations of high stability of shore profiles in longer terms and relative instability of shoreline in shorter scales indicate that the models based on the one-line theory are particularly useful.

Depending on specific coastal conditions and practical necessities one may recommend the refined model derived by Le Mehaute' and Soldate (1980), together with a complex system of computational procedures GENESIS presented by Hanson (1987).

2.7.2 *Shore-Parallel Structures*

Emerging Structures

The interaction of the coast and shore-parallel structures becomes complicated compared with the case of cross-shore structures. A sophisticated system of short-crested waves arises as a result of interference of progressive and reflected waves.

Strong wave diffraction is noticeable about extremities of a structure. A higher angle of wave incidence a flow along structure is observed which promotes scouring at the toe.

Any mathematical modelling of the pertinent phenomenon is a complex task. Some studies, mostly Japanese and American appeared in the seventies to provide a theoretical description and forecast of the morphological changes at shore-parallel structrures.

The study by Madsen and Grant (1976) contains a numerical model of bed evolution for a shore-parallel detached breakwater.

The basic equation used by Madsen and Grant (1976) is the differential equation of continuity for sediment transport and bed changes, together with the time-averaged equation for longshore sediment transport in the direction of the axes x and y:

$$(1-p)\frac{\partial z}{\partial t} + \frac{\partial \bar{Q}x}{\partial x} + \frac{\partial \bar{Q}y}{\partial y} = 0 \qquad (2.\ 30)$$

$$\bar{Q}x = \frac{1}{T}\int_0^T Qx(t)dt \qquad (2.\ 31)$$

$$\bar{Q}y = \frac{1}{T}\int_0^T Qy(t)dt \qquad (2.\ 32)$$

The wave diffraction about the structure is described by the Penny Price algorithm. Wave reflection has been parametrized with the aid of the reflection coeficient.. Examples of computations have been produced for a shore -parallel impermeable detached breakwater of length L = 660 m (ie. six wave lengths). Using their numerical procedures and boundary conditions Madsen and Grant (1976) arrived at bed evolution behind the breakwater (Fig.2.38).

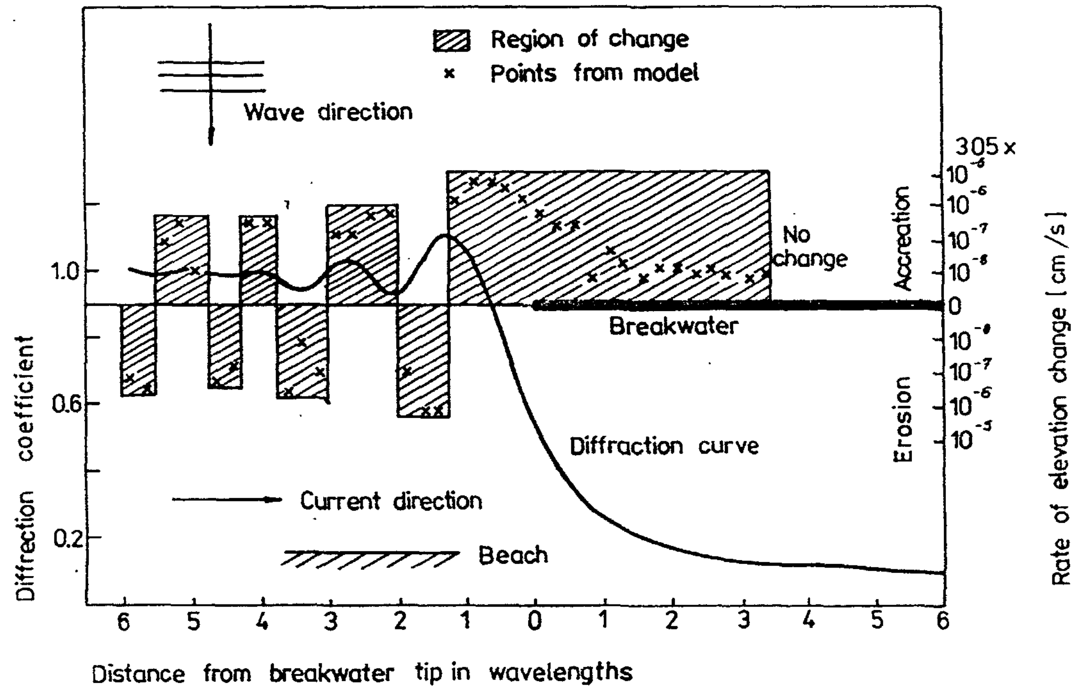


Figure 2.38. Bed Changes and Diffraction Coefficient at One Wavelength Behind Breakwater, Grant and Madsen (1976).

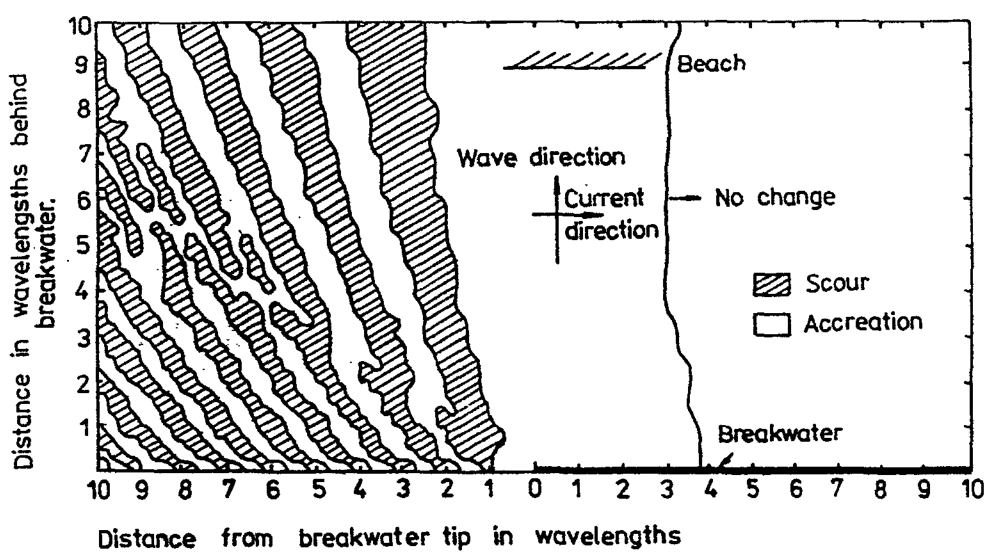


Figure 2.39. Three-Dimensional Bed Changes Behind Breakwater, Grant and Madsen (1976).

It should be noted that a clear-out relationship exists between local maxima and minima of the diffraction coefficient and the accretion and erosion areas. The erosion occurs where the diffraction coefficient increases (1.5 to 2.0 wave length away from the brakwater head). Behind the breakwater head, about one wave length away, the diffraction coefficient diminishes and accretion is observed. The three-dimensional bed changes due to the variation of sediment transport behind the breakwater is shown in Fig.2.39.

By inclusion of wave reflection through adding a standing wave, Madsen and Grant have obtained the three-dimensional bed configuration on the seaward side of the structures.
It should be noted that Madsen-Grant's model provides a fairly general mathematical description of sediment transport and of erosion and accretion zones for a shore-parallel structure quite remote from shoreline.

A forecast method for bed transformation at a shore-parallel structure located closely to shoreline is given by Perlin (1979). This study provides a numerical model for bed evolution in the sheltered area behind a detached breakwater. The model is based on the one-line theory and includes wave reflection and diffraction. Similar to Grant-Madsen's model, the basic system of equations used by Perlin includes relationships for continuity of sediment mass and longshore transport, whereas in terms of the one-line theory one has

$$\frac{\partial y}{\partial t} + \frac{1}{h}\frac{\partial Qx}{\partial x} = 0 \qquad (2.\ 33)$$

The model is reported on in more details by PERLIN and DEAN (1978). The computations were performed for the case when an expanding salient merged with the breakwater and created a tombolo. Dimensionless parameters for optimum location and size ot the breakwater have also been given.

A number of further studies appeared in the eighties, mostly in Japan: Kraus (1983), Mimura, Shimuzu, Horikawa (1983), Kraus, Hanson, Harikai (1984) or Hanson, Kraus (in press). Much as for the cross-shore structures these studies also differ from one another by numerical details having however untouched the basic scheme. The equation for shoreline evolution in all models mentioned reads

$$\frac{\partial y}{\partial t} + \frac{1}{D}(\frac{\partial Q}{\partial x} \pm q) = 0 \qquad (2.\ 34)$$

in which $D \approx h$
q = cross-shore transport or a quantity representing sediment sources and sinks along shore
Q = longshore transport.

Some differences do appear in the determination of the longshore transport. Kraus (1983), Kraus, Hanson and Harikai (1984) or Mimura, Shimizu and Horikawa (1983) and Hanson (1987) introduce the recent formula by Ozasa and Brampton

(1980), while Matsuoka and Ozawa (1983) and Borah and Balloffet (1983) continue to use one of the earlier CERC versions

$$Q = A.P_b.sin\alpha_b.cos\alpha_b \tag{2. 35}$$

in which P_b = wave energy flux due to breaking
A = empirical factor (A = $2.17.10^4$; Savage)

Ozasa and Brampton's formula is to be applied in particular if wave is diffracted about a breakwater or headland and its height changes along shore, giving rise to the gradient $\frac{\partial H}{\partial x}$. The starting points for this method can be found in Komar and Inman (1970).

For oblique wave incidence and diffraction around a structure Ozasa and Brampton's formula reads

$$Q = \frac{H_b^2 \cdot c_{gb}}{16(\rho_s/\rho - 1)(1-p)}(K_1 \sin 2\alpha_b - K_2 \frac{\partial H_b}{\partial x} \cot\beta \cos\alpha_b) \tag{2. 36}$$

in which K_1 and K_2 are model calibration parameters.

Komar-Inman's formula appears if $\frac{\partial H_b}{\partial x} = 0$. The first part of Eq.2. 36 corresponds to the CERC formula for sediment transport due to an oblique wave without diffraction. The other term modifies (reduces) the longshore transport due to the longshore variation of wave height and becomes important if wave is subject to diffraction, that is in the neighbourhood of the structure.

The coefficients K_1, K_2 are not specified clearly. CERC recommends $K_1 = 0.77$, while some other reports suggest $K_1 = 0.1...0.6$. Model tests have shown that the quotient K_2/K_1 varies from 0.5 to 1.5, Kraus, Hanson and Harikai (1984). The two parameters, K_1 and K_2 to calibrate the numerical model, counterbalance the action of the two forces due to the oblique wave incidence and the longshore variation of the wave height.

The practical application of the models consists in the use of the same equation Eq.2. 34 and Eq.2. 35 or Eq.2. 36, for different boundary conditions. Some examples are given by Kraus (1983) and Mimura, Shimizu and Horikawa (1983) who compare the computations of shore evolution with measurements for a single detached breakwater.

Matsuoka and Ozawa (1983) provide an identical comparison both for a single breakwater and a group of three shore-parallel detached breakwaters. In all these studies their authors indicate good agreement of computations with measurements carried out in hydraulic laboratories.

A contribution by Hanson (1987) provides a gross model referred to as "Genesis", already referred to, which makes possible the computation of the effect of systems of structures on the coast. The study contains examples of computations and the physical and mathematical background.

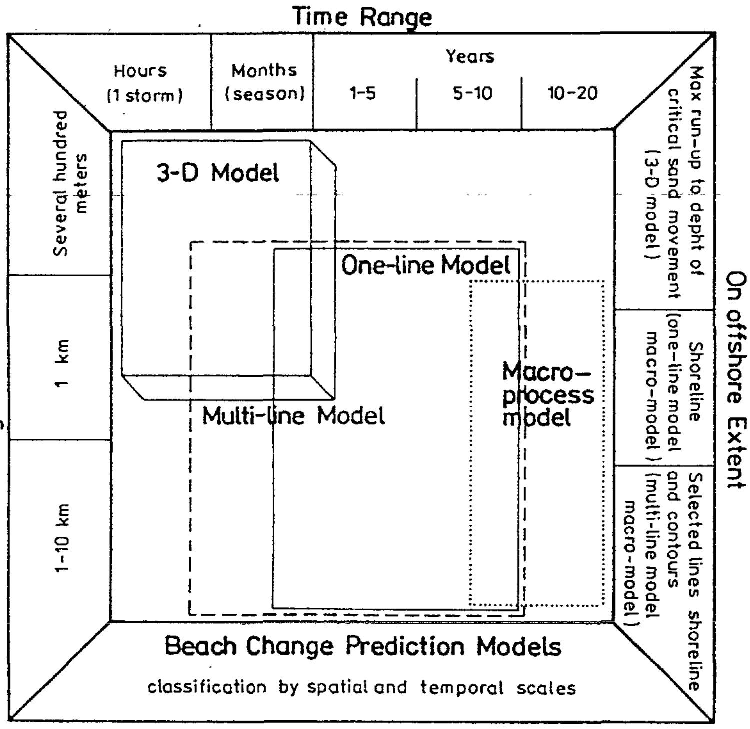


Figure 2.40. Classification of Shore Evolution Models, Krauss (1983).

In the USSR the shoreline evolution at a detached breakwater was modelled by Khomicki (1983):

$$y(x,t) = \frac{S_3}{2\sqrt{\pi a t}} exp(-\frac{x^2}{4at}) \tag{2. 37}$$

in which S_3 = the area between the x axis and shoreline contour at given time t
a = empirical accretion factor.

Testing of Eq.2. 37 has shown considerable effect of the relative structure length on the formation of coast-accretion forms and their shapes. For normal wave incidence the optimum breakwater distance from shoreline Y_B should not be greater than the structure length $L_B(L_B > Y_B)$, while for oblique wave attack $\varphi > 15^o$, the breakwater length $L_B = (1...3)Y_B$.

If the longshore sediment transport is reduced or interrupted the coastal accretion form becomes distorted by flattening along the x axis, proportionally to $\sqrt{t}$, Khomicki (1983).

Kraus (1983) has provided a summary of numerical models and their applicability in the forecast of shoreline evolution at structures. The following four groups of models have been summarised: one-line models, multiple-line models, three-dimensional models and macro- -models. The division is shown graphically in Fig.2.40.
From among the categorised groups the best and most suitable for our tests is the

three-dimensional model. It allows for analyses of local phenomena, but it is not fully applicable due to some theoretical gaps. The integration and averaging of various components of the three-dimentional model bring it to the multiple line model or even to a one-line version.

A number of informations and generalization for shorter times can be obtained with a one-line model. Because of the variety of time scales involved the groups of models categorised above multually intertwine, Fig.2.40.

The M - line model is an expansion of the one-line model and makes possible the representation of shoreline evolution with a few lines, together with a cross-shore sediment transport. This may be exemplified by Bakker's two-line model. There are many possibilities for search of modified solutions in agreement with the complexity of an analysed coast.

The one-line model is applied most frequently since it predicts long-term changes of major coastal properties, and permits multiscale phenomena to be investigated along with local peculiarities of coastal line with regard to energy and accretion. This model is the only practical tool for engineering computations. The fourth group of models is in contrast to the three-dimensional models because it provides general trends in coastline evolution for macroscales in time and space. Therefore models of this group provide rough estimates only. An example is the simplest one- line model with constant wave fields (without refraction and diffraction) and for simplified boundary conditions.

The prediction of waves and currents for the sheltered area between the breakwater and shoreline is provided by the two-line model of Liu and Mei (1976). This model is adequate for a long breakwater located seawards of the break point and small uniform bed slopes. Liu and Mei's solution takes into account the wave refraction and diffraction phenomena but neglects the convection effects and the longitudinal turbulent diffusion, much as for the cross-shore structures.

Submerged structures

Coast evolution is closely related to transmission of wave energy towards beach. Upon prediction of wave flux one can indirectly model the eventual bed changes. Using this type of control one may enforce erosion or accretion. This type of structures is most common in Japan and the USSR.

The review of studies on shore parallel submerged structures and their effect on coastal evolution is given by Onoszko, Tarnowska and Zeidler (1980). It embodies the studies up to 1980 and includes primarily the effect of structures on changes in regular wave height.

Mei and Black (1969) provide theoretical background, basing on small amplitude waves and intended for submerged structures having various widths at their top.

One of the most recent studies was published by Baba (1986). It summarises the existing computational methods for wave attenuation and coastal defence with a shore-parallel submerged breakwater. This analysis, basing on laboratory tests, has shown that Goda's theory is the best among the methods assessed (Lizov 1962, Goda 1967 and 1969 and Seeling 1980). The analytical background is provided by the transmission factor K_t, as noted earlier in Sec.2.3.3.

As emphasized, results of the computations of wave attenuation due to a shore parallel submerged breakwater should be assessed with care. This stems from the fact that the available forecast methods are based on laboratory investigations for regular waves, Mei and Black (1969), Baba (1986). The applicability and clearness of these models also depend on a number of empirical lump factors, which are usually selected quite arbitrarily.

Sea Walls

The forecast methods for beach evolution in the vinicity of shore-parallel structures (sea wall type) should take into account the following principles:

- shoreline must not retreat landwards behind the sea wall frontline
- mass continuity law must be fulfilled
- the direction of littoral sediment drift must agree with nutural tendencies of local sediment transport.

Like in the case of some other sea defence structures, cross-shore (groynes, jetties etc) or shore-parallel (detached breakwaters), also in the case of sea walls, the basic equation for shore evolution assumes the earlier general form:

$$\frac{\partial y}{\partial t} = -\frac{1}{D}\frac{\partial Q}{\partial x} \tag{2. 38}$$

The other equation used in shore evolution models describes the longshore sediment transport, most often such as earlier equations. The effect of coastal structures (sea walls) is incorporated in the boundary conditions (Ozasa and Brampton, 1980).

Most forecast methods for shore evolution at sea walls were devised at the end of the seventies. They all are based on the one-line theory. Some differences stem from various assumptions imposed on numerical schemes and their boundary conditions.

Ozasa and Brampton (1980) proposed a mathematical model for the evolution of sea wall-protected shore by assuming some characteristic boundary conditions for longshore sediment transport. They assumed that the effect of a structure on the longshore sediment transport may be neglected if a sufficiently wide beach berm is created between the shore line and the sea wall. However, if strong erosion occurs on beach and the berm is washed out, the longshore sediment transport rate becomes reduced in the model. This rate decreases to zero for a continuing tendency towards shoreline retreat and its eventual merging with the sea wall. By this means Ozasa and Brampton reach the state of static equilibrium for shore profile.

Hanson and Krauss (1980) proposed a simple predictive model for which some conditions were imposed on shoreline changes at the sea wall. If the local sediment transport on the landward side of the structure appeared then it was assumed that the shoreline did not retreat behind the sea wall but was identical with the sea wall frontline.

The studies by Hanson and Krauss on the effect of sea walls on shore evolution were continued by Krauss, Hanson and Harikai (1984), Hanson and Krauss (1985) and recently Hanson (1987).

The first of these studies includes examples of shoreline evolution computations for a system of sea defence structures i.e. sea walls and some other mesures. The computations were again based on the one-line theory and the authors conclude that the results were satisfactory and compared swell with the measurements carried out at the harbour of Oarai in Japan.

Hanson and Krauss (1985) discuss the limitations arising due to the fact that shoreline must not retreat behind the sea wall. These restrictions should be on the same level of schematization as the one-line theory. Hence the model can include a number of simplifications, such as the neglect of wave reflection, local scour, accretion, flow and undermining. Hanson and Krauss claim that if the shoreline is identical with the sea wall frontline then the boundary conditions with zero longshore sediment transport rate are less justified than the transit type of transport. The direction and volume of the transport are conserved. These assumptions, much as the method itself cannot be applied if the shore at the sea wall comes to a level lower than the water level. The synthetical neumerical model referred to as GENESIS, mentioned above, by which shore evolution at a sea wall can be computed, being an outcome of longer studies by Hanson and Japanese investigators, deserves serious attention.

Mc Dougal, Sturtevant and Komar (1987) provide simple equations for the landward encroachment of sea and the size of the eroded area on the downdrift side of the sea wall. The empirical relationships follow:

$$r = 0.101 L_B \tag{2. 39}$$

for the maximum reach of erosion r, where L_B = structure length; and

$$s = 0.689 L_B \tag{2. 40}$$

for the erosion area (length).

From Equations 2. 39 and 2. 40 it may be seen that the predicted length of the eroded shoreline on the downdrift side of the structure corresponds to about 70

% of L_B while the maximum scour in the landward direction is about 10 % L_B.

In conclusion, it should be stated that shore evolution at sea walls can be predicted with a satisfactorJy accuracy only in a few field cases under strictly observed conditions, Hanson and Krauss (1985). The maximum shoreline retreat at the very proximity of a sea wall, in the form of an erosional embayment, is assessed as 10% of the structure length L_B in the landward direction and $0.7L_B$ in the downdrift direction.

2.7.4 *Dune Erosion*

With reference to numerous parts of this document it is worthwhile to outline forecast methods for dune erosion.

Numerical Modelling of Extreme Shore Events

Since 1954 coastal engineers and scientists have realized that a shore profile maintains a certain dynamically stable shape. Bruun (1962) postulated conservation of volume in shore profile, in the absence of longshore transport rate gradients, as illustrated in Fig.2.41. A similar assumption was made by Pelnard-Considere (1954) who thereby originated a large family of one-line models, which later expanded to two-line and m-line versions, as already discussed.

A somewhat equivalent principle of dune erosion was accepted by Vellinga (Fig.2.42), who performed extensive large-scale experiments in Delta flume, derived a prediction model and calibrated it against field data (hindcast for 58 shore profiles for the storm surge of 1976 which caused mean erosion of 30 m^3/m with extremes up to 80 m^3/m, and for the storm surge of 1953 that brought about erosion of 100 m^3/m). Sargent and Birkemeier (1985) demonstrated that application of Vellinga's model was justified for U.S.East Coast and the Gulf of Mexico. Moreover, Vellinga's prediction is being currently implemented in the Dutch practice of (probabilistic) dune design, to which we refer later.

Kriebel and Dean's (1985) study provides an m-line model in which the Bruun concept is modified with assumptions on power-type shape of shore profile (the Dean profile), mode of wave energy dissipation, etc. The schematic diagrams in Fig.2.43 illustrate the idea of m-line theory (a) and the types of shore assumed (b) and (c). Numerical sensitivity tests of Kriebel and Dean's model have exposed the following effects:

1. storm surge is more important, for beach retreat, than wave height in the initial stage of storm

2. fine-sand beach responses more slowly than coarse sand

3. beach response to storm is long so that only 15 ... 30% of potential damage due to maximum surge is felt in fact.

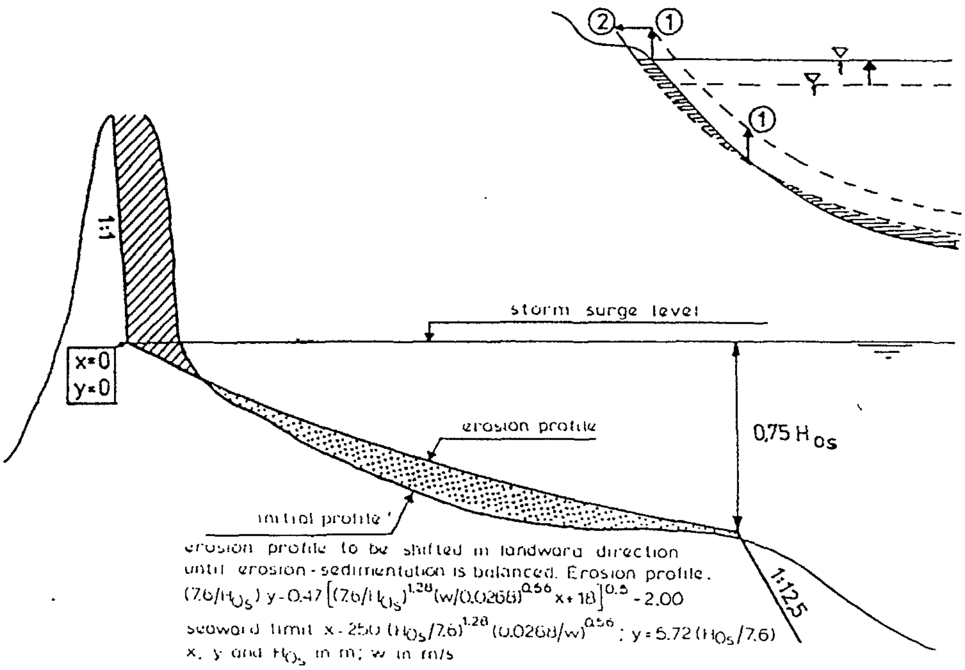


Figure 2.41. Per Bruun's Concept of Equilibrium Profile (top).
Figure 2.42. Vellinga's Model (bottom).

Some results of the testing are presented in Fig.2.44.

Kriebel and Dean fitted their method into a more general scheme of Monte Carlo simulation of erosion climate which permitted probabilistic evaluation of beach volume eroded (Fig.2.45) and a guideline curve for design of dune recession (Fig.2.46). Although the derivation was originally intended for tropical and subtropical storm surges, the probabilistic aspect incorporated via Monte Carlo simulation seems to filter out local or geographic effects so that the graphs might be helpful in approximate design of erosion due to severe storms.

In his probabilistic analysis of dune erosion, van der Graaff (1986) claims that the erosion rate and its prediction depend on the following factors:

- maximum storm surge
- wave height (e.g. significant) in the stage of maximum storm surge
- grain size distribution curve of dune
- initial shore profile
- duration of storm
- wind gusts;

all of them supplemented by

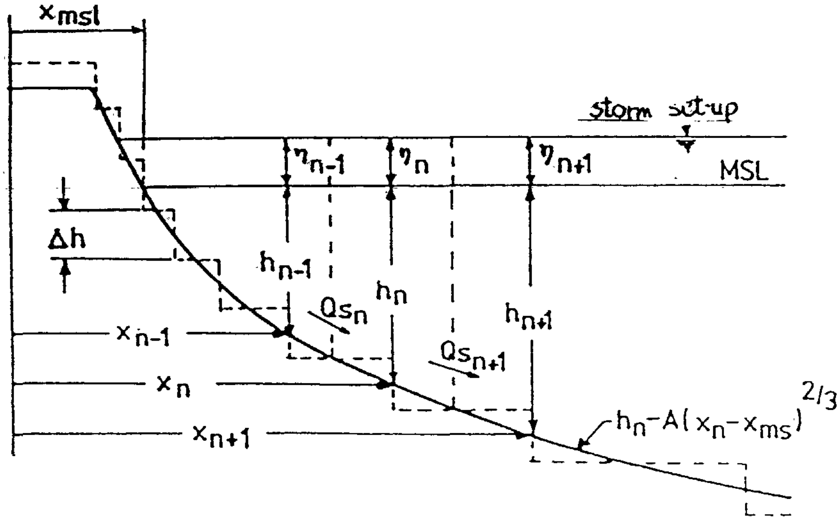


Figure 2.43. Kriebel & Dean's m-Line Model.

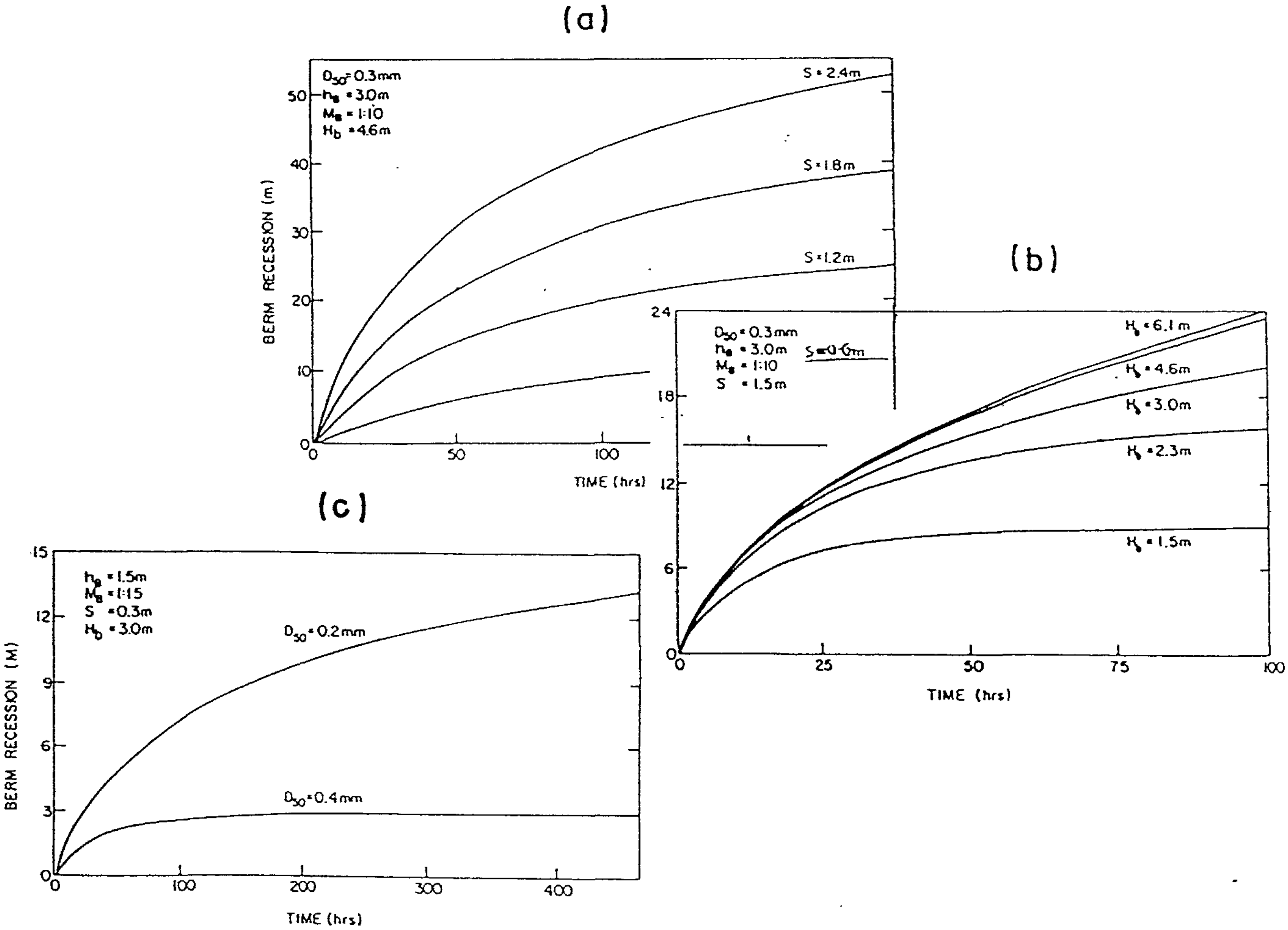
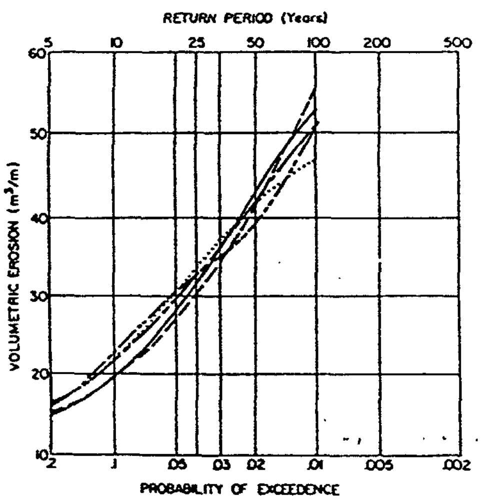


Figure 2.44. Testing of Kriebel & Dean's Model.



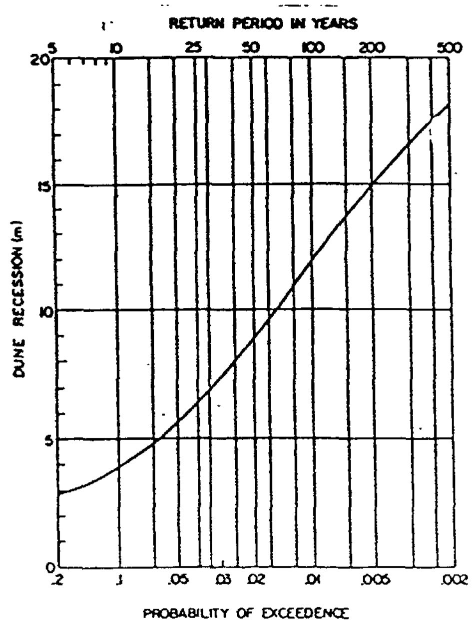


Figure 2.45. Kriebel & Dean's Probabilistic Assessment of Dune Erosion (left).
Figure 2.46. Kriebel & Dean's Design Curve (right).

- accuracy of computations.

Graaff distinguishes three levels of probabilistic dune design, as discussed in our Chapter One, viz. (III) exact probabilistic methods, (II) approximate methods, with linearization about thoroughly selected points and (I) engineering methods employing safety factors. Level III requires multi-dimensional probability density functions, which are generally unknown and involve tremendous amounts of computations, so that they are not used in common practice.
At level II one assumes Gaussian distributions of all mutually independent factors. The exceedance probability of storm surge level is taken by Graff as

$$Pr(\underline{h} > h) = \alpha \exp(-\beta h) \tag{2. 41}$$

in which
$\alpha = 727.86$ and $\beta = 3.01$ for Hook van Holland.

Graaff also assumed that the distribution of significant wave height H_s during storm surge was Gaussian, with the mean value given for Hook von Holland by

$$\mu_{H_s} = 4.82 + 0.6h - 0.0063(7 - h)^{3.13} \tag{2. 42}$$

on the depth of water h extending from 3 to 7 m above MWL.

Wind gusts were supposed to increase the volume of erosion A by

$$\Delta A = 0.125 A \Delta h \tag{2. 43}$$

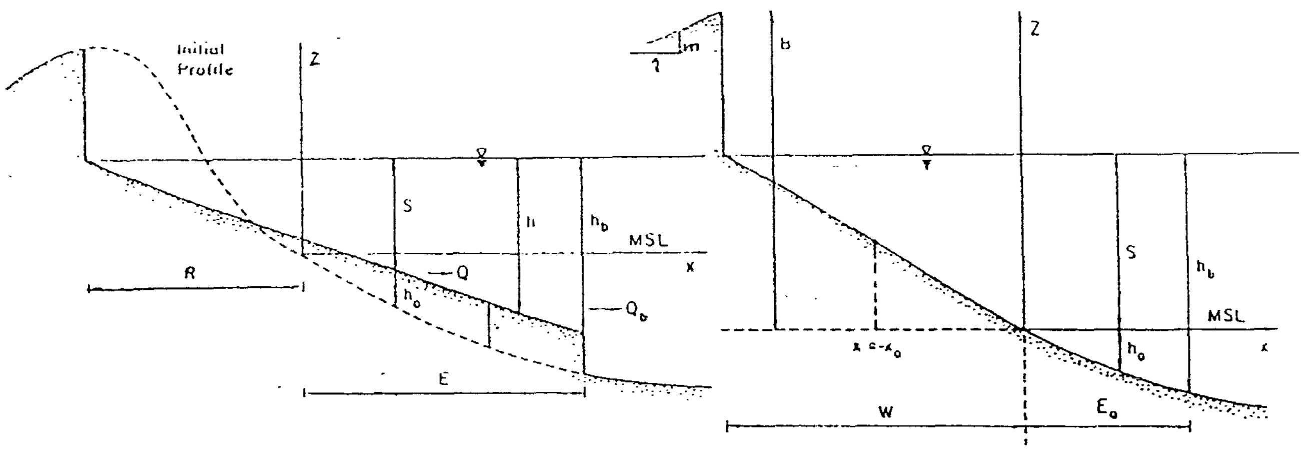


Figure 2.47. Kobayashi's (1987) Model.

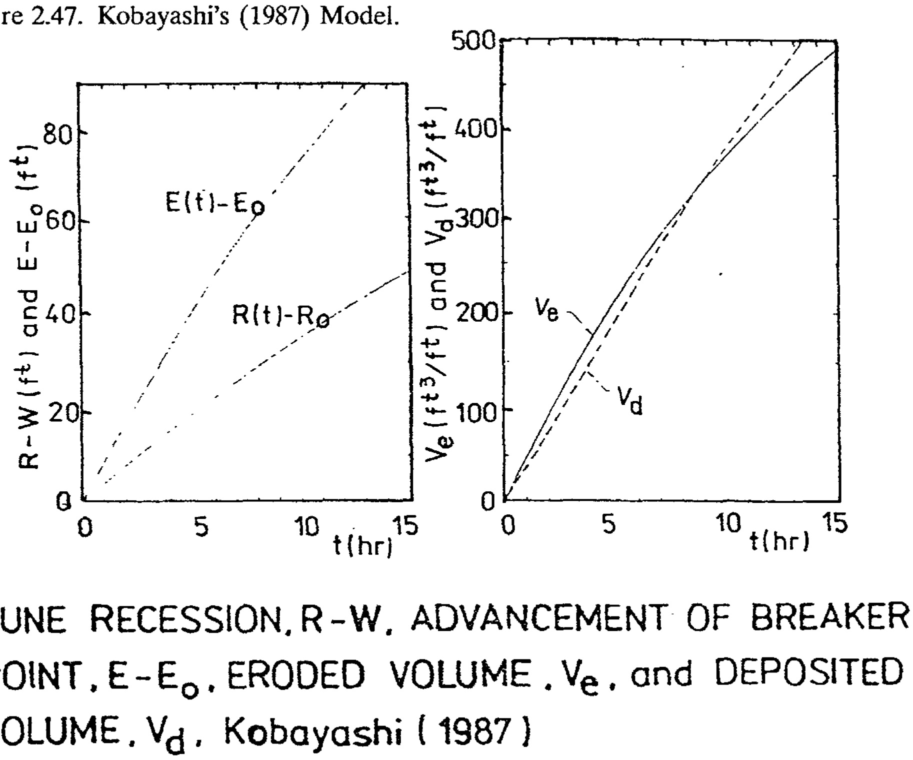


Figure 2.48. Kobayashi's (1987) Results.

in which
Δh is the superelevation of storm surge caused by the gust bumps.

Graaff (1986) took for granted the Vellinga model and carried out computations at level II. Storm surge, i.e. rise in water level has appeared the most important factor.

In order to make our review more complete it is appropriate to mention another analytical method for dune erosion by storms. Kobayashi (1987) refers to Kriebel and Dean (1984, 1985) and proposes a simplified solution for a rectangular storm surge hydrograph. The temporal variations of the dune recession and eroded sediment volume are expressed in terms of the given intial profile, storm surge, breaker depth, and sediment characteristics. The schematic diagrams in Fig.2.47 clarify the concept. Practical results may be obtained from the analytical findings summarized in Fig.2.48.

CHAPTER 3

Near-field effects: scour and other failure modes

3.1 SCOUR

3.1.1 *General*

Scour is a particular mode of instability at the interface of a structure and its environment, and not necessarily the structure itself. Described in a very elementary way, scour is local erosion seen as removal of soil particles from the soil-water interface. As it occurs at a structure and not entirely inside it, scour must **not** be deemed strictly an internal **failure** mode, in terms of Section 3.2. It depends heavily on the layout, geometry and dimensions of structures; the controlling hydrodynamical factors such as waves and currents; and properties of sea bed. Hence it seems more appropriate to place the scour phenomena in the category of environmental near-field phenomena, i.e. in the class of external stability processes. Nonetheless, as it occupies a *very special* place among failure modes, scour certainly deserves this *separate section.*

Erosion of the sea bed close to or at a sea wall, breakwater, or related structure constitutes a frequent design problem. Local scour has often occurred on shingle, sand or clay beaches, but has also affected structures founded on chalk and other soft rock foundations. Erosion of beach or foundation material constitutes one of the major causes of the failure of sea walls in the UK, causing 12% of collapses directly, and being partially responsible for a further 5% . In the USA toe scour has caused many problems both during and after construction.

In considering the loss of bed material in front of a structure it is convenient to distinguish between **general bed levels changes** and **local scour.**

Predictions of changes to general bed levels requires a comprehensive knowledge of the beach morphology, and falls outside of the scope of this chapter as it is treated in Chapter 2. Changes to bed levels close to a structure may frequently be correlated with the changes to the hydrodynamic regime locally due to the construction of the structure. The two principal effects caused by such a structure are the increase in wave orbital velocities due to reflections, and concentration of wave-induced and tidal currents. At shoreline structures the influence of wave reflections is of paramount importance. The effect of current is of significance if structures protrude far away from the shoreline.

In the subsequent text we are beginning with the most prominent exhibition of **scour** that takes place in front of **seawalls** and **offshore breakwaters**, and then continue with **prediction of toe scour** at longshore structures. The information on **scour at cross-shore structures** is much scarcer. **Local erosion** at piles, pipelines and wharfs, although sometimes less important, is more abundant.

As a good introduction, one may examine the general case of a structure situated in a coastal zone.

When a coastal structure is built on the porous seabed of the shoaling zone, the structure itself will obvously react with the natural wave train. Most coastal structures - even flexible rubble mounds - are quite "hard", compared with the highly mobile natural porous seabed sediments. Except under extreme waves, therefore, coastal structures tend to reflect a high proportion of the incoming "normal" classes of waves.

The consequences of this phenomenon are illustrated in Fig.3.1a devised by Smith (1989). Due to the interaction between incident and reflected waves, the hydraulic stirring capacity, in front of the structure is enhanced. As a result of this, the reflected wave generates its own partial "beach" in front of the structure. This results in the so-called "scour-hole", but it is in fact a "negative" or offshore sloping local beach.

When a major storm arrives however, this quasi-stable wave reflection relationship becomes destroyed. The larger storm waves rapidly flatten the seabed in front of the structure —its "bar" effect is eliminated and a larger wave is able to reach the structure itself. This larger wave then breaks directly on the structure and the reflection action becomes reduced almost to zero. This storm effect is depicted in Fig.3.1.b which demonstrates the minimum design wave exposure that could be applied under these circumstances.
A thin layer of porous sediment may be quite capable of "training" normal moderate waves, but it may prove completely inadequate, for coping with trains of very large waves. Then the available sediment capacity may be totally overwhelmed, and the sediment will be wept off the impermeable seabed and deposited in the offshore zone, during Nature's efforts to construct a deeper water storm bar.

3.1.2 *Scour at seawalls and offshore breakwaters*

"Scour" is commonly used as a generic name for a variety of near-field erosion effects. More strictly, it consists not only of the mode of **undermining**, in front of structures as discussed above, but also of **outflanking** as sketched in Fig.1.12.

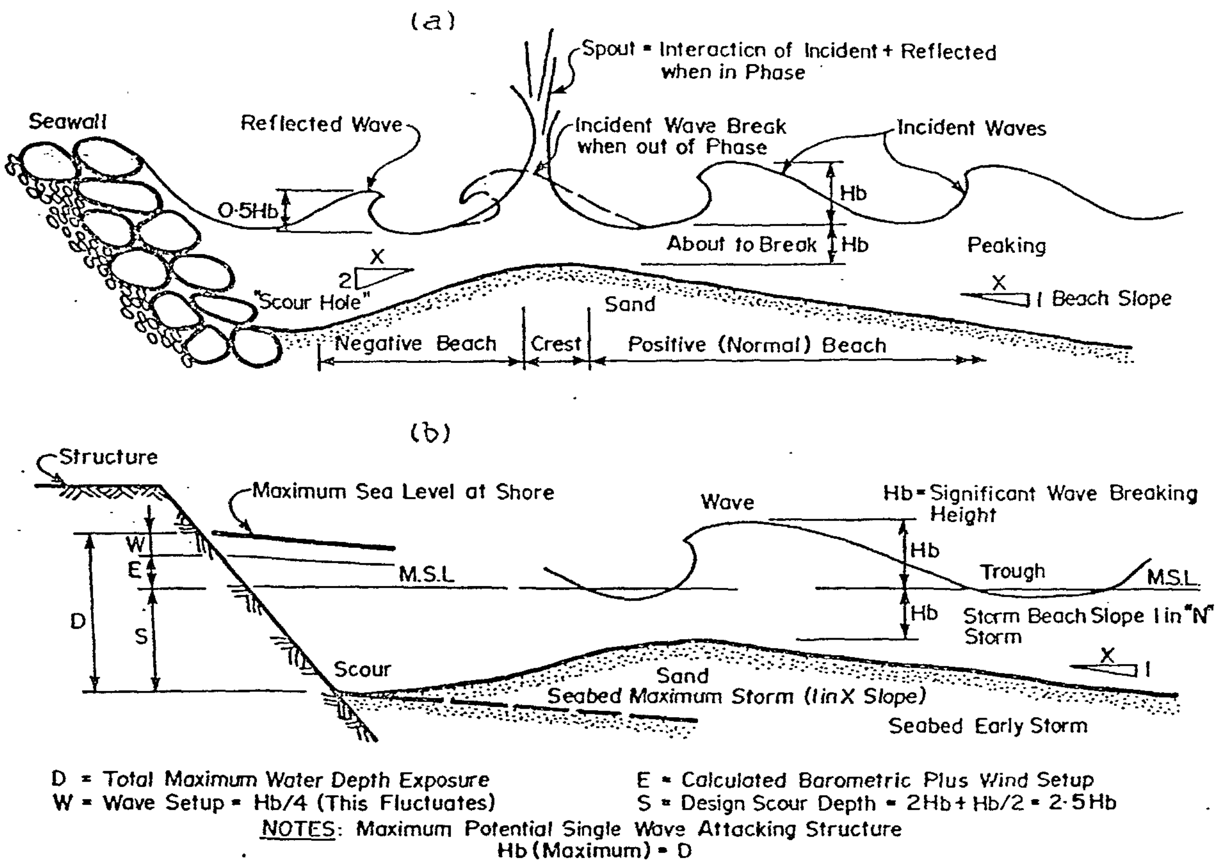


Figure 3.1. Smith's (1985) Conceptual Outline of Scour Generation & Self-Adjustment.

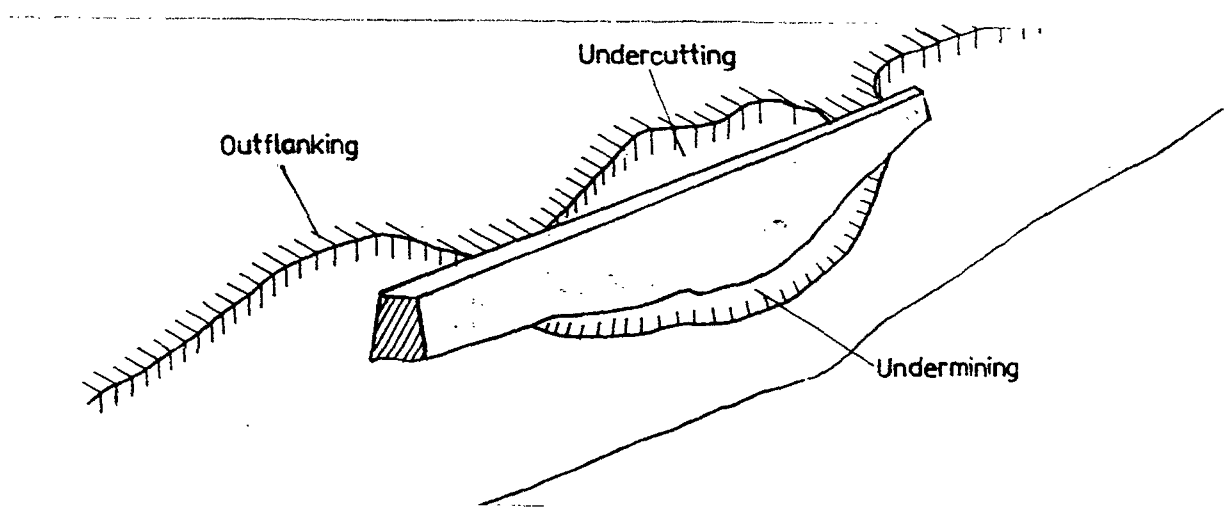


Figure 3.2. Three Modes of Scour at Sea Wall.

In addition, one may also identify the mode of **undecutting**, on the back side of a sea wall (Fig.3.2), or any structure. The latter is caused by water which overflows, overtops, bypasses, etc. the structure, and brings about the said removal of soil from behind the structure. In the following text we will first concentrate on scour, understood primarily as the frontal erosion effect, and move next to outflanking.

It is clear worldwide that scour endangers the stability and performance of any coastal structure. For instance, Aminti et al. (1983) tested in laboratory 13 types of submerged breakwaters (impermeable structures) of trapezoidal cross-section, with various angles of slopes, elevations and width of structure crest, rubble-mound structures, shell structures and special armour structures. Those structures were sited on depths of 2 and 3 m and were exposed to regular waves having heights H = 1...2 m and periods $T = 5...8s$. The objective of the tests was stability of the structure; the investigations have shown that rectangular structures and shell structures exhibited the most critical behaviour relative to **scouring**. Strong erosion appeared at the beach side footing of the barriers, leading in some case to the collapse of the structure. The shell structures exhibited the most critical behaviour relative to scouring.

There has been a lot of controversies on the effects of seawalls on the beach, not only as to their morphological impact and effectiveness in the far field but also with regard to local erosion in the near field.

The summary provided by Horikawa (1981) yields a succinct outline of the possible situations (scour types) and magnitudes. Scour in front a structure takes place if the balance between beach profile and external forces is disturbed locally or over a wide area due to the presence of the structure. Thus, in most of the previous experimental studies on scour, an equilibrium profile without the presence of the structure is established first, and then the beach profile change after the installation is examined.

The effect on scour as a function of the location of installed structures has been studied, for example, by Sawaragi and Kawasaki (1960), Sato, Tanaka, and Irie (1966), Hosori and Arakawa (1967), Chestnutt and Schiller (1971), Noda and Iwasa (1972), Hattori and Kawamata (1977), Xie (1981), and Saeki et al. (1985). Pioneering laboratory experiments on the effects of vertical walls on the beach were performed by Dorland (1940) and Russel and Inglis (1953).

Fig.3.3 illustrates characteristic dimensions of scour: x is the location of the structure measured from the position of the original shoreline, x_b is the original location of breaker point before installation of the structure measured from the position of the original shoreline, h_o is the initial depth at the front of the structure, and Δ_h is the scour depth.

Sato et al, (1966, 1968) examined the scour characteristics for a vertical wall installed at different locations on various beach profiles, as shown in Fig.3.4. The experiments were carried out both in a medium size wave flume with a wave height,

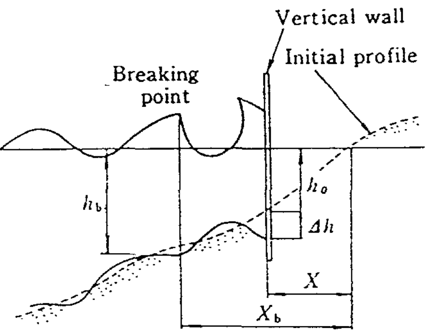


Figure 3.3. Scour Definition Sketch, Horikawa (1988).

H_o, of 8.7 cm and wave periods, T, from 1.3 s to 3.0 s, and in a large wave flume with wave heights from 26.0 cm to 42.4 cm and wave periods from 2.3 s to 5.1 s. The median size of the bed material, d, ranged from 0.21 mm to 0.69 mm. In Fig.3.4, beach profiles are shown with respect to offshore wave steepness, H_o/L_o, and the ratio of offshore wave height to the median size of the bed material, H_o/d. It was found that the scour time history exhibits four distinct types, as shown in Fig.3.5.

Rapid scour always occurred if breaking waves acted directly on the wall. Three of the scour types belonged to this situation: if the structure was installed near the shoreline, the scour hole mostly refilled (Type I); if the structure was installed landward of the shoreline, no refilling process took place (Type II); and if the structure was installed between the breaking point and the shoreline, rapid scouring occured initially but it became much slower afterwards (Type III). Scour developed gradually without refilling, if the depth at the front of the wall is great enough to allow formation of standing waves (Type IV).
Fig.3.6 shows the maximum scour depth at vertical walls installed at different locations on the beach profiles shown in Fig.3.4. In the drawing, the ratio of maximum scour depth to offshore wave height, $\Delta h_m/H_o$ is shown with respect to offshore wave steepness, H_o/L_o, and the ratio of offshore wave height to median size of bed material, H_o/d. The greatest scour appeared to occur about the wall installed at the location between the breaking point $(x/x_b = 1)$ and the shoreline $(x/x_b = 0)$ at the beginning of the experiment, and the scour history in this case was of Type III, as shown in Fig.3.6.

When the wall was installed in the surf zone, significant scour occurred, because the return flow caused by the reflected waves from the structure transported bed material seaward. The effects of reflected waves on the scouring process have been studied by Sawaragi and Kawasaki (1971) and Saeki et al. (1985). Refilling of the scour hole in front of the structures by changing the characteristics of the incident

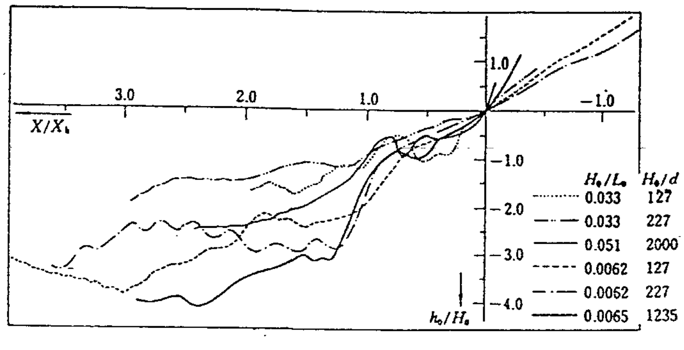


Figure 3.4. Scale Model Beach Profiles, Sato et al. (1966, 1968).

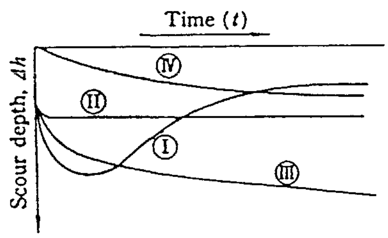


Figure 3.5. Scour Types at Sea Wall.

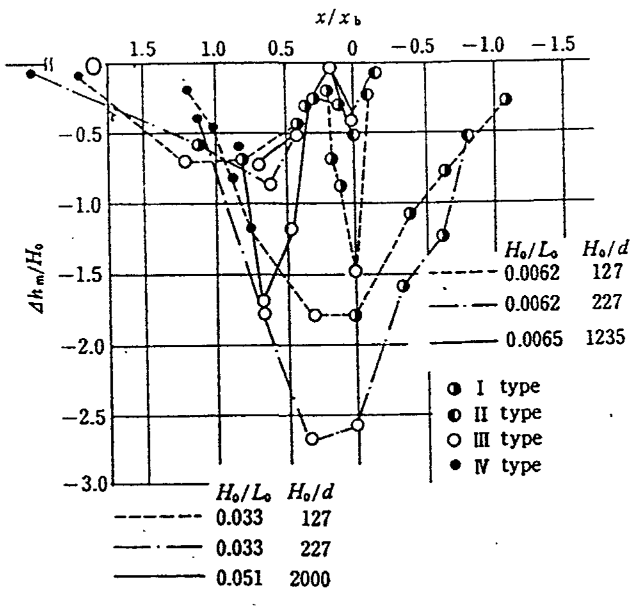


Figure 3.6. Maximum Scour Depth at Sea Wall.

waves has been studied by Hattori and Kawamata (1976) and Hashimoto, Tanaka, and Tsutsui (1981).

Fig.3.7 gives an example of scour in front of a breakwater in the field. The scour depth, Δ_h, was measured at various locations along the breakwater, where the initial water depth, h_o, increases toward the tip of the breakwater. The envelope in this figure suggests that the maximum scour would occur if the breakwater is located at $h_o = 23m$, which will be located between the shoreline and the breaking point under an average rough sea condition. The scour depth would decrease both seaward and landward of the location of maximum scour. Similar results were found in the small-scale laboratory experiments.

Fig.3.8 provides an example of scour at the toe of a detached breakwater which was constructed with concrete blocks in water depths of 3 to 4 m, for the purpose of shore protection (Katayama, Irie, and Kwakami, 1974). Three sections were above 20 years old since inception of the construction works. The scour depth depended upon the crest height of the structure; it was less than 2 m when the crown height was kept from +1.1 m to +3.0 m . However, it was doubled up to 4 m when the crown height became - 2 m due to heavy subsidence. Scour at the toe resulted in subsidence of the detached breakwater, and concrete blocks had to be repeatedly supplied for more than twenty years to maintain the necessary crest height and cross-section. Subsidence of the detached breakwater was investigated by a side-scan sonar (Nishida, et al., 1985).

Fig.3.9 shows the cross-section of the detached breakwater including the buried portion, which indicates that almost twice the number of concrete blocks needed to fill the design cross section were buried due to long-term subsidence. The cause for the severe subsidence in this case was not only the scour but also the loss of the underlying material due to piping and wave agitation.

If structures are placed in deeper water, the sea bed on their offshore side is subjected to scour by standing waves. Hattori (1969) proposed a relationship describing the vertical distribution of suspended sediment in the standing wave. Ishida, Hayashi, and Takahashi (1981) developed a mathematical model for the rate of bed-load movement in the standing wave, which indicates that accumulation takes place at the nodes and scour at loops if there is a strong suspension of bed material near the sand ripples. Such a phenomenon was first reported by Bijker and Wichers (1971).

Irie et al. (1984) (see also, Bijer and Wichers, 1971; Xie, 1981) also identified the existence of two different modes of bed material movement under standing waves: the first mode in which suspended load is markedly transported in the direction from nodes to loops (L-type), and the other mode in which bed load is mainly transported in the direction from loops to nodes (N-type). Resultant bottom profiles associated with these modes are illustrated in Fig.3.10. The net direction of bed.material movement is determined by the relative magnitude of the two modes

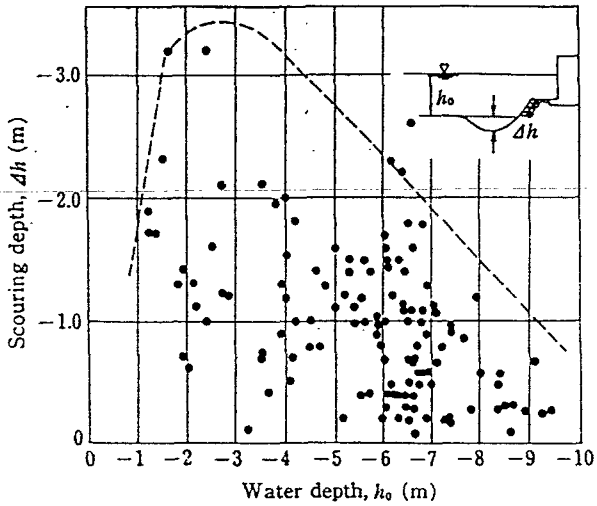


Figure 3.7. Scour at Kashima Port Breakwater, Horikawa (1988).

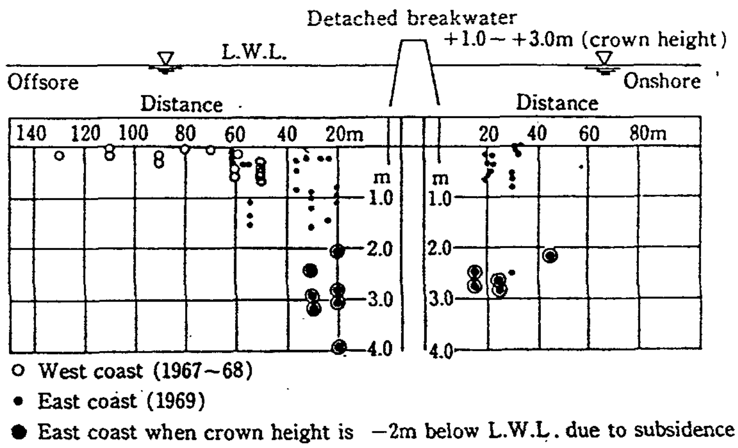


Figure 3.8. Scour at Detached Breakwater.

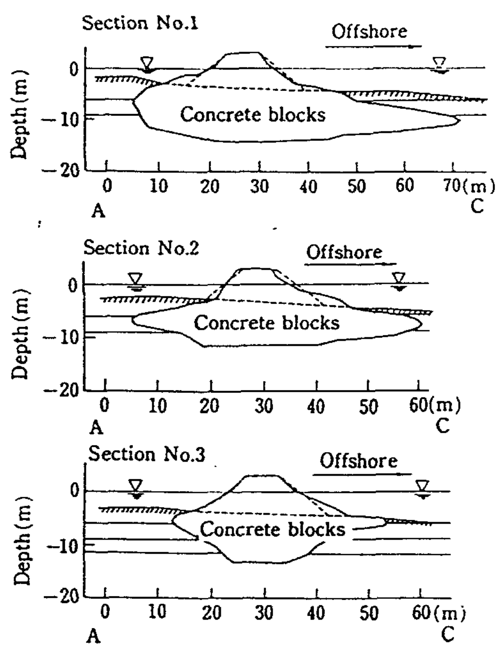


Figure 3.9. Subsidence of Concrete Units at Foot of Detached Breakwater (Original Dashed Line).

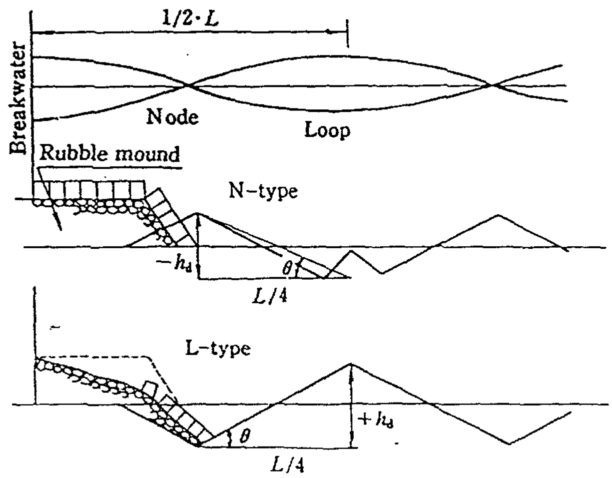


Figure 3.10. Two Types of Scour at Rubble-Mound due to Standing Waves.

of movement, and it may be represented as a function of two parameters, u_0/w_o, and the Ursell parameter, Ur, where u_0 is the amplitude of the horizontal water particle velocity of the incident waves near the bed, w_o is the fall velocity of the bed material, and $U_r = HL^2/h^3$.

Kadib (1963) established that the depth of scour decreased with increasing grain diameter and grew for intensive wave reflection, i.e. for increasing wave steepness.

At present, a major research programme is under way at Hydraulics Research Ltd to study the potential erosion of shingle beaches fronting seawalls,Owen (1989). The research involves mobile bed model studies in a random wave flume, in which the beach profiles in front of the seawall are very carefully monitored for various designs and positions of seawalls. Vertical seawalls, sloping revetments, and rock mounds have been examined, at positions varying from the toe to the crest of the natural beach. The tests are now almost complete and the results are presently being analysed.

Scour depth appears as a complex function of the wave steepness and the dimensionless water depth at the location of the seawall for the natural beach. For seawalls which are located close to or above the still water line the beach level actually builds up, except during periods of very steep storm waves. However, for seawalls located below the still water line erosion occurs for most wave conditions. In reality of course the wave conditions are changing continually, and the net effect on beach levels will be the summation of all the different erosion and accretion depths. When the results of all the tests have been analysed it will hopefully be possible to compare the scour potential of different designs of seawall, and to predict the exact shape of the beach profile after the seawall is constructed.

The effect of waves and currents about a structure is pronounced as spots of intensive erosion and accretion. These spots may be sources of damage, failure and instability, in particular if they occur in the shallow zone. The prevention is sought in the construction of special protective aprons and, most often, additional revetments.

Investigations on seawalls at IBW PAN date back to late sixties (Tarnowska, 1970). Considerable scour was observed at the toe of seawall due to waves of significant steepness and height. Later on, longshore structures have been tested at IBW PAN since 1984 for identification of the major hydromechanical factors controlling the sedimentation at the coastal structures, and their impact on shore evolution. The studies were conducted in two phases: the first, merely for waves and currents around structures, in the tank with fixed bed and the second, with movable bed, for complex measurements of wave, currents and topography. The data for each of the twelve cases studied include stereophotogrammetric maps of the free surface (yielding both the mean elevation of water, wave parameters and other geometric properties of the free surface), velocities of water, and bed topography. The second phase continued through 1988.

Judging from the measured fields of waves and currents, the longshore structure located at the berm ($X_B = 0$) does not bring about significant transformations of shore, as long as it remains in the swash zone. Important transformations take place for the case $0 < X_B < X_b$. The agitation of sea bed in the interference area of short-crested waves on the seaward side of the structure, in combination with the considerable longshore current promotes erosion and degradation of the shore profile.

At the same time, the attenuated motion of water in the shadow area gives rise to the accretion of sediments and perhaps the formation of tombolo type features, as pointed out in our discussion of the far-field effects in Chapter Three. For the most distant structures ($X_B > X_b$), no remote transport of sediments is to be expected seawards of the structure where the seabed is agitated locally but the littoral drift is too weak to move it awey. Erosion might however be anticipated between the beach and the structure.

The above sedimentation patterns, as inferred from the hydrodynamical fields, have been partly supported by the evidence from the mobile-bed wave tank tests. For wave parameters as those in the fixed-bed tests (H =10 cm, T=1 s and H/L = 0.064), upon normal wave attack, there appeared insignificant reflection from the sea wall at water line ($x_B = 0$). Rip currents were at right angles to the structure. Scour occurred at the centre of the sea wall, and the eroded material was accumulated at the tips of the sea wall. These patterns changed for oblique wave attack. Short-crested waves in the reflection zone were superimposed on longshore currents, and the eroded sand was evacuated beyond the sea wall zone, although partly at the downdrift extremity of the structure. This accretion disappeared when the waves were stronger (H = 12.5 cm and T = 0.9 s). It can be concluded that once scour is generated in front of a sea wall, the reflection intensifies the ambient climate, versus the fixed-bed situation.

Hence, one of the most controversial questions relating to beach scour is whether seawalls lead to sand loss in front of them. The Army Corps of Engineers (1977, 1981) indicates that toe scour can be expected in front of a seawall, and Smith and Chapman (1982) describe such an effect in front of rip rap walls in Australia. Observations along the central coast of California have turned up no conclusive evidence that seawalls caused beaches to become narrower over time at any of the study sites. However, this effect is difficult to observe using primarily qualitative data. It might also be quite transitory, only occurring during storms and high tides, when the greatest wave energy is exerted at the base of the wall. In any case, rip rap seawalls, because they are so wide relative to vertical structures, can cover significant portions of sandy beaches, making them less usable for recreation and somewhat hazardous for public access. The summer beaches at many locations have not gone away , but may simply build up against the cliffs, under the revetments, as if the rocks were not there.

A comprehensive review of the effects of seawalls is presented in special issue of *Journal of Coastal Research (No 4, 1988)*; the highlights are emphasized below, although sometimes it is difficult to split up the far- and near-field effects.

A review by Kraus (1988) of the literature on the effect of seawalls on the beach covers approximately 100 technical papers on laboratory, field, theoretical and conceptual studies.

Kraus (1988) concludes that the beach change near seawalls, both in magnitude and variation, is similar to that on beaches without seawalls, if a sediment supply exists. Sediment volumes eroded by storms at beaches with and without seawalls are comparable, as are post-storm recovery rates.

In addition, the shape of the beach profile after construction of a seawall is similar to the preconstruction shape if a sediment supply exists, showing the same number of bars with approximately the same volumes and relative locations. The form of the erosional response to storms at seawalls is typically different, however, with foreshore erosion that occurs on beaches without seawalls manifested as more localized toe scour and end effects of flanking and impoundment at seawalls. Limited evidence indicates that the subaqueous nearshore profile on a sediment-deficient coast with seawalls does not steepen indefinitely, but approaches an equilibrium configuration compatible with the coarser-grained particles comprising the bottom sediment.

Weggel (1988), too, sees as conjecture the claims that seawalls have caused exclusive erosion. He presents examples of seawalls where the beaches fronting them have responded in various different ways. In at least one case, at Santa Cruz, California, a beach formed in front of a seawall where no beach had existed before. In another case a seawall remained buried beneath the beach but became exposed to provide protection and prevent further erosion during a period of unusually high waves.

Six types of seawalls are defined. The classification depends on a seawall's location on the beach and on the water depth at the wall's base. A type-1 wall is located landward of the limit of storm wave runup. Type-1 walls have negligible influence on hydraulic and coastal sediment processes. At the other extreme, type-6 walls are located seaward of the normal breaker line so that they are usually subjected to the action of non-breaking waves. Seawalls type 2 through 5 lie between and have an increasing effect on coastal processes as the type number increases.

In passing, similar findings have resulted from a great variety of tests in a wave tank with movable bed (Pruszak, Tarnowska, Zeidler, 1988).

The longshore structures located at the beach berm have brought the aforementioned scour and evolution of shore, while those constructed in the surf zone have sometimes caused very considerable transformations.

The importance of sand supply in the impact of seawalls is emphasized by Morton (1988). His field studies at three barier island sites on a microtidal storm-dominated coast document the effects of seawalls on (a) relatively stable, (b) slightly erosional, and (c) moderately erosional beaches. Analysis of beach dynamics using aerial photographs and beach profiles indicates that all seawalls reached by storm waves temporarily increase frontal beach erosion by concentratiing scour at the seawall base. These deep scour troughs rapidly fill as beach and bar systems return to equilibrium positions. If an adequate sand supply is available, complete post-storm beach recovery occurs in four interdependent stages, rapid forebeach accretion, slower back-beach aggradation, dune reconstruction, and vegetation recolonization. Even severely eroded beaches in front of seawalls can experience forebeach recovery, but seawalls may reduce or prevent the other three stages of recovery by impending the onshore transport of sand that elevates the backbeach and builds the dunes.

All the field evidence of Morton's study indicates that seawalls locally increase magnitudes and rates of long-term beach erosion with lateral erosion being greatest on the downdrift ends of those walls extending onto or seaward of the forebeach. These protruding structures alter the sediment budget by either intercepting sand transported by longshore currents or preventing sand from entering the littoral system. Beach profiles seaward of seawalls first respond to diminished sand volume by decreasing elevation while maintaining profile shape. As erosion proceeds, the profile above man sea level becomes concave upward and the radius of curvature decreases causing a local steepening of the beach adjacent to the seawall base. With continued erosion, the entire profile becomes subaqueous. Longshore bars adapt to diminished sand volume and increased water depth by migrating landward and developing a curvature that, in plain view, is convex toward the seawall.

Wood (1988) has employed empirical eigenfunction analysis to analyze an extensive Great Lakes beach and nearshore profile data base. He concludes that the primary region of spatial change (outerbar) is similar of armoured and unarmoured lengths of shoreline. Correlation of profile response between armoured and unarmoured lengths of shoreline is strongest for periods of rising lake-level and weaker during periods of falling lake-level. It appears that lack of an upland sediment source on extensively armoured shorelines may hinder the reestablishment of the outer-bar during periods of falling lake level. Analysis of sub-aerial beach response in front of extensive lengths of seawall indicates that beach width diminishes systematically from the downdrift to the updrift end. This observed narrowing also appears to be directly related to a lack of upland sediment source.

Griggs and Tait (1988) report on a one-year study along the central California coast carried out in an effort to resolve some of the critical questions regarding the impact of these structures on beaches. Based on precise, biweekly, wading-depth surveys in the vicinity of four protective structures, a number of consistent seasonal beach changes were observed. (1) With the arrival of winter waves, the summer berm in front of the four seawalls was eroded before the berm in front of the adjacent unprotected beaches. (2) Where sloping permeable revetment is adjacent to an impermeable vertical seawall, the berm in front of the vertical seawall was eroded sooner. (3) Once the berm on the unprotected beach retreated landward beyond the seawall, beach face profiles in front of the seawall and adjacent beach were usually indistinguishable. (4) Accelerated erosion occurred at the downdrift ends of the seawalls and extended as far as 150 m downcoast. This excess scour is believed to result from a combination of wave reflection and sand impoundment upcoast. (5) Rebuilding of the nearshore beach profile occurred in a uniform manne with no obvious differences between seawall backed and adjacent unprotected beaches.

Pilkey and Wright (1988) argue that active beach degradation due to hard shore-parallel structures remains a real possibility and point to a number of mechanisms by which seawalls can accelerate erosion. They have compared the dry beach width on selected stabilized and unstabilized U.S.East Coast shorelines and note that this width is consisently and significantly narrower in front of walls. The more dense the hard stabilization, the narrower the beach. Beach destruction over several decades can be much more important than that caused by single envents or short-term changes.

Terchunian (1988) proposes a procedure for calculation of the amount of sand that would be naturally eroded from the upland versus the amount of increased erosion caused by a coastal armouring structure. Using this information it is possible to determine the amount of beach required to mitigate the potential adverse impacts of coastal armouring structure.

Seawalls and riprap revetments are used on the Oregon coast to protect properties threatened during erosion of seacliffs and sand spits Komar and Mc Dougal (1988). The erosion processes involve the combined effects of swash from storm waves and embayments cut through the berm by rip currents. Most structures on the Oregon coast have not been designed by engineers, and therefore commonly do not follow sound design practices. They often are grossly overbuilt and do not take into consideration the processes of erosion and beach morphology. Particularly difficult is the protection of high cliffs where the structures can defend only the toe of the slope. Questions have been raised as to whether the structures might contribute to erosion of the beach and adjacent unprotected properties. Although such adverse impacts have been demonstrated in Komar and Mc Dougal's laboratory wave-basin experiments, the role of rip currents in Oregon-coast erosion produces significant longshore variability in property losses and tends to mask impacts by structures.

Rip rap walls certainly reflect a smaller percent of wave energy than do vertical wood or concrete walls. However, during high tides and under certain wave conditions, reflected waves from rip rap walls have been observed to combine with incident waves, causing erosion and damage in adjacent areas, especially in small embayments.

Although a rip rap wall absorbs more wave energy than do relatively smooth, impermeable structures, it has a sloping seaward face. Because not all of the wave energy is absorbed, under high tide and storm wave conditions, waves running up a rip rap revetment can damage houses or erode the fill behind the rip rap.

Where maintained, rip rap has proven relatively effective in slowing erosion, but maintenance costs, even for engineered rip rap, are usually quite high. For example, the total wight of rock necessary to protect idividual bluff-top lots in the Santa Cruz area over the last ten to fifteen years ranges from 500 to 2000 tons, or approximately ten to twenty-five tons per foot of ocean frontage.

(At today's average cost of $35 to $45 or more per ton, these walls may cost perhaps a third as much as the value of the property they are protecting).

Scour at the **toe** of coastal structures is a function of the total hydraulic flow field existing at that point, which includes both wave and current components, Eckert (1983). In rubble-mound structures, the toe apron serves primarily to keep scour from undermining the cover layer armour units. Such scour allows some units to slump down, opening a breach in the protective armour through which waves may attack the underlayer. A second function of the toe apron is to prevent the formation of a scour hole at the toe deep enough to destabilize the embankment structure by the oversteepening and lengthening of the ocean side slope. In rubble-mound structures, the toe apron design criteria are generally dominated by the hydraulic criteria.

The effect of water currents at the toe of coastal structures, both in causing scour and in accelerating wave-induced scour must be considered in designing scour protection. Such currents may result from the presence of the coastal structure, i.e., longshore currents caused by the oblique reflection of waves from the structure; or they may have already existed prior to the structure's construction due to specific site conditions, i.e., tidal currents which pass over the site of a coastal structure.

While studies indicate that a scour hole in front of a rubblemound structure may occur anywhere within one-fourth wavelength of the incident wave (Herbich et al., 1965), the actual area to be protected is not generally that wide. The geotechnical concerns of slope stability and foundation bearing strength will generally be met if the bottom sediments are protected within a width of the wall. The construction of a toe apron to avoid undercutting of its edge, or overbuilding in width to allow for undercutting is important to the successful functioning of the apron.

We will examine some of these features in the following subsection.

Outflanking & Undercutting

Undercutting about two different types of coastal structures is illustrated in Figures 3.11 and 3.12. Undercutting, or generally removal of sand from behind a palisade-type sea wall, and beneath concrete blocks placed on fascine between two rows of piles, which occurred at a few locations of Polish coastline, is shown in Fig.3.13. **Outflanking** can be classified as either near-field or far-field, depending on the scale of the structure and phenomena involved. We include some aspects in this chapter, although comments could also be added at places in Chapter 2.

Outflanking occurs when material to either side of a seawall erodes to a point where it threatens or damages the wall itself, or property behind it. Along a progressively eroding coast, all successful, isolated protection structures will be gradually outflanked, because the coastline on either side will erode more rapidly than that behind the wall. This is a relatively predictable process, and should be planned for in the design of any isolated wall in a rapidly eroding area. Most often, it is taken into account through the use of "wing walls" running landward from the ends of the main structure. However, because of high costs and practical difficulties (particularly along cliffed coasts), such future outflanking is usually ignored until it causes property damage. In this case, the costs of outflanking must be considered in the expected maintenance costs and overall lifetime of the structure (Griggs, 1986).

Often, outflanking of one wall leads to the construction of additional walls adjacent to the first. As the amount of continuously protected coastline increases, outflanking becomes a problem in the unprotected gaps. Nonetheless, both for isolated walls as for gaps in protected coastline, one question remains: do sea walls increase the erosion of adjacent areas ?

The answer to this question depends on many factors, such as how far seaward the wall extends, the type of coastal environment, and the type of wall involved. No systematic observations have apparently ever been made to support a generalized answer to this question for the California coast. For two concrete walls in dune

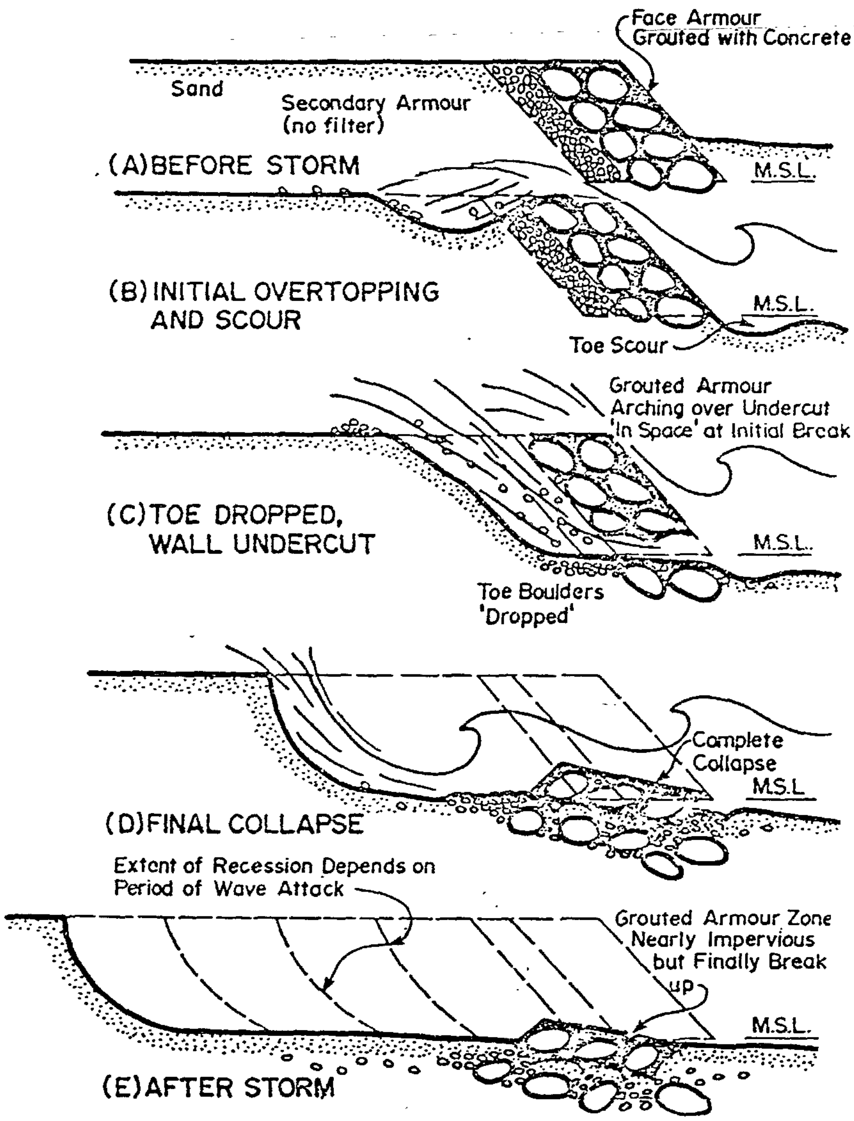


Figure 3.11. Example of Undercutting Behind Grouted Rubble Revetment.

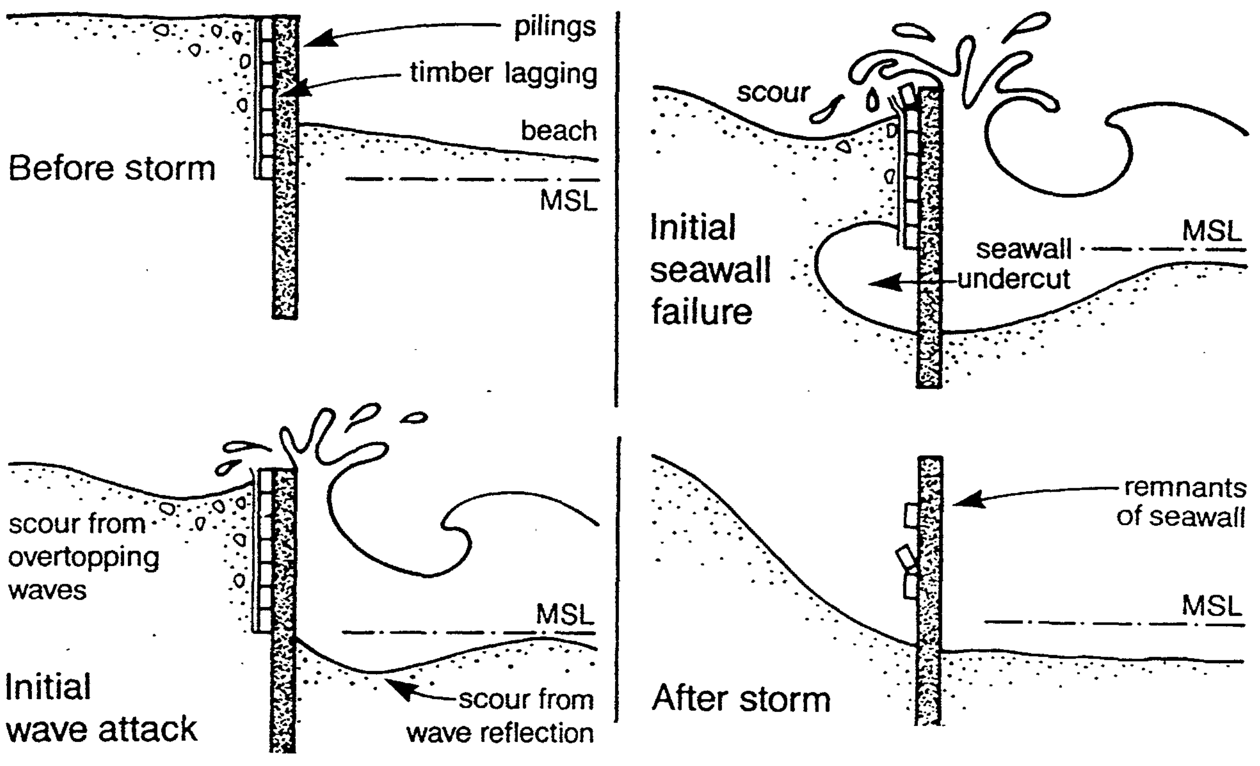


Figure 3.12. Typical Undecutting of Wooden Sea Wall (Smith & Chapman, 1982).

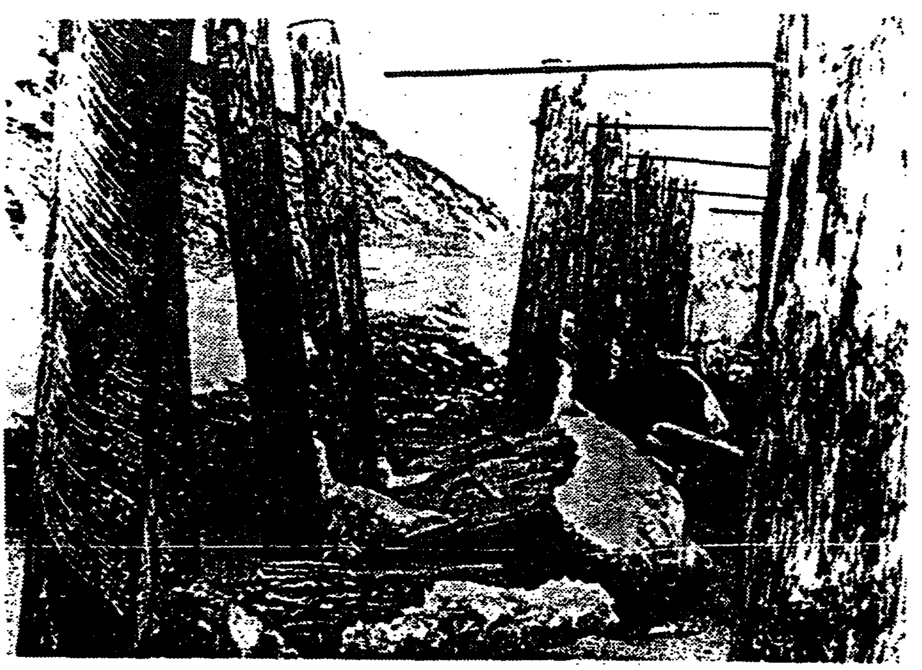


Figure 3.13. Final Stage of Undercutting (and other failure modes) at Palisade Sea Wall.

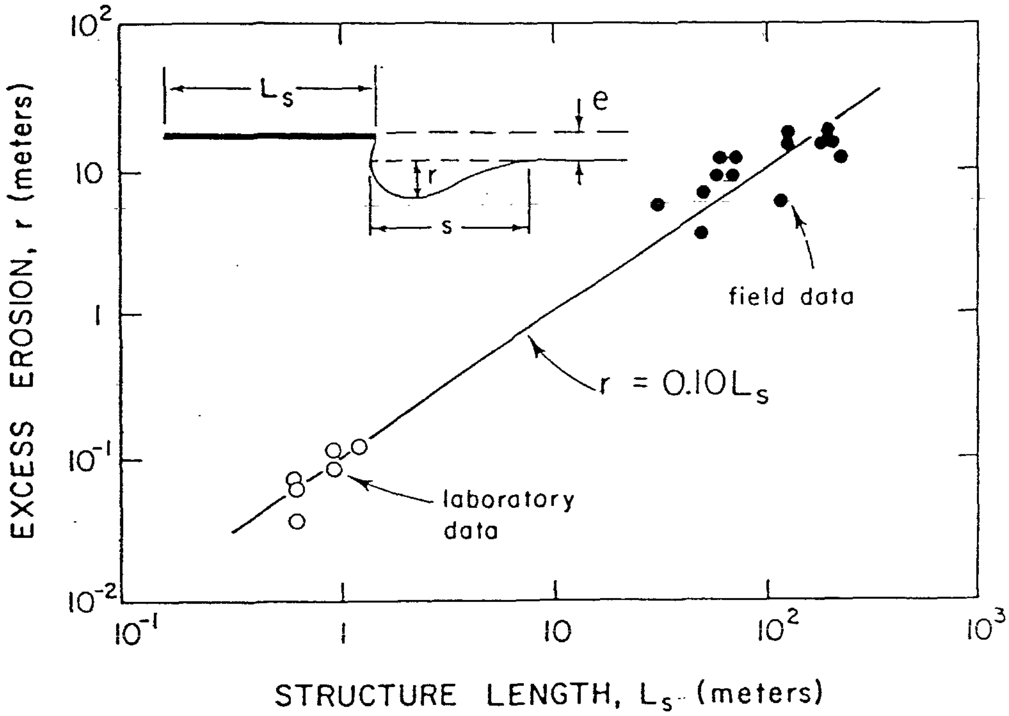


Figure 3.14. Outflanking by Komar and McDougal (1988).

environments, (Sites No.1 and 3), no accelerated wave- caused erosion along the flanks of the walls has been documented. Rip rap at one site settled much more around gaps in the continuous revetment than elsewhere during 1983 storms. However, this settlement may have .been due to outflanking, or to some other factor.

Rip rap revetments placed within some rocky coves appear to have increased erosion problems along their flanks. This increased erosion may be due to a concentration of wave energy in indentations of the rocky coast, especially during storms and high tides. During high tides and low sand levels, wave splash in thrown further inland from these indented areas than from the more linear portions of the bluffs. This situation may be analogous to that created by indented portions of seawalls, or gaps between seawalls.

The outflanking on the downdrift side of a longshore structure reflects the along-coast impact. Komar and Mc Dougal (1988) added their laboratory data to Walton and Sensabaugh's (1978) field data to postulate on excellent agreement of the excess erosion (outflanking) r as one-tenth of the seawall's length, L_S, cf. Fig.3.14. The same data yields $s = 0.69L_S$ for the length of the outflanking area s.

Outflanking is enhanced by overtopping water (or generally , by the water intruding the landward side of seawall, e.g. through structural joints etc.). The most hazardous situations arise if outflanking erosion features on both sides of seawall merge, and a strong current along the structure, on its landward side, endangers stability and puts in question the effectiveness, durability and lifetime of the seawall (or other longshore structures).

3.1.3 *Prediction of Scour*

Prediction methods. Predicting the level to which a beach may be scoured, and the exact wave conditions which will cause scour is difficult, at best. Coastal engineers have used a variety of "scour depths" in designing seawalls. Most of these are within 3 feet of the Mean Sea Level in the Monterey Bay area (where 0 feet Mean Sea Level equals approximately +3 Mean Lower Low Water), Fulton-Bennett and Griggs (1988). However, rip rap has been observed to settle to depths of five to ten feet below Mean Sea Level, as it moves seaward under wave attack. Since thre are no widely accepted formulae for calculating these depths, estimates based on field observations made during or (more commonly) after storms are used. Because of the rapid rate at which beaches may be rebuilt after being scoured under severe storm conditions, observations made days after a high tide period (when the beach is not inundated by storm waves) may be misleading. In areas where bedrock is deep, borings are often used to determine the depth of storm lag deposits, consisting of gravel and cobbles. However, several layers are often encountered, and in the absence of accurate dating methods, the selection of a design or expected scour depth can be quite uncertain.

The depths to which scour occurs depend heavily on how far landward or seaward a structure is located on the beach scour and liquefaction should increase rapidly with increasing distance seaward. Thus, there is a inherent problem in any solution that involves moving a structure seaward-the amount of energy it receivers and the scour at its base will be greatly increased. For example, one rip rap revetment in northern Monterey Bay (Site No.27) was designed for a scour depth of 0 feet MLLW, and a height of 19 feet MLLW - equal to or greater than those of adjacent walls. However, because this wall was approximately 100 feet seaward of the other walls in the area, it suffered much more severe overtopping and undermining during 1983, and ultimately failed altogether.

Toe scour may be considered to be the process of localised erosion occurring immediatelly in front of a sea wall, subject to wave and current action. The scour depth S_d, is therefore the maximum depth of localised erosion, relative to the initial bed level, resulting from a given wave/water level event.

Most prediction methods seek simply to express this scour depth as a function of wave conditions (usually defined by some wave height, H, and wave period, T), water depth at the structure, h_s, and sea wall geometry. Rarely is any account taken of beach sediment size or composition, or of angled wave attack. As a consequence of this latter omission, the prediction methods are almost all limited to wave-induced scour with no allowance for longshore current effects.

The currently available prediction methods for wave induced toe scour may be divided into four categories:

- Rule-of-thumb approaches as typified by the Shore Protection Manual
- Generalised conclusions based upon Japanese research work
- Semi-empirical equations and design graphs derived from hydraulic model tests
- Simple morphodynamic models

Readers should note however that few, if any, of these prediction methods have been fully validated against field data.

Sand beaches

Most of the prediction methods suggested for toe scour apply only to sand beaches. This is somewhat ironic given that it is no sand]y beaches, particularly those with complex topography and seawall/revetment alignments, that one would expect longshore currents to be most influential in determining scour patterns and depths. Consequently the application of two dimensional prediction methodologies may considerably under-estimate the complexity of most natural situations. Nevertheless the methods are summarised in this subsubsection under the classification suggested previously.

(A) Rules-of-thumb

In common with many other researchers the SPM suggests that "the maximum depth of a scour through below the natural bed is about equal to the height of the maximum unbroken wave that can be supported by the original depth of water at the toe of the structure". Subsequent calculations, however, reveal that the orbital velocities at the bottom of the scour hole are still substantially higher than they would have been at the original beach level without the wall. This applies no matter which wave theory is used to perform the calculations, and as such suggests that this rule-of-thumb may under-estimate the actual scour depths.

Dean proposed that for the 2-dimensional situation, with conditions conducive to the formation of a longshore bar, the volume of scour immediately fronting a sea wall will be less than or equal to that volume of material which would have been provided from behind the wall, had the wall not been present. It should be noted, however, that this hypothesis is unproven and is difficult to apply not least because it requires the designer to accurately determine beach profiles for a given material size and incident wave conditions/direction, prior to calculating the volumes of material eroded and hence the depth of scour. At present the only means of accurately determining these profiles would be extensive and prolonged field measurements or a physical model.

(B) Generalised findings

Considerable research has been carried out in Japan using both field data and 2-dimensional physical model tests. Unfortunately the majority of these, and indeed until recent times the majority of physical model studies carried out throughout the world, failed to take any account of the need to correctly scale the sediment response in the modedl. Thus the results are often contradictory. Additionally many researchers both in Japan and the USA have defined the sea wall location, and hence the water depth, in terms of its position within a notional surf zone (ie using a parameter X/X_b, where X is horizontal distance from the shoreline that would exist without the wall, and X_b is the corresponding surf zone width).

Despite these difficulties a number of general conclusions can be drawn from the Japanese studies which may be useful for design purposes.

(a) For waves of steepness 0.02 to 0.04 the depth of scouring is approximately equal to the height of the incident unbroken wave (cf SPM rule-of-thumb).
(b) Maximum scour occurs when the sea wall is located at, or about, the plunge point of breaking waves or in an initial water depth of 1.5 H.
(c) The depth of scouring is directly proportional to the wave reflection coefficient for the sea wall (hence rock armoured walls exhibit less scouring than vertical walls).

(C) Semi-empirical equations

Herbich and Ko (1968) derived an empirical equation, from small scale model tests, for the prediction of general beach lowering (ie scour averaged over a distance of 15 feet, in the model, seawards of the wall) for both vertical and smooth sloping sea walls, in which the maximum distance-averaged scour depth is given in terms of the water depth at the wall, the wave height at the wall (incident plus reflected wave heights), horizontal velocity within boundary layer, under standing waves, and some mechanical properties of sediment.

Song and Schiller (1973) obtained a semi-logarithmic regression equation for the prediction of scour depths at the toe of a vertical wall again based on small scale model tests

$$S_d/H_0 = 1.94 + 0.57IN(X/X_b) + 0.72In(H/L) \tag{3.1}$$

where
S_d = the depth of toe scour
H_o = depth water wave height
X = horizontal distance from the original shoreline
X_b = horizontal distance from the original shoreline to the breaker point (i.e. original surf zone width)
H = deep water standing wave height
L = local wave length.

(D) Simple morpho-dynamic models

Steetzel (1987) presents details of a simple morpho-dynamic model for predicting beach changes near the foot of revetments. Although the model is based on a relatively limited range of test data it has been evaluated against near full scale model data and shown to given reasonable predictions of scour development.

Shingle beaches

Until recently very little work relating to toe scour on shingle beaches appears to have been undertaken. Some results are however available from a recently completed set of hydraulic model experiments set up to examine the behaviour of such beaches in front of vertical, sloping and rubble mound sea walls, for a range of water depths and wave conditions (Owen, 1989). A series of design graphs are presented for the prediction of dimensionless scour depth, S_d/H_s, as a function of wave steepness, S_m, and a dimensionless water depth h_w/H_s, where h_w is water depth at the wall. The graph for vertical walls in shown in Fig.3.15. Although this applies only to scour depths developed at 3000 waves, correction factors have been derived to enable other wave durations to be considered. For circumstances leading to the development of scour conditions in Fig.3.15 these factors are given as:

$$S_{dn}/S_{3000} = \begin{cases} 0.127 \ln N - 0.03 & \text{for} \quad h_s/H_s < 1.0 + 25 s_m \\ 0.149 \ln N - 0.21 & \text{for} \quad h_s/H_s > 1.0 + 25 S_m \end{cases} \tag{3.2}$$

where
$N =$ the number of waves in a storm or record and
S_{dn} = the scour depth equivalent to that number of waves.

3.1.4 *Scour at Groynes and Cross-Shore Structures*

Groynes suffer frequent failures and damage, particularly if they are permeable and constructed of timber piles. Major modes of failure are scouring due to rip current along groynes, accompanied by different eddies and secondary currents, concentrated mostly about the head of groyne and at the extremities of a groyne system. If a strong storm adds to these effects one may encounter partial damage of a groyne system. All scour modes can combine to remove sand from the vicinity of groynes and their constituents. An example is provided in Fig.3.16, fairly similar to that at the piled sea wall illustrated in Fig.3.13. Another example of failure of timber-pile groynes is given by Bakker et al. (1984). Thirty percent of the piles were damaged, primarily in the deep water section of the system.

Field observations of Dutch groynes provide the following summary given by Rakhorst (1984)

1. Erosion prevails over accretion on average coastline protected with groynes

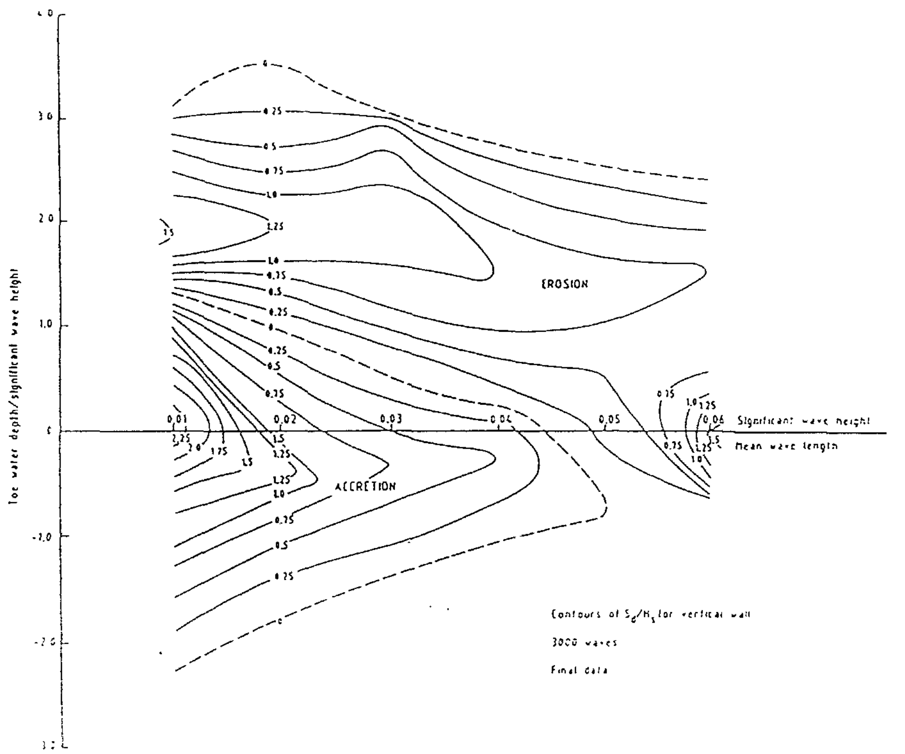


Figure 3.15. Scour Depth on Shingle Beach, Owen (1989).

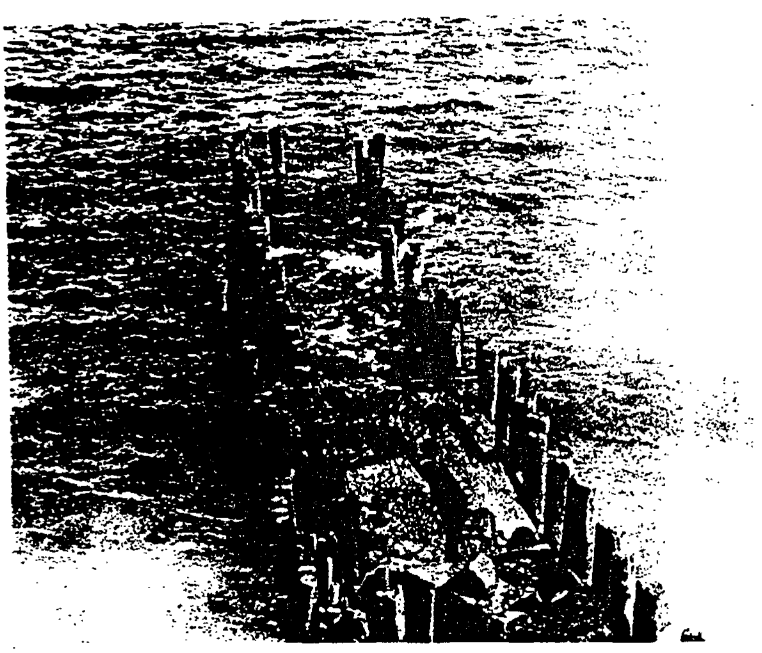


Figure 3.16. Removal of Sand Beneath Concrete Units Placed Between Two Rows of Groyne Piles, on Polish Coastline.

2. Erosion of shoreline after construction of groynes slows down from maximum 6...16 m/year (prior to construction) to 0...6 m/year (protected coast). It is noted that the erosion process is not arrested entirely

3. Erosion intensifies at extremites of groyne systems

4. The use of groyne system leaves uncertainties as to whether and how the groynes affect positively the coastline to be protected, as the protection effects depend heavily on proper dimensioning of groyne system.

Bakker et al (1984) stipulate that in the case of a groyne system in tidal seas the first check point should be an evaluation of the risk of washing out of piles by shoreward motion of tidal channels. One might face situations that even the maximum practical pile length will not be enough to guarantee constructional stability.

If this risk is acceptable, or if it can be eliminated (for instance by periodical sand supply), Bakker et al (1984) feel, that the use of pile screens deserves serious consideration:

> 'Based on theoretical and experimental evidence positive effects can be expected; especially in areas with a large tidal range, piles have better effect during a longer part of the tidal cycle than stone groynes with flat berms.
>
> Most of the negative experience can be attributed to construction errors, which in the future can be avoided. For instance, in Oostkapelle and Europort the piles were too short at the site where the washing-out occurred, because at these locations no heavy erosion had been foreseen. It seems to be of much importance that the screens are well incorporated in the dunes, in anticipation of a certain temporary removal of sand. At present, mussles are regularly removed from the screens.'

The depth of pile should make allowance for possible washing of soil and scouring, cf. Sec.4.2.

The effect of coastal structures on the transformation of shore has been studied in IBW PAN since 1978 (Pruszak & Tarnowska 1982). Mathematical modelling has been combined with laboratory investigations in a wave tank with movable bed, 60 x 40 x 0.9 m in size. The beach with a uniform slope of 4.5 percent consists of quartz sand having $D_{50} = 22$ mm. The wave fields produced by a regular-wave flap generator were measured together with water velocities, sediment transport rates and bed changes. Both parallel and normal (longshore and cross-shore) structures were tested (Tarnowska 1985, 1986).

The **cross-shore structures** have been represented by a single impermeable groyne with vertical walls, situated normally to the parallel (original) isobaths, and having a length L_B which was $L_B < X_b$, or $L_B > X_b$, where X_b stands for the width of the surf zone. The "deep water" angles of wave incidence were 15°, 30° or 45°.

It has been found (Tarnowska, 1985) that the length L_B is of paramount importance. Because of the littoral drift generated by breaking waves and of the wave interference, the erosion on the upwave side of the groyne was located at different places: at the head of the short structure ($L_B < X_b$), and about the centre or the deepest section of the long structure ($L_B > X_b$). The material eroded at the short groynes transported seawards of the groyne head line, while for the longer structures this material is recaptured somewhere at the heads either on the upwave or on the downwave side

The laboratory measurements are confirmed by some field observations. Since the same erosion patterns are to be expected in the laboratory and in the prototype, it is worthwhile to note that under various wind wave conditions encountered in the field the relative length $\frac{L_B}{X_b}$ varies with a storm climate. Hence, at a certain stage of a storm the structure recaptures the eroded material if the ratio $\frac{L_B}{X_b}$ is greater than unity, and subsequently loses in when L_B/X_b falls below unity.

Rip currents are known to appear along groynes and cross-shore structures and to create erosion problems. These problems become even more serious about the groyne head where the rip current meets with longshore currents generated by waves or other coastal factors. Quite often the "live cross-section" of the littoral drift shrinks considerably due to the seaward protrusion of groynes so that the scour at the heads may reach alarming proporties.

3.1.5 *Local Erosion at Single Piles and Cylinders*

The dynamic equilibrium of sea bed is disturbed if piles or any other structural units appear, so that changes in currents and waves about this unit induce sea bed changes. The material eroded is deposited somewhere on the downdrift side of the obstacle. The disturbance gives rise to erosion processes which stabilize after a certain time, and a new dynamic equilibrium is reached.

The mechanism of scouring about piles has been analysed in many studies but is far from being uniquely defined. Many controversis arise on the rate and magnitude of scour due to various hydrodynamic factors. Eadie and Herbich (1986) conclude that random waves and currents produce some 10 % higher changes than a current alone. Quite different opinion is represented by Nieroda and Tsuchiya (1988). The latter investigators assume that the erosion due to combined waves and current is lower than that generated by permanent currents. Kawata and Tsuchiya attribute this phenomenon to strong vertical eddies in the vicinity of pile. These eddies lift the sea-bed material and bring it in suspension; the suspended grains are removed from the neighbourhood of pile. Superposition of waves and currents damps down the eddies and decreases the local scour about pile. Similar opinion is represented by Breusers (1972) who analysed the laboratory tests of the 60-ties carried out by Abou-Seida, Palmer and a group of researches in the Delft Laboratory. The controversy is explained, at least partially, by Bijker and Brujn (1988). The latter assume that wave breaking is responsible for the phenomena discussed. If non-breaking waves superimpose on steady current, the scour about pile is smaller than that caused by current alone. This in agreement with the results of Nieroda and Dalton or Kawata and Tsuchiya. Combination of breaking waves and currents increases the local scour, in agreement with Eadie and Herbich.

The magnitude of local scour depends on sediment transport rate and the intensity of waves and currents. If currents combine with non-breaking waves, the sediment transport is four times greater than for wave alone, which points to the importance of currents. The sediment transport due to currents and breaking waves is claimed ten times greater than that caused by currents, Bijker et al. (1988). Waves prove most important and they control significantly the local scour about obstacles.

Prediction of local scour may be based on the above studies. Eadie and Herbich (1986) suppose that the shape of scour hole is circular, symmetric and with slopes close to natural friction angle. Maximum scour is encountered on the downdrift side about 40° to the left or right of the wave ray at the distance about 0.2 D, where D stands for pile diameter. Kawata and Tsuchiya (1988) localized the incipient scour on the downstream side, deflected by 45°, thus almost exactly at the place where subsequent maximum scour occurs.

The local scour is a function of many parameters, among which one may enumerate depth of water h, significant wave height $H_{1/3}$, velocities of wave- current field u and v, pile diameter D and sediment and water properties d, ρ_s, ρ, ν.

The effect of these factors is diversified. Breusers (1972) concludes that the effect of depth h is insignificant for $\frac{h}{D} > 2$. The depth of local scour increases with u and v up to a certain time, and depends simultaneously on the ratio of u and v. The effect of grain diameter d is negligible for $3mm > d > 0.2mm$. The depth of scour k_e is shown by Breusers to the proportional to the pile diameter D.

Dimensional analysis of Eadie and Herbich (1986) shows the following general dependence of the depth of scour k_e:

$$k_e/h = f(k_e/H_{1/3}; k_e/D; k_e/d_{50}; N_{RP}; u/\sqrt{gh}; u/\sqrt{gD}; u/\sqrt{2k_e g}; N_s) \quad (3.3)$$

in which

$$N_{RP} = \frac{uD}{(k_e/D)(\nu/k_e u)\nu} \quad (3.4)$$

$$N_s = \frac{u}{\sqrt{g d_{50}(\rho_s - \rho)/\rho}} \quad (3.5)$$

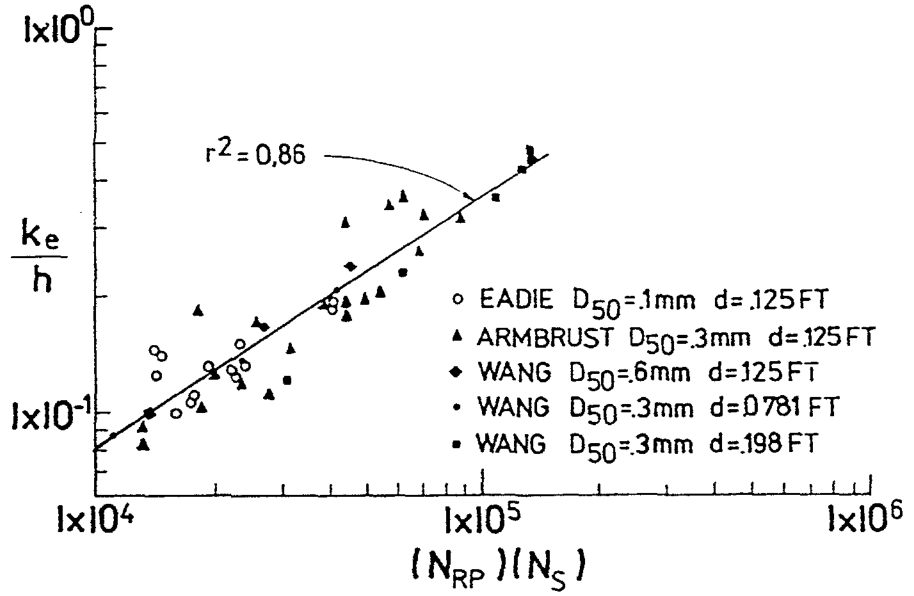


Figure 3.17. Scour at Piles due to current, Eadie & Herbich (1986).

The tests performed for currents and random waves indicate that sedimentation number N_s and Reynolds number N_{RP} are most important (Fig.3.17). In this case the above equation reads:

$$k_e/h = 1.941 \times 10^{-4}[N_{RP}N_s]^{0.654} \tag{3.6}$$

Scour depth k_e depends on N_{RP}, N_s and h. This is in agreement with earlier findings of Breusers (1972) who concluded that local scour about piles depends primarily on velocity of water, although originally the effect of h and d was deemed less important. The examples given by Breusers indicate $k_e/D = 1.4...6$. Specific values depend primarily on pile diameter and conditions of measurements; laboratory tests have given $k_s/D = 1.5...2$.

Local scour about piles may also be controlled by adjacent obstacles. If the spacing of piles in a groyne is smaller than six diameters $l < 6d$, the depth of scour k_e increases with decreasing l, compared with single pile situation.

The erosion process about pile is unsteady. It is fast in the initial phase and slows down with time. The scour due to combined waves and currents appears and evolves faster than that due to current alone. In may cases the erosion generated by current or tides achieves 90% of its extremum (equilibrium) after several days (weeks), Breusers (1972). The shape of the scour hole is similar no matter whether due to currents or combinations of waves and currents. This statement holds true for cases with prevailing currents. Differences may appear if wave-induced velocities are of the same order of magnitude, Eadie and Herbich (1986). The scour due to waves and currents is depicted in Fig.3.18.

3.1.6 *Local Erosion due to Ship Motion and Screw Race*

Motion of vessels in docks and ports, and foremost the operation of ship screws bring about very negative effects with regard to stability of bottom. Erosion and scour holes are common features in the vicinity of wharfs or across aproach channels. There are quite a few studies, among which one may cite Manual (1987), Fuehrer and Knebel (1988), Verhey (1983) or Robakiewicz (1989). We have used the latter to extract the following highlights, partly because the publication by Verhey (Delft Hydraulics No.303, 1983) is readily available to a western reader, contrary to the cited one.

Local scour caused by screw race is primarily controlled by water velocities. Two formulae may be proposed as a result of the theory of ideal propellers. For the configuration shown in Fig.3.19. one has

$$V = 1.6 \cdot n \cdot D \cdot \sqrt{K_T} \tag{3.7}$$

in which

n=screw revolutions (per second)
D=screw diameter (m)
K_T=screw thrust factor.

If the coefficient K_T is unknown or diffucult to determine one may assume

$$V = 0.95 \cdot n \cdot D \tag{3.8}$$

Average value $K_T = 0.35$ is proposed by Fuehrer and Knebel (1988).

Another formula stems from the power of vessel's driving engines:

$$V = C \cdot [\frac{P_b}{D^2}]^{1/3} \tag{3.9}$$

in which

P_d=engine power [kW]
C=empirical factor, which may vary from $C = 1.17$ for screw races to $C = 1.48$ for screw without races.

The average velocity on the axis of race jet may be proposed in the following experimental form found by Robakiewicz (1966) in his laboratory tests:

$$V_m = 1.515 \cdot V/\sqrt{x} \quad (m/s) \tag{3.10}$$

in which

V – velocity computed by Eq.3.8 or Eq.3.9
x – distance from screw to point in question.

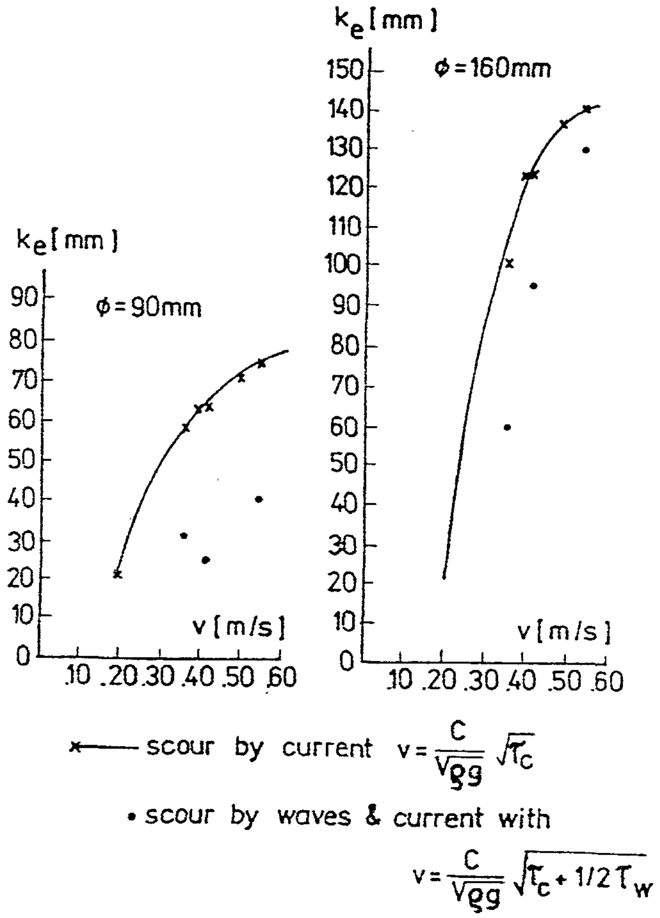


Figure 3.18. Scour at Pile due to waves and current, Eadie & Herbich (1986).

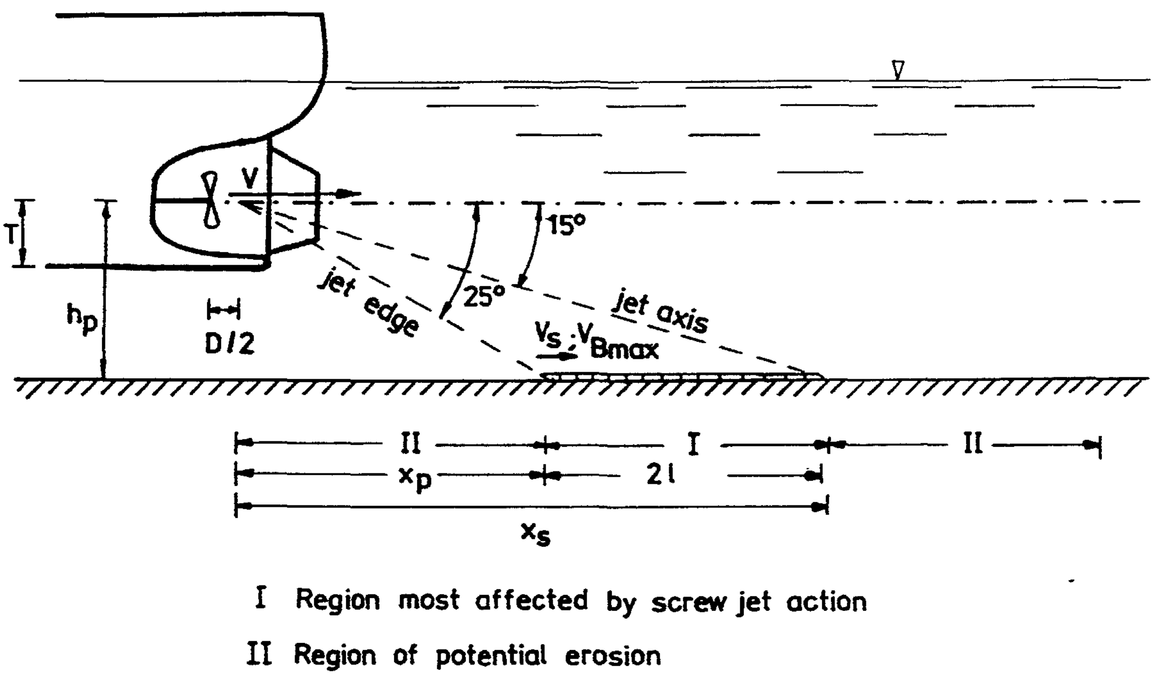


Figure 3.19. Propeller-Induced Scour.

The screw jet reaches bottom at the following closest distance:

$$X_p = h_p \cdot 2.1445 \quad [m] \tag{3.11}$$

while the horizontal distance between screw and the axis of screw jet reads

$$X_s = h_p \cdot 3.7321; \quad [m] \tag{3.12}$$

in which
h_p – elevation of screw level above bottom in metres.

The area between x_p and x_s is most vulnerable to the effect of the screw jet. Maximum velocities generated by screw are obviously most hazardous. These velocities may be computed by the formula proposed by Fuehrer and Roemisch:

$$V_{Bmax} = VE\frac{d}{h_p}; \quad (m/s) \tag{3.13}$$

The velocity V in Eq.3.13 is computed from Eq.3.8 or Eq.3.9. The experimental factor E reads 0.71 for ships with rudders located centrally with regard to screw while E = 0.42 is suggested for ships with other rudders or without them at all; E =0.25 holds for screws in races.

Knowing the water velocities generated by ship screw one may determine the erosion processes and prevent the negative effects.

Anti-scour concepts may be divided into two groups. The first group of designers tends to increase the depth of water at walls, while the other group opts for anti-scour measures in the proper sense. The selection of passive or active measures must be based on economic and technological factors.

Increasing depth of water at wall decreases maximum velocity at bottom. This is caused by a higher distance between screw and bottom and by the turbulent entrainment in the jet. The following reduction of maximum velocity and depth of scour is estimated due to increase of water depth.

Increase in water depth by	30%	60%	100%
Reduction in maximum velocity	50%	75%	80%
Reduction in depth scour by	30%	60%	80-90%.

In the design of antiscour measures one may determine the minimum depth of water for which a revetment or blanket will be subject to erosion by a given velocity of water V_{Bmax}. Robakiewicz (1989) proposes the experimental formulae by Führer and Römisch:

$$h_m = \frac{\frac{V_{bmax}}{B^2}}{9.81 \times 1.65} \quad [m] \tag{3.14}$$

in which
B = 1.25 for ships with central rudder
B = 0.90 for ships with other rudders.

For example, for a municipal ferry the depth of water at wall should be $h_m = 9\ m$ in order to mimimize the cost of anti-scour measures. Determination of adequate depth of water for sea going ferries is more difficult. Therefore it is more reasonable to provide appropriate bed protection measures in this case.

General principles of bed protection from screw jet are given by Manual 1987. The length of the protected bottom segment should be equivalent to the section subject to screw jet. It may read

$$2l = 2h_p(cot15° - cot25°) \cong 3.2h_p \tag{3.15}$$

The revetment should have proper size, and must be arranged on a fascine mattress or geotextile with a filter subgrade. For ships at quays the width of protected section should be at least equal to the width of ship. The parallel band, having a width equal to 50% of the protected width, may be weaker than the primary band.

3.1.7 *Erosion at Underwater Pipelines*

The pipelines in the marine environment are subject to strong waves and currents. Their safety is endangered not only by the forces induced but also by the scour generated in their vicinity. We shall not dwell on many aspects of these phenomena but rather confine ourselves to the problem of scour itself, which may have many common features with other erosion phenomena in the coastal zone.

One may distinguish the following three types of local scour about pipelines (Fig.3.20): **tunnel erosion, lee erosion and luff erosion**. The first type of erosion is the primary mode of scour. Its magnitude determines the depth of scour with regard to the original bottom. The existence of this mode modifies the flow of water about pipeline and further increases the area of erosion.

Lee erosion occurs on the downdrift side of pipeline. This is associated with turbulence about pipeline and eddy shedding. The third type of erosion (luff) erosion occurs on the updrift side, only if tunnel erosion does not appear. If waves control the phenomenon, and not current, the tunnel erosion is the primary and exclusive mode of erosion.

We shall now focus attention on the cases of local scour generated by currents, wave and combination of currents and waves. The following factors control the phenomenon:

- mean velocity of V
- pipeline diameter D
- elevation pipeline above bed, d_o, related to depth of water h

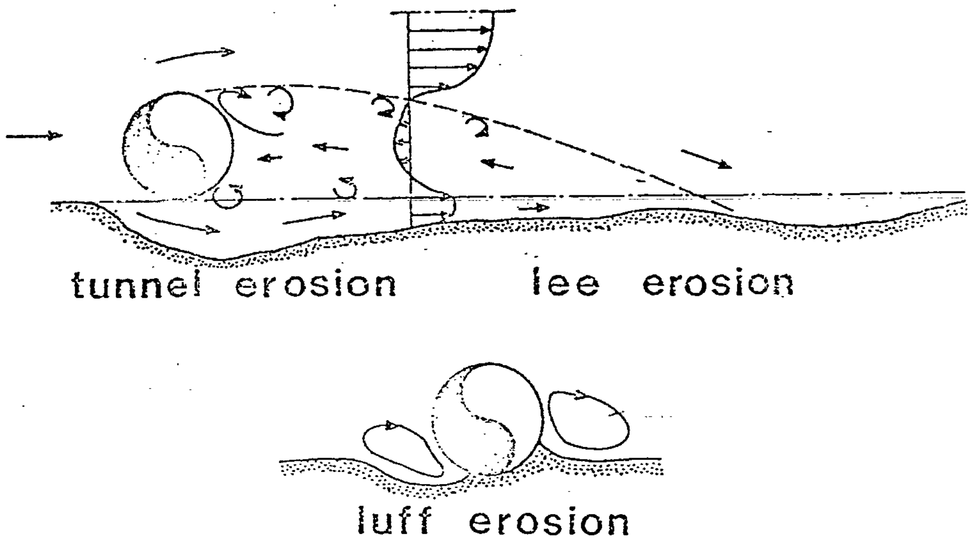


Figure 3.20. Three Types of Erosion at Pipeline, Leeuwestein (1989).

- grain diameter d
- wave height and period H, T, respectively.

These factors will appear in the subsequent formula for the transporting capacity of water and velocities about pipeline. In general it is assumed that local scour will be lower if the ratio$\frac{d_o}{\Psi}$ becomes negative.

Scour below pipeline will not grow if the ratio $\frac{\alpha_o}{\Psi}$ reaches -0.5 to -0.7. Maximum erosion will appear if the pipeline is at the level of natural bed, $\alpha_o = 0$.

Local scour caused by currents is controlled by the gradient of sediment transport rate in the vicinity of pipeline. It is obvious that ecrosion occurs if the sediment transport rate of pipeline is lower than its counterpart on the downdrift side. Equilibrium appears if the sediment transport rates on both sides of pipeline are equal, $Q_o = Q_p$.

The depth of scour k_e is the primary parameter to characterize the magnitude of scour. There are formulae which determine this parameter as a function of environmental conditions. One of them has been given by Kieldsen (1974):

$$k_e = 0.972[\frac{V^2}{2g}]^{1/5} D^{4/5} \tag{3.16}$$

This formula was used by a group of Dutch investigators who have modified it in the eigthies:

$$k_e = 0.929[\frac{V^2}{2g}]^{0.26} D^{0.78} d^{-0.04} \tag{3.17}$$

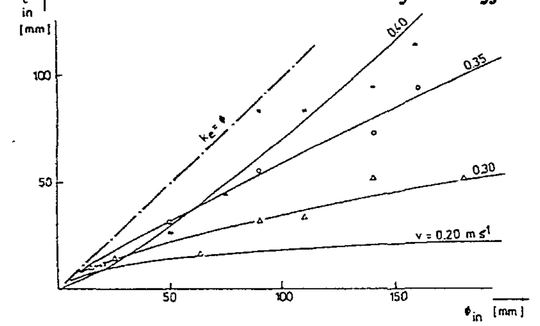


Figure 3.21. Scour Depth at Pipeline, Leeuwestein (1984).

The difference consists basically in inclusion of the grain diameter d. Both formulae show that depth of local scour is more sensitive to pipeline diameter D than to velocity of current V. The boundary conditions for which equations (20.7) and (20.8) hold true are summarized in table below.

Boundary conditions of hydraulic model tests given by various authors, Bijker and Leeuwestein (1984).

Investigator	h[m]	V[m/s]	d[μm]	$D[\mu m]$	$\frac{d_o}{\phi}$
Kjeldsen(1974)	0.43; 1.43	0.20...0.52	74	60...500	0; -
Ast and de Boer(1973)	0.21; 0.26	0.29...0.65	220	49...88	+;0;-
Jansen(1981)	0.36; 0.38	0.10...0.25	150	40...50	+;0;-
Meerendonk et al(1981)	0.31	0.17...0.25	150	50	+;0;-
Delft HL (1982)	0.38;0.40	0.26...0.58	90; 170	19...75	+;0;-

The plus sign denotes local elevation of pipeline above bed, while the minus sign corresponds to buried pipeline.

The graphical dependence of the depth of scour h_e on pipeline diameter D and velocity V found in Delft is shown in Fig.3.21 borrowed from Leeuwestein (1984). The straight line $k_e = D$ determines the maximum scour below pipeline. The limit quantities $k_e = D$ were obtained in many laboratory studies.

Respective formulae for field conditions are less available. The assessment of local scour in prototype conditions is usually done by extension of laboratory tests. The scaling may be done as suggested by Leeuwestein (1984).

The Delft experiments show that the depth of local scour k_e given might underestimate the real scour under storm conditions.

The duration of scouring about pipeline depends considerably on the type of erosion. In the case of lee erosion its duration, that is the time from incipient erosion to the state of equilibrium, is long as the intensity of eddies behind popeline is very high and steady. The tunnel erosion may be described by logarithmic function, Bijker.at al (1984).

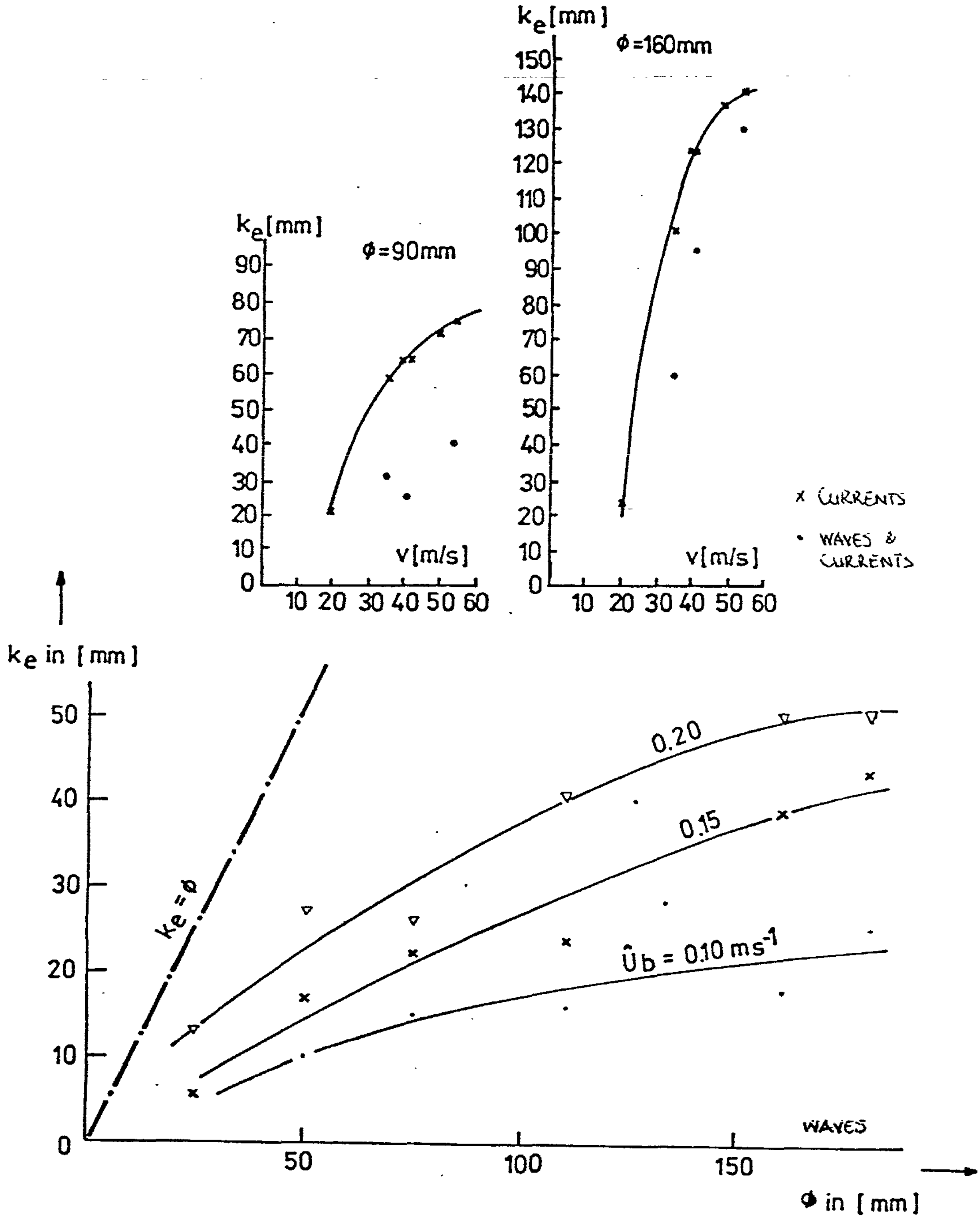


Figure 3.22. Leeuwestein's Graph for Erosion at Pipeline.

Local scour due to waves deviates from that for currents. The time scale of wave-induced velocities is much shorter than that of steady currents. As the velocities are two-directional, the mechanisms are obviously different. Laboratory tests have shown that the depth of scour due to waves is greater than that caused by currents. The maximum erosion occurs beneath the lowermost (central) point of pipeline and is symmetric. The depth of scour and stabilization time depend on the amplitude of orbital velocities at bed, A_B and wave period T. If the orbital velocities about pipeline, u, are greater than the critical scour erosion velocity $u > U_{cr}$ and if a is greater than x_{cr}, the erosion will continue. In the opposite case erosion will cease. The quantity x_{cr} denotes the distance from the centre of pipeline to the point with the critical velocity $u = u_{cr}$. Experiments show that the pipeline diameter is less important than in the case of current-induced scour.

In the case of waves one may propose the following general function for the depth of scour beneath pipeline:

$$k_e = f(D, u_b, T, d) \tag{3.18}$$

The analytical form for this general function has not been determined, although it is intensively studied, mostly in laboratory.

Leeuwestein (1984) proposes a graphical form for Eq.3.18, cf. Fig.3.22. The graph and adequate selection of scale may determine scour in prototype conditions.

Local scour caused by a combination of waves and currents is the most frequent case in prototype conditions. Waves may approach at different angles, and the resultant sediment transport rate will also vary significantly. The Delft investigations point to increase in depth of scour if strong waves and weak currents intervene. The opposite superposition, that is weak waves and strong currents bring about reduction in erosion, compared with the case of sole current. The presence of waves in strong currents gives rise to erosion of sediment from beneath pipeline and repeated accretion in the erosional niche.

Laboratory investigations show that the combination of waves and currents for identical shear stresses τ (compared with exclusive currents) brings about smaller erosion.

In general, combination of waves and currents produces scour depth contained between the upper value given by currents and lower value given by wave case. Bijker (1984) assumes that the lower boundary of local scour is about 30% of the upper limit.

3.2 FAILURE MODES OF COASTAL STRUCTURES

3.2.1 *General*

Damage can be defined as a certain change in the state of structures. In turn, the state of a structure is described by the following three characteristics of the structure:

(1) external boundaries ;
(2) configuration and cross-sections;
(3) integrity of constituents.

Changes (1) and (2) often correspond to a certain physical loss or displacement of material of the structure and its surroundings. This in turn may cause a certain loss of functions. This loss can often be observed or measured easily by setting up an efficient monitoring programme.

In practice there may be a gradual loss of functions with increasing damage. Therefore **failure** can be regarded as to correspond to a state of some **ultimate** degree of damage, to be reached after a certain time and that can be linked to functional requirements of strength or serviceability.

Some failure modes can be accepted to occur repeatedly up to a certain limit (eg. the displacement of an armour stone in mound). For other modes not even a single occurrence can be accepted (eg. liquefaction of the subsoil under a breakwater).

Once a failure has occurred, repeated failure becomes likely. Respective occurrence of one mechanism leads to increasing damage (e.g.loss of filter material in a revetment). The frequency of repetition determines the evolution of damage. In general, the degree of damage increases with the loading level.

For the sake of clarity one should discern a **failure mode** from a **failure factor**. The former is understood as a certain manner in which a structure transforms mechanically (i.e. passes from one state to another) while the latter is a physical, chemical, biological, or any other phenomenon leading to the above transformation. In addition, *failure mechanism* will be a notion used for description of transient processes while *failure mode* will denote the ultimate transition to a clear-cut state of damaged structure.

An important question is whether, at a certain constant loading level, the rate of damage will decrease or increase with time or whether the failure frequency will decrease or increase. In the first case a practical (asymptotic) limiting damage level can often be defined, which can be accounted for in the design process. The same may still hold for the second case, but often these cases serve extra attention. These are also cases with **progressive damage development**. For most structures this cannot be accepted when uninterrupted functioning is needed.

Progressive damage is usually considered to indicate failure. When a sudden, clear-cut increase in damage cannot be observed, the point of failure is taken arbitrarily, for a particular set of conditions denoting, by a certain definition, the critical degree of damage.

Progressive damage due to a partial failure mechanism may finally lead to the initiation of other partial failure mechanisms or to total failure.

In the structural design, a respective probability must be determined for each failure mechanism. By using the failure tree and the probabilities of the various subsystems the ultimate probability of failure for the entire system can be found.

One should note in passing that temporary failure is not always a detrimental property but instead can be turned into success. This is well illustrated by the berm breakwater concept (Meer, van der, 1987). The berm breakwater is an unconventional[1] design, in which displacement of armour stones in the first stage of its lifetime is accepted (and indeed intended). The original cross-section of a berm breakwater consists of a lower slope, a horizontal berm (just above high water) and an upper slope. After the displacement of blocks in the stage of profile formation the revetment becomes more or less statically stable.

In the wake of the above categorization of the state of coastal structures and measures, one can put **failure modes** in three distinct **classes** referring to and encompassing

1. near-field **neighbourhood** of structure
2. structure as a **whole**
3. **interior and constituents** of structure.

The **first class** includes, by and large, local erosion in the immediate vicinity of the structure, or local deformations of the interface of the structure and its ambient environment. One can distiguish the phenomena of **scour, undermining, outflanking, surface run-off erosion, local sliding**, etc.

One often puts among failure modes particular states of 'boundary conditions', that is the behaviour of water and the environment surrounding the structure. These regimes include **overtopping** by waves and surges, **ice impact, ship collision, surcharge loads, dredging effects**, etc. Such an approach, although justified to a certain extent in view of the analogy to deformations of soil (cf. scour etc.), is nevertheless *rejected in this document*. It is our feeling that, for instance, overtopping is not a failure mode proper but instead an **external failure factor** which

[1]The idea of unconventional design is also strongly built into Khomitskiy's (1983) concept of **analog structures**. The latter are coastal engineering measures which simulate as close as possible the natural response of beach and shore to the dynamic action of the nearshore environment. The measures incorporate natural features as "nuclei of shore protection" which are harnessed to intensify the defense characteristics. Hence a submerged breakwater as a counterpart of natural underwater bar or reef, etc. The concept is illustrated more fully in subsequent chapters, particularly in Chapter 6 for other methods of unconventional design

might bring about damage or eventual failure of structure through any type of failure mechanism, e.g. surface run-off erosion.

Scour and **local erosion** are discussed in Section 3.1. As noted elsewhere, sometimes it is difficult to make clear distinction between near-field and far-field phenomena. We hereby regard local erosion a phenomenon caused undoubtedly by a structure in an area measured in characteristic dimensions of the structure. If this area is larger one speaks of far-field erosion, or accretion. The far-field phenomena have been covered in Chapter 2, although a certain overlapping with this chapter has appeared unavoidable.

The **second class** consists of macroscopic movements of structures, such as **overall linear and angular deformations**, including **overturning or tilting**, **sliding**, **settlement**, **heave**, etc. The structure is deemed one entity subject to the same mode of failure in all parts. Local sliding, etc. caused by similar mechanisms are not considered within this class.

The **third class** includes internal failure mechanisms, primarily microscopic, which control the behaviour of constituents or units of structures, and not entire structures. The stability of these components depends on resisting the influence of local loads (or failure factors in our terms). Local stability problems may ultimately give rise to the failure of the structure as a whole.

Strictly speaking, the number of local failure modes is limited to local linear and angular deformations, by analogy to the second class. However, since the variety of 'local' zones and their combinations is infinite, a strict categorization of local failure modes is neither practical nor even possible. One may nonetheless distinguish numerous failure mechanisms which determine the internal states of structures. The examples of **geotechnical** failure mechanisms are:

local sliding, local bearing failure, piping, migration, liquefaction, pumping, settlement (with subclasses of consolidation, compression, migration, shrinkage, loss of apparent cohesion), heave, suffosion, consolidation and cyclic compaction, seepage and groundwater flow .

The latter two can also be placed among **hydraulic** failure mechanisms, together with **uplift, movement of armour units (including turning, lifting and removal)** etc.

The earlier remark that one failure mechanism can bring about both overall and local failure mode becomes obvious in the ligth of the above examples.

The above categorisation of failure modes and factors is outlined in Fig.3.23.

3.2.2 *Outline of Failure Modes*

Some failure modes are typical of all types of structures, while some other create particular problems in certain specific coastal defence measures. We begin with a

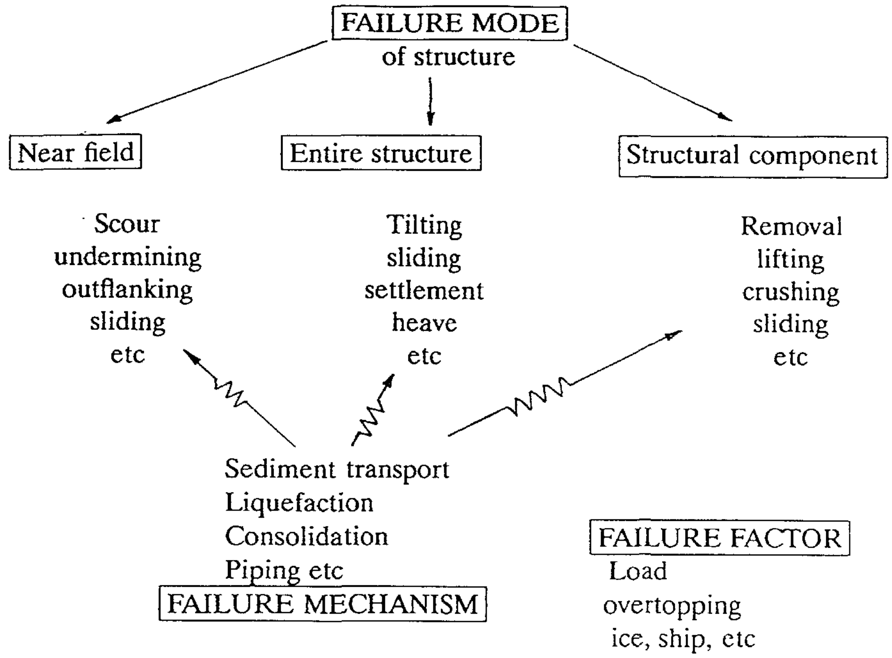


Figure 3.23. Schematisation of Failure Modes and Factors.

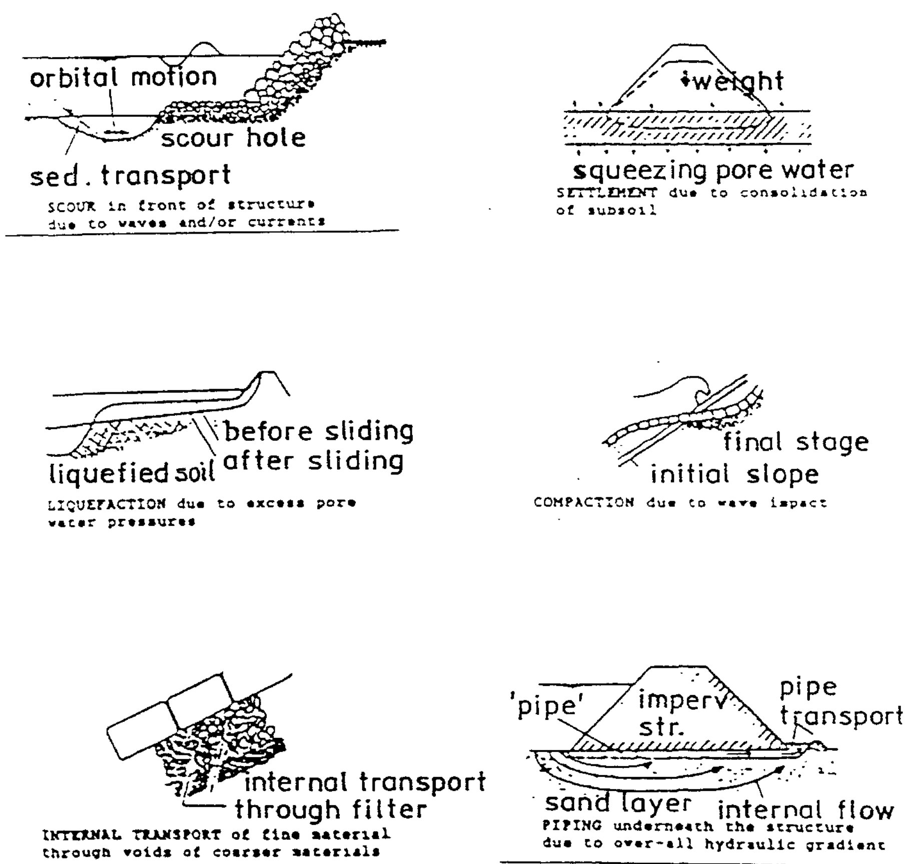


Figure 3.24. Subsoil Failure Modes.

rough qualitative overview of some general damage phenomena.

Geotechnical Failure Modes

Examples of failure mechanisms are illustrated in Fig.3.24.

Bank or slope stability can be considered in terms of

a. overall stability - the stability of the bank as a whole

b. local stability - the stability of elements of the structure in resisting the influence of local, hydraulic or other factors; local stability problems may ultimately lead to failure of the bank as a whole.

(a) Overal Stability Failure Mechanisms

Slip circles and deep sliding- this is a conventional soil mechanics stability problem: influence of rapid draw-down should be considered. Pre-existing slip planes within the soil, or lenses and bands of weaker material can have a significant effect on bank stability. Side investigations should be planned to enable such situations to be identified.

(b) Local Stability Failure Mechanisms

Local sliding - local shear failure parallel to the slope within the soil mass or at the soil-structure interface.

Local bearing failure - local shear failure in the soil in planes not parallel to the slope.

Scour - removal of soil particles from the soil-water interface by current or wave induced shear forces within the coastal environment: possibly in combination with hydraulic gradient forces or by rainfall run-off above the water line.

Piping - a form of concentrated seepage through a section of soil which is either more permeable than its surrounding or is subject to a particularly high hydraulic gradient. Concentration of flow may lead to transport of soil particles and loss by regressive erosion. Once transport is established, the hydraulic gradient will increase and transport more particles resulting in an erosion pipe.

Migration - transport of material within the soil mass beneath the structure. Migration may be parallel to the shore causing slumping and an S-profile or through the structure in which case material is lost from the shore. A special case is suffusion, which is the transport of the finest particles from between a soil skeleton fromed by coarser material.

Liquefaction - complete loss of grain to grain contact by an increase in pore water pressure or by shock loading of a loosely compacted granular soil. Consequent loss of effective stress results in a zero shear strength and the soil behaving as a liquid.

Pumping - extrusion of particles under shock loading or impact in connection with liquefaction at an interface.

Settlement - deformation due to reduction in volume of soil. settlement may be caused by the following mechanisms:

- Consolidation -squeezing out of pore water from a cohesive soil
- Compression - more dense packing of particles due to extra loading
- Migration etc. - loss of material from (within) the bank
- Shrinkage - drying out of saturated cohesive soil
- Loss of apparent cohesion - some sandy soils exhibit apparent cohesion when dry(when saturated, this cohesion is lost, which results in settlement).

Heave - deformation due to increase in volume of soil. Heave may be caused by:

- Frost - expansion of ice crystals within the soil mass on freezing
- Swelling - certain soils take up moisture and swell after removal of confining pressures.

Fig.3.25 illustrates various failure mechanisms.

Many of the local stability failure mechanisms can be overcome by good construction methods: proper compaction of the subsoil can considerably reduce the risks of piping, migration and liquefaction.

Uplift

If the filter and cover layer of a revetment are of relatively low permeability compared with the subsoil, they will be subjected to uplift forces caused by the outflowing water. If the uplift forces are not fully counteracted by the selfweight of the filter and cover layer then the consequences of uplift pressures are severe. Firstly the effective weight of the cover layer is reduced, reducing friction between it and the underlying layer, so the sliding of the cover layer becomes possible.

Secondly, although the effective normal stresses within the filter layers and subsoil are reduced, the shear stresses (due to self weight) remain the same hence causing the possibility of S-shaped deformations of the embankement as shown in Fig.3.26. The determination of uplift pressure is therefore an important part of the design procedure for structure stability.

The following parameters are relevant to the uplift pressure:

H = wave height
T = wave period
k_{sub} = permeability of sublayer
k_c = permeability of cover layer

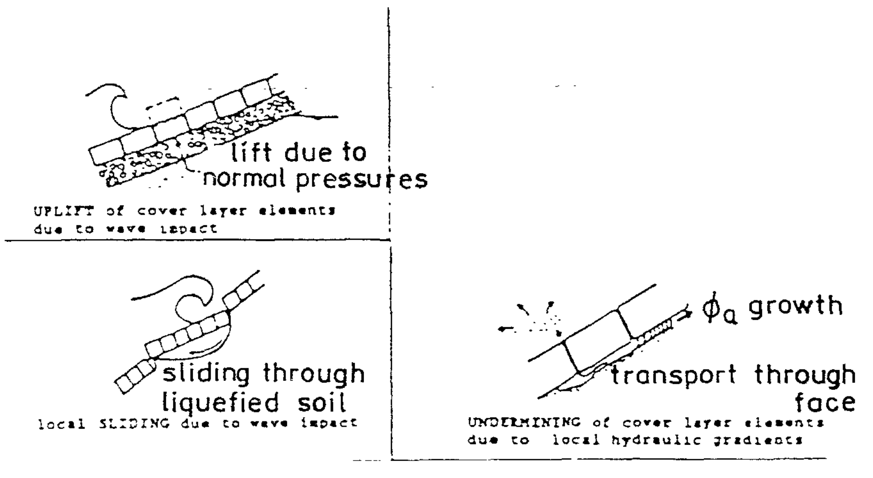


Figure 3.25. Examples of Various Failure Mechanisms.

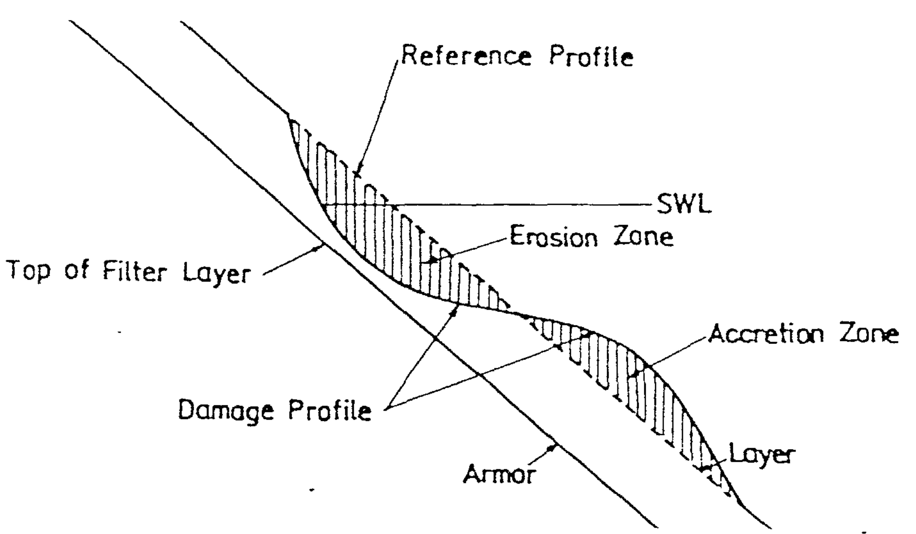


Figure 3.26: S-shaped Failure Profile of Embankment.

D = thickness of cover layer
α slope angle of embankement with respect to the horizontal
R = elevation of phreatic line in the sublayer beneath the revetment due to wave run up.

Determination of uplift pressures is rather difficult in view of the number of parameters which are involved [15]. As a guideline no uplift pressure will ocur if the permeability of each layer beneath and including the coverlayer is 20 to 50 times greater than the permeability of the underlayers (including the subsoil). If uplift pressures are to be further assessed it may be necessary to perform model tests or construct test embankments.

3.2.3 *Failure Factors versus Failure Modes*

Only few of the **failure modes** and **mechanisms** described above can be expressed well in terms of the **failure factors**, which give rise to damage, Here we will analyse some examples in which the term *loading* is used as a synonim for failure factor and the term *response* denotes damage or failure.

It should be remembered that failure mechanisms are often dependent on each other. The performance of one mechanism may initiate, either directly or indirectly, another mechanism.

Loading: weight.
Loading *parameters (principal)*: specific density of materials.
Loading *parameters (secondary)*: pore (water) pressures, time.
System characteristics: soil compressibility and -permeability, thickness of compressible layers.

Response: lowering of structure crest and horizontal deformations.

The weight of a structure causes an increase of the total soil stress of the subsoil. The subsoil responds both directly, through compaction of its structure (initial or primary settlement) and indirectly (secondary settlement). The latter is mainly caused by consolidation. The rate of this secondary settlement is therefore dependent on the drainage of the low-permeability layers in the subsoil. Settlement decreases the structures capability of withstanding high water levels and uprushing waves. For a revetment, differences in settlement lead to an uneven revetment surface. This makes the stones of the armour layer more susceptible to hydraulic loading.

The way in which the loadings act on the rock structure can best be described in the general schematization given in Fig.1.33. In this schematization there are five basic elements:

- boundary conditions;
- external transfer functions;

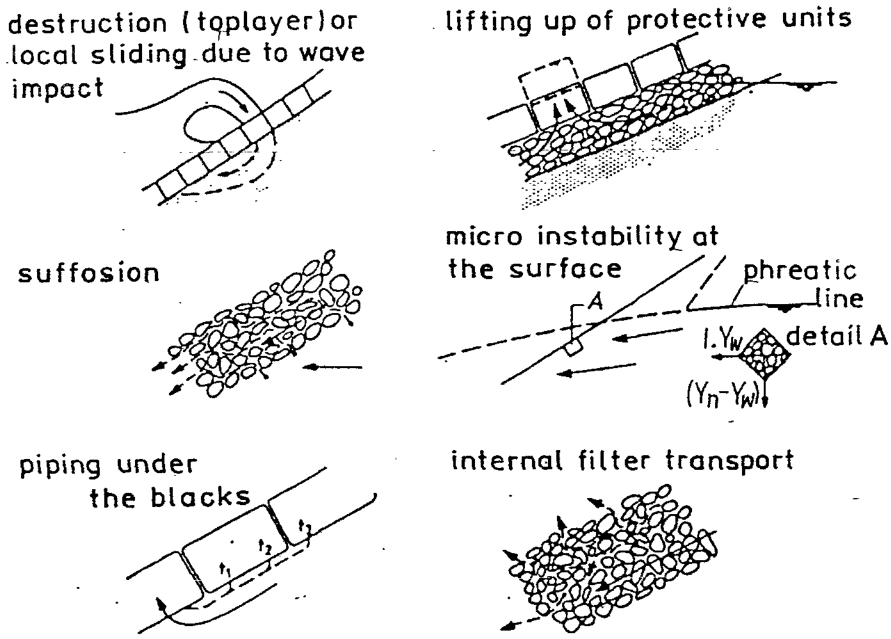


Figure 3.27. Failure Modes and Mechanisms in Revetments.

- loadings (forces, pressures);
- internal or system transfer functions;
- response of system (displacements).

This schematization or response model applies to the three categories of loading and to each element (e.g. stones and sublayers of revetment) or to the structure as a whole. In this model failure occurs if, beyond the failure load S_F the response increases rapidly at increasing loading. Beyond this failure point usually another system becomes dominating. An example is the plastic deformation of a steel wire due to a tensile force after that the linear elastic strain rate has been exceeded. Reaching of the elastic limit can be considered as the point of failure and the large plastic deformation then determines the failure mechanisms or failure mode.

In general, failure mechanisms are named after their consequent displacements or movements. Failure modes or failure mechanisms are thus characterized by a relatively large increase of response due to a minor increase in loading.

For *revetments* there exist many failure mechanisms each corresponding to a loading from one of the loading categories mentioned above. An overview of some principal failure mechanisms for revetments, together with the responsible loading is given in Fig.3.27 and in the subsequent description.

Failure Mode : Movement of armour units

Loading: waves, currents.
Loading *parameters (principal)*: wave height/-period, velocity.
Loading *parameters (secondary)*: time, angle of incidence.
System characteristics: stone diameter/-density, permeability.
Response: rocking, sliding, lifting, rolling.

Waves and currents determine the lift and drag forces acting on the stones of the cover layer of a revetment. The inertial forces are also determined by the stone characteristics. The stone weight, but also forces due to friction and interlocking with other stones, are the stabilizing forces. The dynamic (loss of) balance of all these forces may result in a great variety of the above mentioned stone movements. These responses may be accounted for in the design, but may as well initiate other failure modes. Most evident is damage of the filter layer.

Failure Mode: Migration of sublayer - or core material
Loading: hydraulic gradients, internal flow.
Loading *parameters (principal)*: water pressures/-velocities.
Loading *parameters (secondary)*:
System characteristics: layer permeabilities/-thicknesses, grain size.
Response: material transport out of structure.

Due to a difference in water level or due to local (generation of) excess pore water pressure an internal flow may be established. When a certain critical hydraulic gradient and the corresponding flow velocities occur, the finer grains are transported out from the inner layers through the coarser material of the upper layers. Often this transport can also easily pass the cover layer resulting in a loss of material from the filter layer and/or from the core.

Failure Mechanism: Piping
Loading: hydraulic gradient.
Loading *parameters (principal)*: water velocities/-pressures.
Loading *parameters (secondary)*:
System characteristics: grain size, "pipe" length.
Response: material transport out of structure.

Piping is convergent migration of material through internal channels or "pipes". Deposition of material can often be observed if these internal channels reach a surface, which is exposed to the air. Typical values for the critical gradient can be expressed in terms of the quotient of a difference in water level or head and a characteristic length of a (part of) the structure.

Failure Mode: Sliding of structure
Loading: weight of structure.
Loading *parameters (principal)*: density of building materials.
Loading *parameters (secondary)*: pore water pressures, slope angle.
System characteristics: soil friction angle (cohesion)
Response: sliding of (a significant part of) structure including subsoil, collapse.

From the combination of loading (weight of cover layer and core material) and geometry of the cross section (slope angles) a distribution of the total soil stress results. Together with the actual pore water pressures these soil stresses determine the effective grain sgtresses in the core of the structure and in the subsoil. By using the internal friction factor the grain shear stresses are obtained. For the given loading and geometry of the structure the sliding surface is determined by the surface with the minimum value of the safety ratio.

The safety ratio is defined as the ratio between the moments of the resulting shear forces along a possible sliding surface and the resulting loading forces respectively.

Failure Mode: Sliding of cover layer

Loading: weight, waves

Loading *parameters (principal)*: stone size/-density

Loading *parameters (secondary)*: wave height/-period, slope angle, pore pressures

System characteristics: friction angles of cover-/filter layer, permaebility of cover-/filter layer

Response: sliding of cover layer along the slope.

For the sliding of the cover layer in principle the same procedure should be applied as for the entire structure . However, a simplified analysis can be used by considering only the sliding surface determined by the interface between the cover - and the filter layers.

Due to the wave impact excess pore water pressures are generated just under the cover layer.

These excess pore water pressures reduce the effective grain stresses and shear stresses. Sliding of the cover layer occurs if the shear component of the weight of the cover layer along the slope can not be compensated by the effective grain shear in the filter layer.

Failure Mode: Scour

Loading : waves, currents

Loading *parameters (principal)*: orbital-/current velocity, turbulence intensity

Loading *parameters (secondary)*: wave period, angle of incidence

System characteristics: sediment grain size, structure slope, stone size

Response: degradation of seabed in front of structure.

Waves and currents cause resulting water movements near the seabed, which may generate a sediment transport. Interactions with the structure (wave reflection, generation of turbulence) may affect the natural sediment transports. Relative to the natural sediments most structures can be considered rigid and inerodable, although some may be permeable to sediment. These properties impose the structure boundary conditions to the transport processes.

Failure Mechanism: Liquefaction

Loading: pore water pressures, waves, earthquakes
Loading *parameters (principal)*: pore water pressure, wave height/- period, (relative) shear amplitude, acceleration, frequency
Loading *parameters (secondary)*: number of waves/loading cycles
System characteristics: thickness of compressible layers
Response: serious deformation of structure, collapse.

Liquefaction is caused when excess pore pressures are generated to such a degree that the effective grain stress and consequently also the effective shear resistance vanish. Under these circumstances the smallest external loading will be able to cause sliding or even a total collapse of the structure, which derived its stability from this soil. Pore water pressures are generated by cyclic loading of which some evident examples have been listed. The common element of these loadings are cyclic shear stresses, which, through changes in compaction, in fact generate the pore pressures. Additional dissipation of pore pressures is enabled by drainage.

In the design process, one is most interested in the ultimate limit state (U.L.S.) of a failure mechanism. This state is reached when the acting extreme loads are just balanced by the strength of the structure. If the ultimate limit state is exceeded, the structure will collapse or fail. The concept of the ultimate limit state is known from the elementary theory of the strength of materials[2].

In many cases, the variety of the processes involved cannot be described as yet. Therefore a "black box" approach is followed in which the relation between critical strength parameters, structural characteristics and hydraulic parameters are obtained empirically.

3.2.4 *Examples of Failure Modes*

Rubble-Mound Breakwaters

These modes of failure are instructive for both coastal and ocean engineers. Heavy damage often occurs when the armour is completely soaked or fluidised in wave uprush. The breakdown pattern therefore may be disastrous and capable of moving large masses.

Common reasons for breakdown of rubble mound breakwaters, whether the mound is composed of natural or artificial blocks, include (Bruun & Kjelstrup, 1983):

[2]This description is restricted to the stability of the front slope of revetment. Moreover, the instability as a result of hydrodynamical processes is taken into account only. Starting with the hydraulic input data (waves, water levels) and the description of the structure, external pressures on the seaward slope can be determined. Together with the internal characteristics of the structure (porosity of revetment and secondary layers) these pressures result in an internal flow field with respective internal pressures. The resulting load on the revetment has to be compared with the structural strength, which can be mobilized to resist these loads. If this strength is inadequate the revetment will deform and may ultimately fail

1. 'Knock-outs by plunging waves when

$$\xi_b = tan\alpha\sqrt{\frac{H_b}{L_0}} < 2.0 \;\; \& \;\; > 0.5 \tag{3.19}$$

2. Liftouts (by uprush-downrush) usually resulting from combinations of uprush and downrush and toe velocities in an arriving plunging wave.
3. Slides of the armour as a whole. This happens in particular at steep slopes, which are subjected to high waves of periods close to resonance (i.e. uprush-downrush period is close to wave period).'

The failure is caused by combinations of buoyancy, inertia and drag forces supported by the effect of hydrostatic pressure from the core of the breakwater. These forces all seem to reach their maximum value for lowest downrush which occurs at resonance or for:

$$\xi_b = tan\alpha\sqrt{\frac{H_b}{L_0}} \approx 2.0 \tag{3.20}$$

Experience, however, has shown that large single or double waves. 6.30) may be particularly dangerous. This has been observed directly in the field and some of the large failures of multilegged blocks may be attributed to the occurence of such waves or groups of waves.

4. 'Gradual breakdown or failure due to "fatigue". Fatigue starts with smaller movements of the blocks, which steadily increase, and by which the blocks are gradually moved out of intimate contact with neighboring units or from the first sublayer, and perhaps simultaneously suffer from tear and wear due to their rocking or bouncing around, thus impacting other blocks and causing damage. This process is particularly important for multilegged blocks, when such damage may be directly observed or "heard". Occurrence of resonance making the uprush/downrush per particular effects due to the constant rocking motion with specific friction and interknitting between blocks, and partly some other effects, cause structural ruptures due to bending stresses and other fatique forces.'

Other types of wave trains e.g. wave series with deep troughs , causing deep rundown, and therefore high downrush velocities as well as higher hydrostatic pressures from the water table in the core, are very dangerous. Natural rock is a compact mass and its resistance against movement is based on its weight and friction against other blocks. When useful weight is decreased due to buoyancy the resistance decreases to about half. The wave situation depicted therefore is very dangerous, because the slope meets wave No. 2 in submerged condition. It is the most dangerous of wave trains, hydrodynamically speaking, that determines

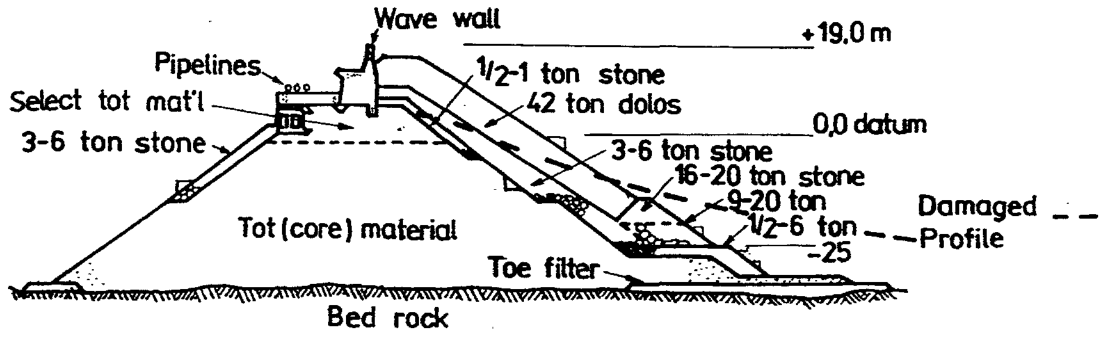


Figure 3.28. Sines *(Portugal)* Breakwater.

the stability or failure

Kjelstorp and Bruun (1983) provide three examples of serious destruction of rubble-mound breakwaters experienced after 1970 for the following breakwaters: one of rock (Akranes, Iceland), one of tetrapods (Arzew el Djedid, Algeria) and one of dolos (Sines, Portugal).

Profile of the Sines breakwater before the massive destruction is shown in Fig.3.28.

In 1978-1979 winter season the breakwater experienced 3 storms with the following wave heights and periods $H_b \approx 8...10$ m and $T_s \approx 16 - 19$ sec.

> This was followed in February 1978: $H_b \approx 9$ m and $T_s \approx 19$ s
> in December 1978: $H_b \approx 8 - 8.5$ m and $T_s \approx 16$ s
> in Febrary 1979: $H_b \sim 9$ m and $T_s \approx 19$ s.

These storms, and particularly the February 1978 one caused extensive damages. The main failures took place in what may be termed "mass slides" or "mass departures" spreading debris over a large section of the offshore bottom. The debris was heavily damaged.

The mode and character of the failure seems to indicate that the failure was mainly the result of a massive lift, movement and bouncing of the 42-ton armour blocks. Blocks however may have been damaged before the storm due to insufficient strength but as the armour layer probably became "a live mass" considerable damage undoubtedly happened during the storm and at the breakdown. The blocks apparently were too small and structurally speaking too weak. The toe structure probably was insufficient too, being very small. When the armour failed the massive wave screen on the top became exposed directly to wave attack which by reflection eroded the base of the screen so that it tumbled over.

Hence the following reasons of the failure seem to have combined:

a) Inadequate wave data and inadequate studies (as a consequence)
b) Blocks too light for slope
c) Blocks too weak, in structural terms
d) The mamoth wave screen proved to be impractical.

Barends, Kogel et al. (1983) also analysed the Sines case and concluded:

- The geotechnical stability of a breakwater under extreme wave loading conditions is strongly affected by the combined effect of wave seepage and wave impact forces both of a typical dynamic nature.

- In most cases instability will not occur as sudden instantaneous complete slope failure, but rather by a progressive deformation along the slopes, the rate of which depends on the magnitude and duration of the storm.

- The in-situ initial state (actual material properties, stress state and cross-sectional geometry) is of significant importance to the functioning of the structure under design storm conditions. Built-in devices to detect the initial state are recommended.

Another example of damaged mound is provided by the Akranes breakwater (Iceland) made of natural rock, and not of dolos.

It is often claimed that rock mound structures fail in a more "graceful way" than concrete block structures. The wording refers to the fact that failure of rock structures is a rather slow process by which the mound gradually loses blocks, normally by outwashes on the outer slope. Massive failures are not common with rock mound structures unless failures take places by heavy overwashes which rapidly flatten out the mound carrying crown blocks down on the outer slope and over on the inside of the mound. Another type of massive failure takes place when the mound structure is capped with a large screen in the form of a high vertical wall, perhaps sticking up several meters above the rock slope in fornt of it. So heavy uprushes cause erosion of the mound in front, perhaps lowering it considerably, thereby increasing the reflection from the wall, Kjelstrup and Bruun (1983), and in the case of the Akranes breakwater in Iceland the wave screen did not collapse because it was built on the top of a caisson but the mound in front suffered severe damages.

The Akranes breakwater is located in a relatively protected area, where storm waves occasionally may reach about 4-m H_s and $T_s = 14....16$ sec. A severe storm in December 1980 caused damages in an area where wave action concentrated. A movie taken when the storm was at its peak demonstrates the pressure of large "mountain waves", which swamped the breakwater entirely in massive uprushes. In the following deep downrushes large water masses under high pressure from water inside the breakwater poured out of the mound with detrimental effects on the stability already weakened by the collision between the uprush and the wave

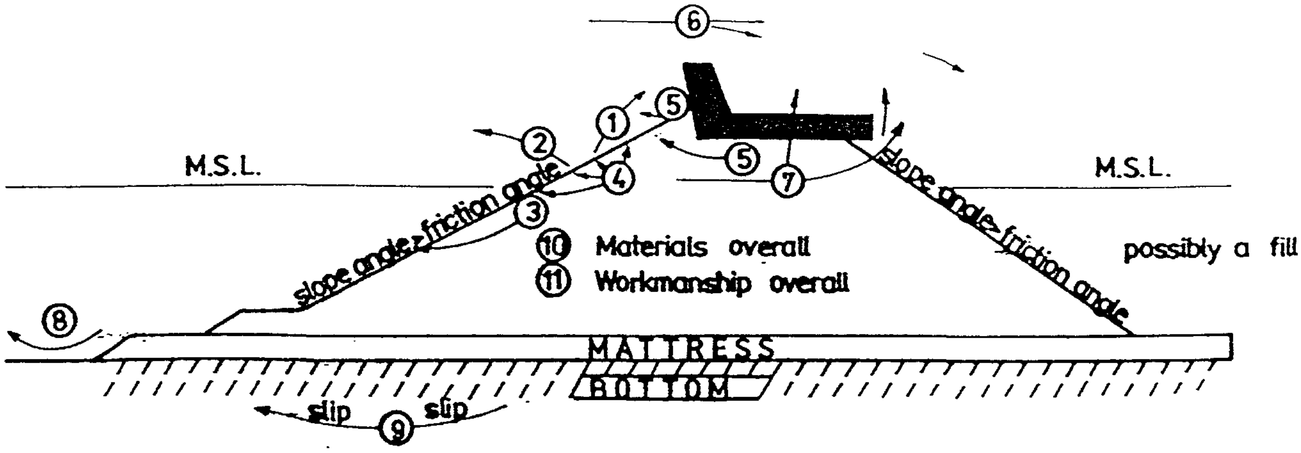


Figure 3.29. Failure Modes of Rubble Breakwaters, Bruun & Kjelstrup (1983).

screen causing massive outwashes of blocks in front of the wall. The experience clearly defeated preocuppation with any concept of 'Design Wave' as related to a spectral wave of a certain height and period. The main damage was clearly caused by a sollitary type, which apparently first lifted the armour blocks by buoyancy and momentum, next washed them down by downrush combined with a large outflow of water from mound. It is in this respect of no importance whether the mound is built of rock or of concrete blocks.

This most severe storm was peculiar in the way that the wave action was composed of 2 different kinds of wave action: a swell of about 2 m/10 sec and a wind wave of about 3m/6 sec.

Heavy damages took place to the outer of the breakwater. The fill was carried down on the outer slope levelling the crown 6 m down to about M.S.L.

The reason for the very extensive damage may be sought in the above mentioned combination of wave actions. The swell caused high uprushes and deep downrushes which is often detrimental to stability. The shorter wind waves probably contributed to the damage by "pounding effects" shaking blocks loose of bonds with other blocks and weakening the structure. Combinations of the two generated "mammoth" peaks as well as troughs, equally dengerous to stability. It is possible that a weak toe structure contributed to the rapid downfall.
The breakwater has been rebuilt with 1 : 2 slope, heavier rock and a more substantial toe structure, Kjelstrup and Bruun (1983).

The variety of failure modes of rubble-mound breakwaters is depicted in Fig.3.29.

The statistics of breakwater damage has been outlined in Fig.3.30. One of the failure modes, *settlement*, is depicted in Fig.3.31. The damage of rubble, with reference to both breakwaters and the forthcoming revetments, is illustrated in Fig.3.32.

Revetments

As described elsewhere in this study report, a block revetment consists of three major layers: top layer, filter layer, and base. Den Boer et al (1983) analysed the wave-induced pressures on a slope and distinguished eight types of resultant loads, during one wave cycle, which can lead to **failure of the top layer**. Later investigations by Delft Hydraulics (Burger 1983, 1984) have clearly shown that the **most critical** failure mode of the top layer is the uplifting and subsequent ejection of individual blocks as a result of combined high water pressures in the filter layer (internal load) and low water pressures outside slope (external load). This failure mechanism has been shown to occur only if the relative percentage of openings between the blocks is smaller than approximately 10 % of the top surface of the block. The design method proposed by Burger et al.(1988) is therefore confined to revetments with relatively closed top layers.

Bakker and Meijrs (1988) identify the following three major modes of failure mechanisms which must be considered in design of flexible revetments, viz:

i) – uplifting of blocks
ii) – sliding of revetment
iii) – instability of sublayers.

They focus attention on sliding of revetment, in particular over a filter layer.

Failure mechanisms in flexible revetments are paid a lot of attention in the design guidelines being prepared within WG 21 under PIANC auspices. The extensive experience of Dutch investigators, reflected in these guidelines, are highlithed in this document.

In general, the following three principal failure modes can be distinguished for (placed block) slope revetments:

1. Unacceptable motion or displacement of an individual element from the toplayer of a revetment. This can be either an artificial concrete block or a natural rock. It will be clear that this failure mechanism will lead to instant damage of the revetment.

2. Unacceptable internal transport of grains in a slope revetment's sublayer, from one fine sublayer to a coarser sublayer, or from a sublayer through the toplayer. Through undermining this leads indirectly to failure of the toplayer.

3. Unacceptacble sliding or liquefaction of one of the sublayers, leading to unacceptable deformation of the toplayer.

If the load on the toplayer exceeds a certain critical value this will lead to the following possible failure modes for the toplayer:

- Motion of an individual loose block.

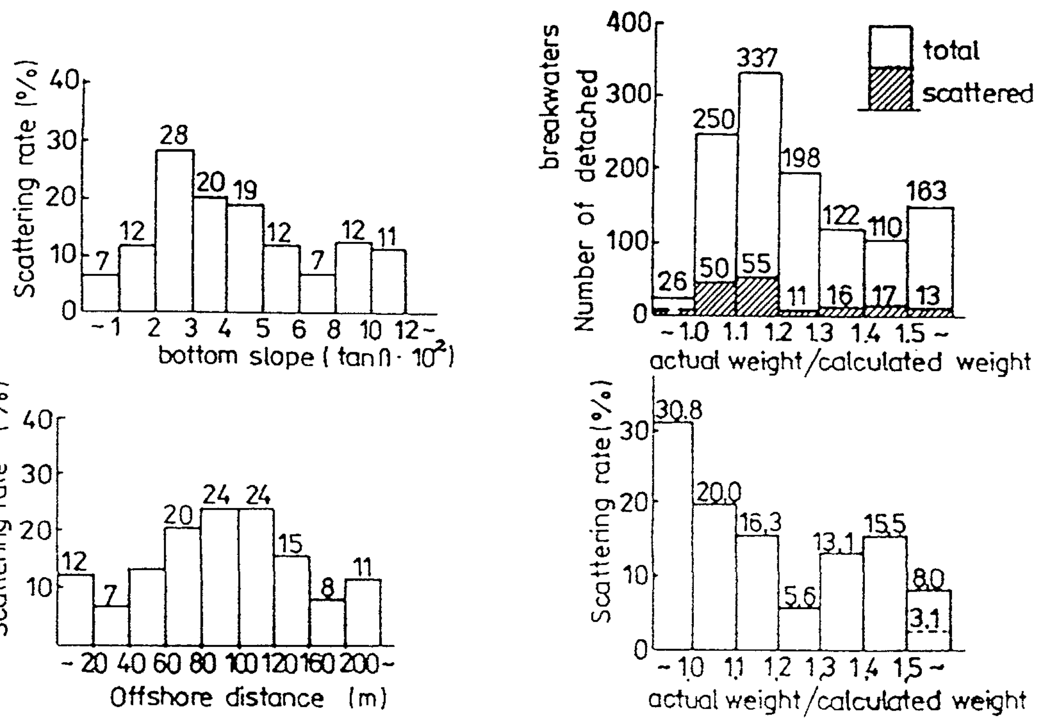


Figure 3.30. Damage Ratio of Breakwaters, for Different Mechanisms, Seiji et al.(1987).

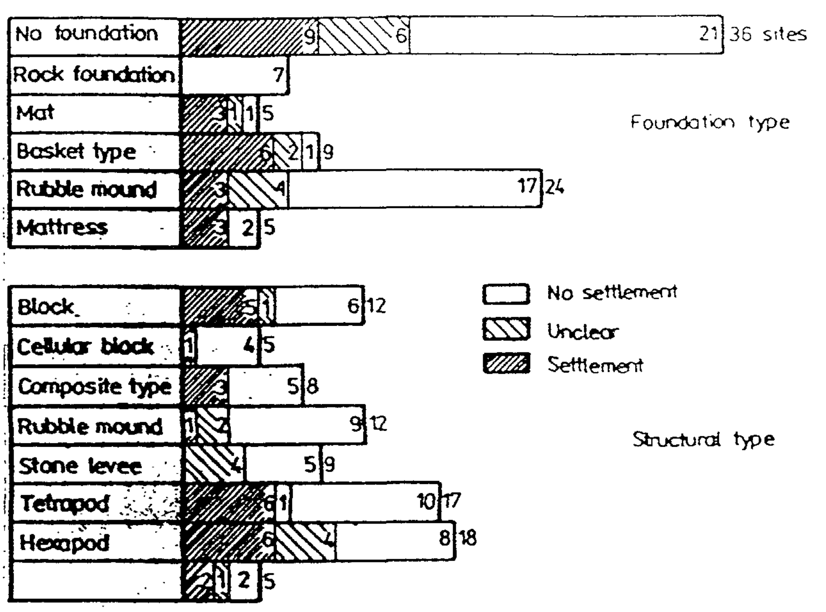
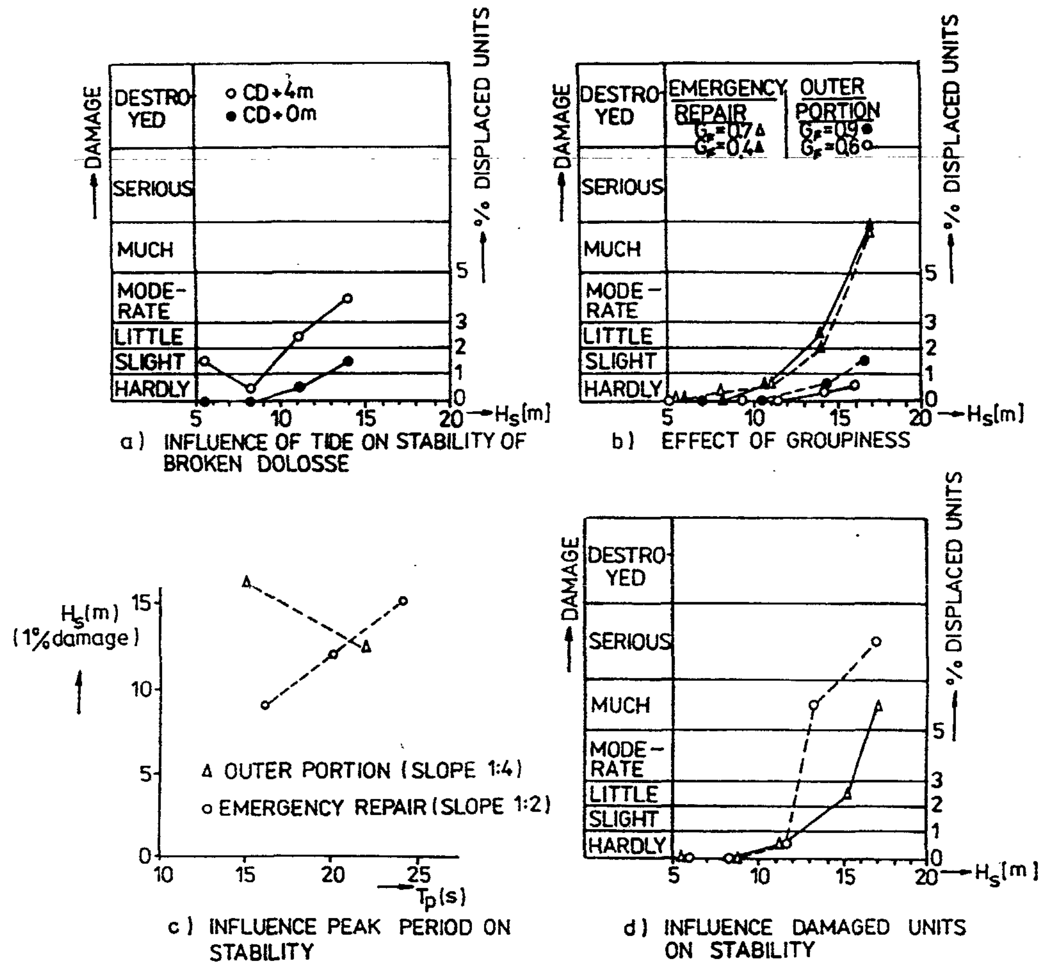


Figure 3.31. Settlement of Detached Breakwaters, Watanabe & Horikawa (1983).



The damage was classified as follows

hardly	: 0-1% of the units removed : no gaps
slight	: 1-2% of the units removed no gaps
little	: 2-3% of the units removed gaps of 0-5 units
moderate	: 3-5% of the units removed gaps of 5-10 units
much	: large gaps in top layer of primary armour
serious	: secondary armour visible trough small gaps in the primary armour
destroyed	: secondary armour affected core visible

Figure 3.32. Rubble Stability, Mol et al.(1983).

- Uplifting of an individual lose block.

- Sliding of the entire toplayer (or at least a significant part of it) from the system of sublayers.

- "Breathing" of the toplayer (the periodical upward and downward motion of part of the toplayer) in case of a more or less coherent toplayer, where the movement of an individual block is limited by the neighboring blocks.

The former three failure modes are possible for placed block revetments with loose blocks, the latter two can occur for revetments with some kind of interlocking between the blocks.

It shows that the resultant loads on the toplayer, together with the composition of the toplayer can lead to the following two modes of transport of granular material that may lead to damage of the revetment:

- Erosion from granular filter material through the joints between the blocks or holes in the blocks from the toplayer.

- Erosion of the fill material from the joints between the blocks or holes in the blocks from the toplayer.

In case the holes in the toplayer are larger than the grains in the underlying filter layer, this sublayer may be washed out through these holes. Two different mechanisms can be distinguished:

1. Erosion by outflowing water through the holes by the flow perpendicular to the toplayer
2. Erosion by the external water flow up and down the slope by wave impact, runup and rundown.

The first mechanism will take place during the moment of maximum rundown; the pressure gradient over the slope induces an outgoing water flow that may be able to transport the individual grains of the filter layer.

The second mechanism can occur during or just after the moment of wave impact or during the running down of water on the slope. This downrush generates eddies in the holes in the toplayer, by which the individual grains are eroded from the sublayer. Of course, a combination of both mechanisms is also possible.

For larger holes the velocity of the outflowing water decreases as a result of the large cross-section and the reduction of the gradient over the toplayer. In this case mechanism 1° is unlikely to occur, while mechanism 2°, as a result of the larger hole diameter is more probable.

Together with the composition of the sublayer under consideration one can witness the three different transport phenomena of granular material:

- Piping or hydraulic transport downward along the slope of granular material through initially small holes or channels in the granular sublayer at the interface with the toplayer or a geotextile

- Instability of interfaces between different sublayers; which in practice means that the finer (deeper) sublayer is washed out by the hydraulic flow through the voids in the coarser (higher) sublayer

- Internal instability of one of the sublayers, because of discontinuities in the grain size distribution of a wide graded granular material.

Internal instability of sublayers is another point of importance. A granular filter is called internally stable when the fine fraction of the filter cannot be washed out between the larger grains. The mechanism of internal instability is often referred to as suffosion or internal erosion.

Internal instability can only occur within materials with a very wide gradation (gentle sieve curve). This type of material is used more and more in slope protections, where industrial waste materials (slags, silex, minestone) become a cheap alternative for natural granular materials.

Internal instability is a dangerous failure mode of slope revetments. A gradual increase of the permeability of the filter layer leads to:

- higher internal pressures in the filter layer, endangering the stability of the toplayer
- Instability of the interface between filter layer and base material, which can lead to undermining of the entire structure.

Therefore internal instability should be avoided or the consequences should be taken into account in the design.

The internal hydraulic loads together with the composition of the sublayers can lead to the following soil mechanical failure modes:

- Flow sliding of insufficiently packed sand layers.

- Sliding along slip circles for slopes that are too steep for soil mechanical stability.

- Temporary periodical fludization directly underneath a "breathing" block from the toplayer.

The soil mechanical failure modes mentioned above only refer to direct failure of the revetment under excessive hydraulic loads, but not to general soil mechanical processes that involve the entire flexible protection structure and that not directly related to hydraulic loads, for example consolidation.

For each of the failure modes adequate methods of calculation must be given. It should also be realized that there is always an interaction between different failure modes, which makes the entire design process considerably more difficult.

Fulton-Bennet and Griggs (1988) provide the following summary as to the performance of revetments.a Although rip rap is undoubtably the most common form of coastal protection in the region studied, many of its alleged benefits may not hold during severe storm conditions:

(1) Rip rap revetments do not always exhibit the coherent "flexibility" portrayed in some engineering publications. Instead of settling as a cohesive unit, individual stones tend to separate as they rotate and/or settle, often moving seaward in the process. This causes the upper part of the wall to become less stable.

(2) Rip rap walls may fail quite rapidly, often leaving behind gaps of arcuate landslide-like scarps of oversteepened rip rap or exposed fill. Because many walls are designed as low as possible to minimize costs, even minor settling can allow significant overtopping, erosion, and damage behind the wall.

(3) Rip rap revetments built over steep, loosely consolidated materials require carefully planned drainage systems to avoid erosion of material behind the rock. Numerous rip rap walls were outflanked or partially failed because of erosion of material behind the rock. Some partially failed because of erosion from uncontrolled street or building runoff flowing behind or around them. Filter cloth is not always a practical solution to this problem, especially on slopes steeper than 2:1. Through-the-wall drainage pipes or conduits are often damaged as large rocks shift or settle.

(4) Although, at most times, placing new rocks on top of old, settled ones is relatively simple, repairing a rip rap wall while it is being overwashed by storm waves is extremely difficult, and at many locations, beach access is impossible under such conditions.

(5) Rip rap walls certainly reflect a smaller percent of wave energy than do vertical wood or concrete walls. However, during high tides and under certain wave conditions, reflected waves from rip rap walls have been observed to combine with incident waves, causing erosion and damage in adjacent areas, especially in small embayments.

Dykes

For the dyke as a whole, instability may occur due to failure of subsoil, front or rear slope. Each of these failure modes may be induced by geotechnical or hydrodynamical phenomena.
A brief overview of the failure mechanisms of dykes, dams or banks is given below.

Micro-instability of the soil material at the inner slope may be caused by seepage and a high phreatic plane.

A slip circle at inner slope can be due, *inter alia* by a high phreatic plane in a dyke. This will be the case when the duration of the high water level is long or permanent.

A slip circle in the outer slope may occur when a low water follows an extreme high water (or sudden draw-down). The body of the dyke is heavy with water and slides down.

A slip circle in the waterway bank may obstruct the fairway. This instability can be caused by a rapid draw-down of the water table in the waterway or the presence of weaker or impermeable layers in the subsoil.

A local shear failure (sliding of a revetment) parallel to the slope may also be the consequence of a rapid draw-down or hydraulic gradients perpendicular to the slope.

Erosion (removal of particles) of the dyke/bank protection or the bed may be caused by wave or current induced shear forces sometimes assisted by hydraulic gradient forces.

Piping (internal erosion) may occur i.e. the gradual formation of a material entraining flow under an impermeable revetment or through a local concentration of permeable material in the dyke body/foundation. When the "pipe" enventually reaches the high waterside the process of internal erosion will accelerate.

Migration indicates the transport of material behind the revetment. The transport may be parallel to the bank causing local slumping of the revetment or vertical resulting in the aforementioned S-shaped profile. Material may also be lost through the revetment when filter requirements are not met.

A liquefaction may occur in loosely packed sands under influence of a shock or a sudden draw-down. In this case the sudden increase or pore pressure reduces the shear strength practically to zero and the soil behaves as a liquid.

Pumping is seen when the revetment bends under external pressure and thus generates a flow of water underneath. The flow entrains particles of the soil.

Settlement is due to consolidation, compression, migration, oxidation of organic material (i.e. peat layers).

Horizontal sliding or tilting is mostly unlikely for a dyke or an earth dam, however, for rigid structures it is of paramount importance.

Ice may severely attack the revetment during winter time.

Heave of the soil may be caused by the formation of ice crystals within the grain skeleton of the soil during the winter.

Ship collision against the dyke/bank may cause considerable damage.

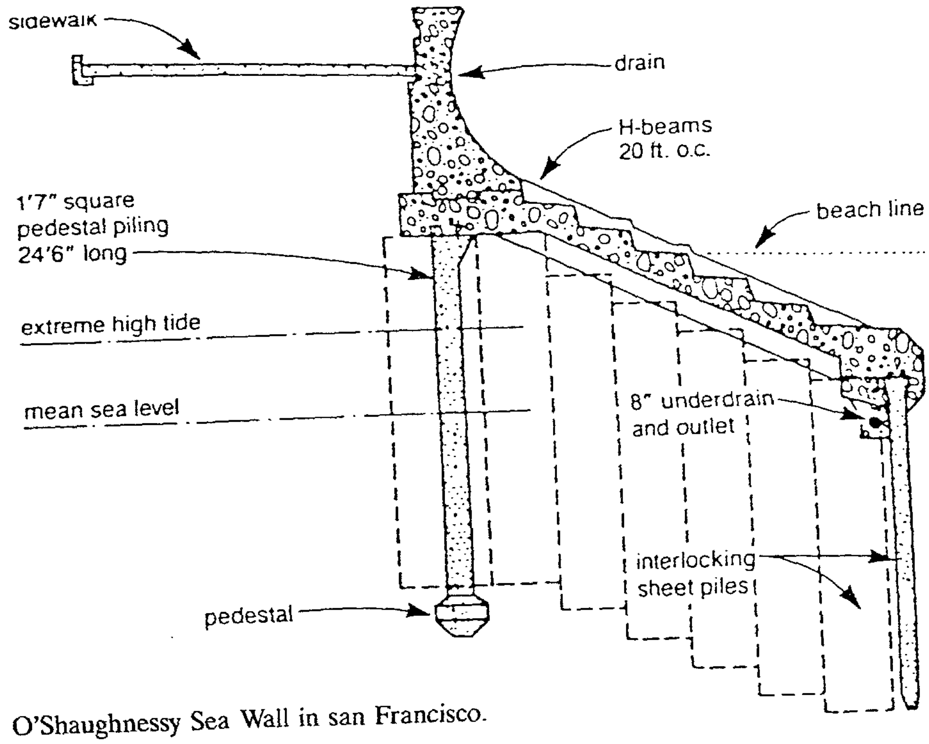


Figure 3.33. O'Shaughnessy Sea Wall in san Francisco.

Sea Walls

U.S. Army Corps of Engineers (1975) lists the following as typical causes of failure for concrete sea walls fronting the Great Lakes:

(a) loss of foundation support
(b) inadequate penetration
(c) scour at toe
(d) outflanking
(e) inadequate height.

Observations show that these causes of failure are also typical for the U.S. West Coast walls (Fullton-Bennett & Griggs, 198.). Loss of fill behind walls due to piping (the subsurface removal of loose sediment, soil, or fill, due to water flowing through voids or holes), gullying, and/or undermining are also prevalent. Although the Shore Protection Manual (US Army Corps of Engineers, 1977) states that failure of rigid sea walls is most likely to be catastrophic, many walls in the study region were endangered by a gradual removal of fill over several days or storm periods. Several concrete walls survived undermining or loss of fill because property owners had the time and money to add new fill and toe protection, preventing structural failure.

The O'Shaughnessy Seawall in San Francisco (Fig.3.33), completed in 1929 at a cost of \$ 575.000, is the single most successful protective structure within the study region. This massive concrete wall has survived the test of time because it incorporates design and construction elements that prevent each of the typical causes of failure.

Erosion of materials above and behind sea walls is one of the most widespread problems observed, perhaps because it can be caused by greenwater, wave splash, and even spray. The effect of very large volumes of runoff water on coastal structures has often been underestimated and should always be considered in designing those structures.

Erosion behind sea walls is a complex process, combining direct wave action, falling splash and spray, subsurface piping, and gullying by surface runoff. Saturation of soils may play a major part in the last two processes.

Frequent inundation of soils and sand by wave splash and spray (and often rain) can create temporarily saturated conditions within the upper layers of soil, resulting in ponding behind sea walls. This occurred behind virtually every vertical sea wall observed during the storms of 1983, even those with filter cloth and other "through the wall" drainage systems. If ponding is severe, water will begin spilling over the side or front of a sea wall, in some cases causing loss of fill or outflanking. Once saturated, soil becomes increasingly susceptible to gullying and piping. Both these processes tend to follow path of least resistance, gullying being most likely around the flanks and low portions of sea walls, and piping occuring at minute cracks, joints, tears in filter cloth, or other regions of concentrated flow.

Where water cannot exit directly, it may flow behind a sea wall, parallel to shore, for hundreds of feet, before finding a weakness or gap in the barrier (Site No. 25, 1983). To counter this problem, the O'Shaughnessy sea wall incorporates a series of deep cutoff walls at right angles to the wall, at one hundred-foot intervals, so that if the section of the wall failed or was undermined, it would be less dlikely to affect adjacent sections.

Undermining of sea walls occurs when foundation material (usually sand, fill, or rock) is removed by waves. This may take place not only when beach sand is scoured or fluidized, but also where bedrock erodes rapidly during storms. In either case, the result of undermining is often rapid loss of fill from behind a wall, and in some cases, structural failure. Undermining of rigid walls can be difficult to recognize, since it may remove subsurface material, while leaving visible portions of a wall and paved surfaces behind the wall intact. Undermining of rip rap walls is more obvious, since these "flexible" structures will settle into the undermined area.

3.2.5 *Some Other Failure Mechanisms*

Dykes and similar defense structure can consist of **cohesive soil.** This is the case studied at IBW PAN (Polish Academy of Sciences; Stepkowska, 1987), in which soil of the dykes was generally weak (mud, sometimes peat and organic soil) or medium cohesive (homogeneous mud, some 10... 15% of organic soil), with pronounced properties of swelling and shrinking (the strength of unconsolidated dyke being 7... 15 kPa, versus the consolidated one of 30 kPa, for the 500-kPa modulus of compressibility at σ = 10... 50 kPa.

The following failure modes have been observed:

a. excessive permeability, accompanied by downstream piping

b. considerable and diversified settlement of the dyke crest (up to 50 cm)

c. loss of stability in case of overfill, accompanied by seepage outflows on the downstream side

d. mechanical erosion by waves.

The following strata of the dykes can be identified:

I. Upper stratum of granular soil, with large aggregates and coarse gravel. Cohesion is minor, permeability k exceeds 0.01 cm/s.

II. Central stratum of mud, with open cracks and considerable permeability (close to 0.01 cm/s). The thickness of this stratum is 2 m (below dyke crest, or 1.5...1.6 m at the minimum).

III. Lower stratum of primary consolidated mud (10^{-8} cm/s).

The **failure mechanisms** or **arch-factors** include primarily alternate **drying** and **freezing** (mostly for stratum I), **ageing** of soil and gradual **aggregation** (mostly for stratum II) and **swelling**.

Drying and **freezing** enhance the generation, existence and intensification of internal bonds between soil particles in a primary aggregate (referred to as 'domain'), between domains in the aggregate of aggregates (denoted by 'cluster') and, generally, between lower-order aggregates in higher-order aggregates of cohesive soil. IBW PAN tests (Stępkowska 1987) have shown that the size of aggregates grows during dry storage. Mud samples taken from the Northern Harbour of Gdańsk and stored for one year had aggregates by 20 to 40% larger than initially. Another Baltic mud doubled its aggregates upon tencycles of freezing and thawing. Hence, a soil having a moderate cohesion, subject to cyclic drying and freezing, can behave as a granular medium; this was indeed observed in the upper stratum of the dykes studied.

Ageing of the system water-clay is most probably respondible for cracking in stratum II. This is linked to gradual aggregation of clayey minerals, i.e. transition from microaggregates to higher and higher-order aggregates. The older the soil the larger the size of the high-order aggregates, and the more vulnerable the soil to cracking and enhanced seepage. It is interesting to note that the process of aggregation can be impeded by controlled use of 'impurities', such as coarser grains, which would prevent smooth aggregation, much as admixtures impede the analogous process of crystallization. - Another method of preventing aggregation consists in homogenization of the aggregate structures.

Swelling of clay is caused by exchangeable cations, which in all colloids constitute a diffusion layer about particles. The thickness of this layer depends on many factors, first of all water content. If the latter is much lower than the liquid limit, the adjacent diffusion layers push out, by analogy to osmotic pressure. The repulsion is in balance with the negative pore pressure.

If there is free inflow of water, the soil system swells by increasing its wave content. If swelling can continue over a longer time span, the average size of particles can decrease, the so-called delamination occurs, accompanied by the growth of the water content and system volume. Delamination has been observed in bentonite, with reduction of particle size to one-half of the initial value. Similar behaviour, with the reduction to one-third, was recorded for Grimmen clay. Upon consolidation and shearing the mean size of the clay particle returned to its original magnitude (Stępkowska, 1987).

Failure-prevention methods for **cohesive soil in dykes** may be based on the tests reported by Stępkowska (1987). The measures that can be used are described briefly in Chapter 5.

Adel, Bakker and Breteler (1988) discuss **internal stability** of filter layers used in protection slopes along banks and dykes employing *waste materials*. In some materials, such as minestone wastes, their wide grain size distribution favours **washing out of fine grains** through the **skeleton of coarse grains**, which is one of the **failure mode** discussed, this time enhanced for particularly fine particles. Two effects prevent direct application of the concept of boundary instability to the problem of internal instability. Firstly, transport of base material into a filter layer occurs only at the boundary layer between base and filter. In contrast, in the case of internal stability, fine grains in the filter itself can be removed everywhere, thus increasing the mass of the transported material. Secondly, the capacity of water to contain material is limited. When water is saturated, no further erosion can occur downstream. Since the water accumulates the material collected, less and less material can be removed downstream, due to the saturation of water. This feature leads to a characteristic erosion length.

Mechanics of piping in dykes is analysed extensively by Sellmeijer (1988).

The design measures aimed at preventing the failure modes presented in this Chapter are discussed in Section 4.2 and are illustrated in Chapter Five.

CHAPTER 4

Geometrical and structural design

4.1 GEOMETRICAL DESIGN (Morphological Dimensioning) of COASTAL STRUCTURES

4.1.1 *General*

As already mentioned in the foregoing sections, the design procedures for coastal structures should include **geometrical design** and **structural design** reflecting respectively the far-field and near-field requirements imposed on structures. The description of the far-field effects is contained in Chapter 2 while the near-field mechanisms are dealt with in Chapter 3. This corresponds to our 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.

Accordingly, this Section 4.1 is devoted to the **geometrical design** while Section 4.2 provides quidelines on **structural design** of structures and their constituents. Both are supplemented by Section 4.3. placing emphasis on **forces and stability,** primarily with regard to structural constituents.

4.1.2 *Cross-shore structures (groynes)*

Types of Groynes and Their Paremeters

The groynes encountered most frequently have vertical walls and are rectilinear, normal to the shoreline. Their standard shape is often supplemented by additional elements which enforce wave diffraction and thereby increase accretion or diminish erosion around the structure. In plan view groynes have the shape of the letters L, Z or T.

Groynes are made from different materials. Timber, steel, concrete, natural rubblemound, precast elements or even sandy bituminous revetments are used in the USA (SPM 1977) on an experimental basis. In Great Britain groynes used traditionally are vertical-wall, rectilinear structures made of concrete, steel or timber (Brampton and Motyka 1983). Some other volutions are sometimes seen in the form of the letter T or Γ, which is implemented through adding structural elements situated parallel to shoreline. The rubble-mound groynes are constructed from stone or concrete elements, but due to their higher cost they are more seldom. Permeable groynes are also encountered, e.g. two timber groynes in Bournmounth or tubular steel groynes on the East Anglian coast having 40% clearance and being zig-zag in plan view, Tomlison (1980). Groyne lengths vary from 30 m on gravel beaches to 300 m along sandy beaches. The groyne spacings are different and vary from 1 : 1 to 1 : 4, after Tomlinson (1980).

In the Netherlands one often uses rubble; groynes are low and streamlined, while their seaward tips are sometimes strengthened with bituminous materials. In Germany the most frequent groynes are rectilinear and normal to shoreline. Sometimes rubble groynes are made of natural stone. The joints are filled with bituminous materials and the surfaces are made smooth. The crowns of the groynes on the land side are slightly higher than those on beach. Slightly inclined groynes project ca 0.75 m above the beach datum while the strongly inclined ones emerge ca 0.5 m. The ordinates of groyne heads are about 0.50 m above mean water level, and the slope of the groynes corresponds to the mean slope of the beach profile.

The groynes constructed in Poland are timber or reinforced-concrete palisade structures, along with few groynes made of steel sheet (cf. Table 14 in Appendix 2-1, Basinski 1963). These structures are basically impervious but sometimes permeable configurations are used. The length of Polish groynes is 90 m on the average, their spacing being about 60 m (cf. Table 15 in Appendix 2-1).

Groynes are one of the most controversial coastal defence measures. In many instances they are successful but equally well they might be situations with detrimental environmental effects. These effects are due to local conditions of the coastal environment, design characteristics of sediment transport, and general variability of many factors. Groynes are usually applied if the **longshore sediment transport**

predominates. One must be aware of the erosion on the down-drift side of a group of groynes. Such erosion may require additional artificial beach nourishment at places particularly vulnerable to damage. Accretion can be encountered on the up-drift side of a groyne system. The temporal changes include variation of geometric characteristics of groynes, their relative length in the surf zone etc.

Basic dimensions of each groyne are its length, height and width. The length and height of groyne depend on coast morphology and long-shore sediment transport rate. The width of groyne also depends on the wave forces, so that a groyne is usually designed as narrow as possible.

Further quantities include permeability of groyne and groyne orientation with regard to shoreline. As groyne systems prevail in practice, and not single groynes, one must also specify parameters of the system configuration, primarily spacing of groynes.

The diversification of recommendations on different groyne parameters, in various parts of the world, is well exemplified in Appendix 2-1, from which the ranges of design groyne parameters can be consulted whereas given below are some most commonly advisable quantities.

Dimensioning of Groynes

Our analysis of characteristic parameters for groynes permits the following conclusions.

Groyne Length

It is not defined in a unique way. More precisely, one should consider the following basic composition of a groyne:

a horizontal shore section

b intermediate sloped section

c outer section

This will be done in the following subsubsection where data on groyne length are combined with groyne height.

The total groyne length i.e. the distance measured from the groyne head to the point of the maximum runup, should be basically the **width of the active longshore transport zone.** Different estimates are given in literature for this quantity, and they are determined in various ways depending on the choice of the level of reference (on the sea side this level being above or below the low water level while on the land side it is counted from the berm or from the crown of the protecting structure, e.g. a seawall located behind the zone of beach transformation).

The protrusion of groynes in the seaward direction should depend on the quantity

Table 4.1. Recommended length of subaqueous segment of groyne, in metres.

Type of sea bed and sediment Grain diameter	slope $< 1:25$	sea medium	bed $> 1:25$
fine	30 m	24 m	18 m
medium	26 m	20 m	16 m
coarse	22 m	16 m	12 m

of sediment to be entrapped.

In tideless seas most of the longshore transport takes place in the surf zone, shoreward of the breaker line. The most common practice for tidal seas is that the boundary for the maximum sediment transport across the beach profile is determined by tidal oscillations. The studies of the Wallingford Laboratory for regular waves have shown that up to 90% of sediment transport occurs in the zone landward of the ebb-line.

The following recommendations are additionally contained in the Polish guidelines (Engineering Tables, Szopowski 1956).
If renovation and reconstruction of already constructed groynes are intended the above recommendations must not be observed.

The tests conducted in hydraulic laboratories by Nagai (1956) and Nagai and Kubo (1958) have resulted in the conclusion that the optimum length of the groynes that entrap the longshore moving sediments is 40% of the width of the surf zone limited on the seaward side by the plunging breakers. This recommendation does not entirely agree with other guidelines.

Groyne Height

The elevation of groynes controls the quantity of sediments entrapped from its longshore portion. It is generally recognized that the higher the groyne the more sediments are entrapped by the groyne system. However in practice it appears that wave reflection also increases with increasing groyne height. More flow perturbations enhance local scour and thus bring about lower bed levels in the vicinity of groynes. For sandy beaches groynes are constructed as relatively low features. The recommended heights are 0.5 to 1.0 m above the bed level. For shingle beaches the opinions are divided since some investigations recommend the usage of structures similar to those which are intended for sandy beaches, but on the other hand some others recommend high groynes because wave reflection is likely to be correlated with lower erosion.

Some structural solutions make possible fitting groyne heights to the changes in bed profiles during the periods of storms and relatively calm weather. One also diversifies the groyne height in different sections.

Horizontal shore section. This portion of the groyne extends landward from the desired location of the beach berm. It serves to anchor the groyne and to prevent outflanking. The minimum, and most economical, height for this part of the groyne will be the height of the desired berm plus the height of normal wave uprush. When constructing stone groynes, an additional foot is sometimes added to decrease movement of sand between large capstones. The length of this section will vary but it must extend landward of the desired berm crest to the point of wave uprush, and prefarably beyond if economically feasible. This section may be horizontal or it can slope slightly seaward to parallel either the existing beach profile or the desired profile if a wider beach is desired.

Intermediate slope section. This portion of the groyne should parallel the slope of the foreshore the groyne is expected to maintain. Its length will depend on the amount of material the groyne must retain to create the desired beach. The height of this section is similarly determined.

Outer section. This part of the groyne extends seaward from the intermediate section. For reasons of economy and safety, height of the outer section should be minimized. The proper length of this section is controversial. Some recommend extension to the outer edge of the zone defined by the breaking point of normal waves. Others suggest shorter extensions. The best choice will depend on the volume of fill required and on the wave charcteristics. The outer section of groynes may be modified at their ends into curves, hooks, angles, T- or L-letter, in the hope that downdrift recession will be reduced. Such modifications are ill advised since they only shift the zone of maximum recession and invite excessive scour.

Groyne Width

Groynes are generally narrow. Their width is nearly always dependent on the wave forces they must sustain and the strength of materials used. The narrowest groynes are of steel or timber and the widest of rock rubble, synthetic bags of sand or rock-filled gabions.

Additional important elements of groyne design are permeability and orientation with regard to shoreline.

Permeability of Groynes

It is a measure of the extent to which the structure will pass littoral drift material. A permeable structure may alleviate the downdrift recession associated with impermeable groynes. Balsillie and Berg (1972) find that the relationships between littoral forces, permeability and resulting shore behaviour have not yet been quantified adequately. However, it is generally supposed that permeable groynes should be used only in systems and that permeable groynes need a large amount of littoral drifft to be effective.

Permeable groynes can be constructed in a wide variety of materials. Any form of construction that will allow sand to pass through the structure is suitable, providing structural integrity is secured. Rock and precast concrete units can be used to construct groynes with varying, and usually unpredictable, degrees of permeability. Permeability of groynes constructed from these materials depends on core design, size and packing of the components, and width of the structure. Structures with little or no core of smaller stone are more permeable, since sand can pass easily through the interstices of the larger components. Likewise, large rocks and shapes, loose packing and narrow width result in greater permeability. Permeability can only be approximated and will decrease with time due to fouling organisms and debris.

Permeable groynes may also be constructed from wood, steel and aluminium sheet-pile. These structures appear as bulkheads constructed perpendicular to the shore, possessing 'windows' or openings at regular intervals through which sand passes. In this way permeability can be controlled by varying the size and location of windows. Designing permeability into structures is less than an exact science and is best accomplished by on-site experimentation or past experience at sites with similar conditions.

Permeability of groynes in a system considerably reduces cost of coastal defence. Compared with stone filled impermeable groynes, the cost of permeable groynes with timber piles is up to 10 to 25% lower, Bakker et al. (1984).

Permeable groynes are also recommended if one intends to reduce a tendency towards recurrence, possible scouring at groyne head, and negative effects on the down-drift sides of a groyne system.

Local waves and currents, sediment transport rate, and other coastal factors determine the spacing between individual piles in a groyne.

Groyne Spacing

It is generally given in terms of the ratio of the distance between two neighbouring groynes and their average length. The experimental studies on the relationship between the groyne length, spacing and location with respect to the shoreline, including the angle of wave incidence, have been carried out in a wide range of parameters since the fifties. The tests conducted in hydraulic laboratories by Nagai (1956) and Nagai and Kubo (1958) have suggested that the groyne spacing depends on the angle of wave incidence.

Groyne spacing for a particular site depends on the wave climate, the angle of wave attack, the steepness of the beach profile, the grain size of the beach material and the economic factors related to construction costs and land use. Proper spacing is necessary for an effective groyne system. Groynes spaced too widely produce excessive recession on the downdrift sides. Groins spaced too closely produce a net offshore transport of sand thus increasing erosion. Correct spacing accretes material on the updrift side of each groyne which extends to the base of the next updrift groyne. The selection of the correct spacing for a site depends on a study of the site. When an adequate study cannot be made before a system is built, it is recommended that the groynes be spaced too widely rather than too closely. It is easier to build intermediate groynes in a widely spaced system, should that prove necessary, than it is to remove groynes from a closely spaced system.

An overly high spacing brings about considerable energy fluxes towards the shore and thus causes high sediment losses along the protected coastal sections. On the other hand, if the spacing is too small it makes impossible for sediment to enter the bays between groynes and stimulates the transport of sediments along shore seaward of the groyne head-line.

Nagai and Kubo suggest that the location and spacing of groynes should be taken according to Table 12 in Appendix 2-1.

Basing on his studies in a wave flume with sediments having different densities (such as bakelite, pumice and natural sand) Barcelo (1968) gives the optimum spacings of groynes, depending on the angle of wave incidence summarised in Table 13, Appendix 2-1.

It should be pointed out that there is no unique definition of groyne length and this gives rise to different recommendations by various authors as to the relationships between the groyne length and spacing. Brampton and Motyka (1982), basing on their hydraulic tests and field investigations emphasize the diversification of conditions for which the conclusions were drawn. They indicate that the optimum groyne spacing should be linked directly to the angle of wave incidence. The more normal the wave ray the lower the effective groyne length and thus the lower the groyne capacity to entrap sediments. Therefore in order to reach the desired efficiency of sediment entrapment the groyne spacing should be the lower the greater the angle of wave incidence. Under natural conditions on beaches subjected to waves from different directions it is necessary to determine the shoreline configuration for each direction of wave incidence. This requires deep-water wave forecast basing on wind data and then determination of shallow - water wave parameters by inclusion of refraction and diffraction with various numerical methods (cf. Section 3.5).

The dimensionless groyne spacing being accepted practically hithero varies in wide intervals. Tomlinson (1980) idicates that this quantity can vary from 1 : 1 up to 1 : 10, cf. Tables 2 and 8 in Appendix 2-1. The recommended relative groyne spacing for sandy beaches varies from 1 : 1 to to 1 : 4, while those for shingle beaches are about 1 : 1.

Brampton and Motyka suggest that due to high costs groyne groups should be constructed with wider spacing. For example, on shingle beaches they recommend initial spacing of 1: 2, which can be changed to 1 : 1 by inclusion of additional shorter groynes in between the existing ones, if it turns out necessary. The initial

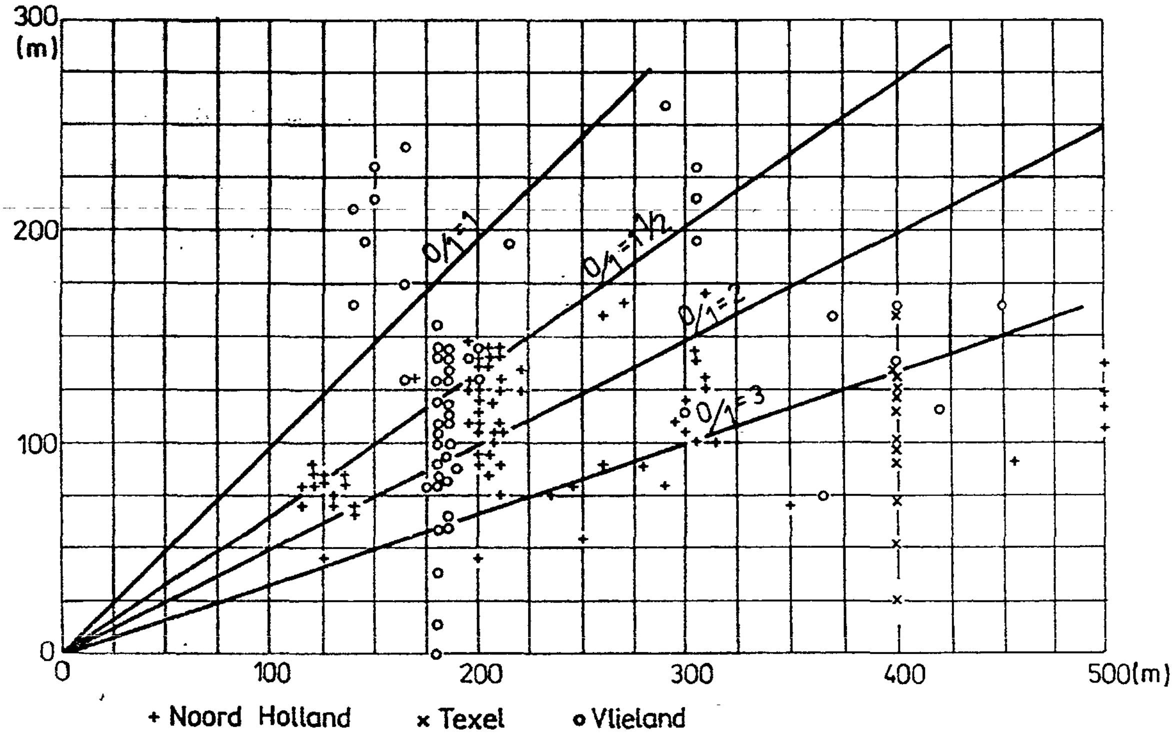


Figure 4.1. Groyne Spacing (x-axis) vs. Length (y-axis) in the Netherlands, Rakhorst (1984).

spacing for sandy beaches should be 1 : 4, which can be changed to 1 : 2 subsequently, with groynes of full length.

An overly high groyne spacing on shingle beaches can bring about zig-zag shoreline configuration and an asymmetric bed topography in the vicinity of groynes. This phenomenon is not clear-cut on sandy beaches.

Small spacing of groynes (eg. smaller than 1 : 1) can cause worse operation of a group of groynes (Tomlinson 1980). After having filled up the first bay between groynes, sediments move along shore either through overtopping the groynes or by passing the groyne head. Observations have shown that small spacing of groynes have little chance for the sediments to accumulate in subsequent fields due to the tendency to remove the sediments seaward of the groyne head line.

In most cases a single groyne cannot provide adequate protection and a system of groynes must be constructed to protect longer sections of shoreline. The basic processes of accretion and recession characteristic of single groynes also characterize groyne system. In addition to its ability to protect any length of shoreline, a groyne system derives an advantage from the interaction of the individual groynes. The downdrift side of each groyne benefits from accretion on the updrift side of the adjacent downdrift groyne. This often widens the beach.

A groyne system may be built in two ways. One may build the groynes and wait for the natural sand movement to bring the coastline to its stable configuration or,

Table 4.2.

Bed Material	Nearshore current	
Grain Size	Insignificant (up to 0.2m/s)	(Strong)
Fine	1.5	3/4
medium	2.0	1.0
coarse	3.0	1.5

once the groynes are in place, one can import sand to bring the beach to its final state at once. The second method increases the cost of the system but minimizes downdrift recession. To shorten the time between groyne construction and final fill, it is customary to construct all the groynes of the system simultaneously.

Most investigations recommend spacing ratios of 1:2 to 1:3.

Basing on many years observation of Dutch coastlines Rakhorst (1984) concludes that the optimum spacing of groynes, rated to their length, is a crucial issue in view of effectiveness of a groyne system.

Depending on local conditions this relative spacing may vary from 2 to 3, if the absolute spacing a is in the range from 120 to 500 m, Fig.4.1. Hence this agrees with the above guidelines contained in Technologies . . . (1982) and shore Protection Manual (1977). The relative spacing recommended in Poland is given below.

The spacing is suggested as a multiple of the subaqueous part of groyne, that is the central segment, measured from the shoreline.

Orientation of Groynes

Relative to the average shore line, it is usually normal (perpendicular). It has received fairly little investigation and is presently controversial. Generally, it is believed that the depositional characters of groynes is related to angle of wave attack on the structure, there being some optimum angle for sand retention. Evidence exists that groynes oriented slightly updrift from normal to the shore-line are most effective (as well as the least costly). However, more research is needed before reliable recommendations concerning orientation can be made because, on the other hand, physical tests have indicated positive effect of groynes if they are inclined in the downwave direction, although variability of wave under natural conditions can bring about some detrimental effects in many cases (local erosion).

Practical Remarks

Dutch groynes are usually constructed of timber, 0.25... 0.30 m in diameter, Bakker et al (1984). Groyne height depends on waves and tides and is defficult to define in a unique way. Overall length of piles must be designed with the account for natural sea bed changes due to morphological transformations. It is generally assumed that 60% of total pile length is driven into bed, Bakker et al. (1984).

Polish guidelines of 1956 (Engineering Tables, Vol.IV) stipulate that groynes are to be constructed if soil permits driving of piles. One row of groynes is applied, with piles being spaced at less than 5 cm. One usually employs timber piles, but steel or reinforced concrete is also acceptable. The diameter of timber piles should exceed 20 cm. Piles are free standing, without connecting units.

The guidelines recommend 6-m length of the subaeral segment of groyne. The downdrift side of a groyne system should be made gradually shorter so that a smooth transition to natural unprotected beach is attained. There should be at least two shortened groynes within this transition section. The accurate angle of groyne head line, with respect to natural beach should not exceed 2°.

If groynes prove successful with regard to morphological effects and entrapment of sediment, so that groynes themselves are buried in sand, it is recommended that they be extended.

Permanent morphological effects of groynes can be concluded from at least 25 months of operation, upon elimination of seasonal changes in beach transformation. Both subaeral and subaqueous areas should be considered in the analysis.

If a longer coastal segment is to protected with groynes one is advised to enlarge the spacing and fil it in afterwards with additional groynes. Thus probability of driving additional piles must be inherent in the original design.

Further remarks with regard to durability of groynes are provided in Section 4.2, in addition to those offered in Section 3.1.4.

4.1.3 *Offshore breakwaters*

The functioning of offshore breakwaters depends on their geometrical proportions and local environmental conditions.

Offshore breakwaters are often constructed as rubble-mound breakwaters of trapezoidal cross-section, having different slope angles on the seaward and shoreward sides.

Distinction is made between low and high, i.e. **submerged** and **emerging** breakwaters. It is assumed that the former have a height of less than 40% of the water depth and relative submersion $(d - h_B)/h_B > 0.5$

For details from which the following quidelines are derived, the reader is referred to Section 2.3. In particular, the following points of interest are suggested

- Select *transmission properties* of the breakwater to compromise on the functions of sheltering and sedimentation control
- Decide on tombolo or salient, depending on the quantity of sediment to be entrapped or passed by. Use can be made of *Figs 2.9, 2.11, 2.14, 2.15*;

- Decide on more intensive control of hydrodynamics and sedimentation (offshore breakwater in closer proximity of shore line) or allow for weaker interaction (the distance Y_B greater, but not beyond the surf zone, stretching to Y_b);

- For different configurations of breakwater(s) in plan view, try Spataru's design chart in Fig.2.16.

The following suggestions may prove helpful.

Emerging Breakwaters

Taking into account the stability espects and cost of structures Franco (1985) recommends the **seaward slopes of 1:1 to 1:3** for an offshore breakwater. The overall stability of the structure also depends on the landward slope, which should counteract the erosive effects of the overtopping water; the **landward slope 1:1** is recommended most frequently. Other investigators suggest rather flatter slopes.

Crest width and **elevation** above mean water level are also important parameters affecting dissipation of waves. Crest width is generally about 3 m, which can be increased to 5 m at exposed places, and even more if stronger energy dissipation is required. **Crest datum** is generally 0.5 m to 1.5 m above still water level. Breakwaters with higher crest dissipated more energy but must resist higher forces and bring about higher reflection. The design of structure often determines the elevation of crest, for instance in rubble-mounds field from front side it is necessary to maintain a certain datum above SWL. One should remember that the crest datum is generally subsiding during operation of a structure.

Length of offshore breakwater is generally 100... 200 m) which corresponds to 2...6 mean wave lengths). In the case of segmented breakwaves operating in a system of detached breakwaters one also assumes 3...10 mean wave lengths for a single detached breakwater.

Gap width between segments of detached breakwaters in a system is usually taken as one wave length, which becomes 25...30 m.

The distance from original shoreline is usually 20...100 m. An additional condition arises if one has to provide adequate diffraction upon angular incidence of waves, which occurs if the distance from the shoreline exceeds double width between breakwater (gap width), that is at least 60...70 m. Offshore breakwaters should be situated in parallel to the crest line of predominant waves, which most generally means shore parallel configuration.

Submerged Breakwaters

Laboratory tests and field investigations in a variety of conditions for a sandy coast of Japan and pebble coasts of the Caucasus have exposed the following features.

Depth of siting of submerged breakwaters is generally from 2 to 5 m, but an optimum depth must be determined eachtime for a given coastal segment.

The distance of submerged breakwater from shoreline depends critically on the depth of foundation and slope of sea bed. The most frequent solutions incorporate distances from several tens to a few hundreds meters. Submerged breakwaters located in the region of breakingwaves and in the surf zone cause higher scour if their height is higher. The submerged breakwaters situated on the seaward side of the breaking line cause higher erosion if their crest is lower.

Crest datum was generally 0.5 to1.0 m below SWL in the practise applied till 1963. Adequate altenvation of waves was belived to achieve if the elevation of submerged breakwater exeede 0.7 depth of water. Steep waves usually exposed lower effect of submerged breakwater.

Crest width is found in all investigations as a factor improving wave dissipation for increasing crest width. Economic assessment must determine selection of this parameter. The minimum crest width (perhaps made dimensionless with respect to wave height or wave length, or both) is an important factor of wave breaking, energy dissipation, and overall interaction mechanisms. The optimization should aim at minimum cost (volume) of breakwater and maximum possible dissipation. No clear-cut guidelines are known so far. Some hints can be found in PIANC WG21 (1992), where i.a. Fig.4.21 shows the distribution of energy in the vicinity of a reef breakwater.

Cross-section shape of submerged breakwaters may be selected in a variety of ways, but it is generally assumed that trapezoidal cross-section is better than the rectangular one. The best morphological effects, pronounced as acretion of sediment between the submerged breakwaters and shoreline, have been reached for the slope of the seaward wall ranging form **1:2 to 1:3**. Better dissipation of waves, lower reflection, and easier transport of sediment over the structure were observed for these slopes. Lower erosion at the foot of the structure was also noticeable.

The investigations carried out in Poland and the Soviet Union, in the fifties and sixties, identified the following favourable shapes of cross-section: trapezoidal (Fig.4.2.A; f...h), rectangular with a vertical landward wall (Fig.4.2.A; i,j,k), trapezoidal with a concave arch on the seaward wall (Fig.4.2.A; t) and hollow elliptic (Fig.4.2.B; f). Various design techniques to minimize downdrift effects as well as maximize the resulting shoreline protection are discussed by Fulford (1985), who claims that in most situations a combination of coastal defence measures is generally the best solution. The following quidelines are offered for offshore breakwaters.

- *Breakwater Length Vs Location With Respect To The Shoreline*

  Tombolo formation can usually be prevented if the structure length is less than the distance offshore, (SPM, 1984).

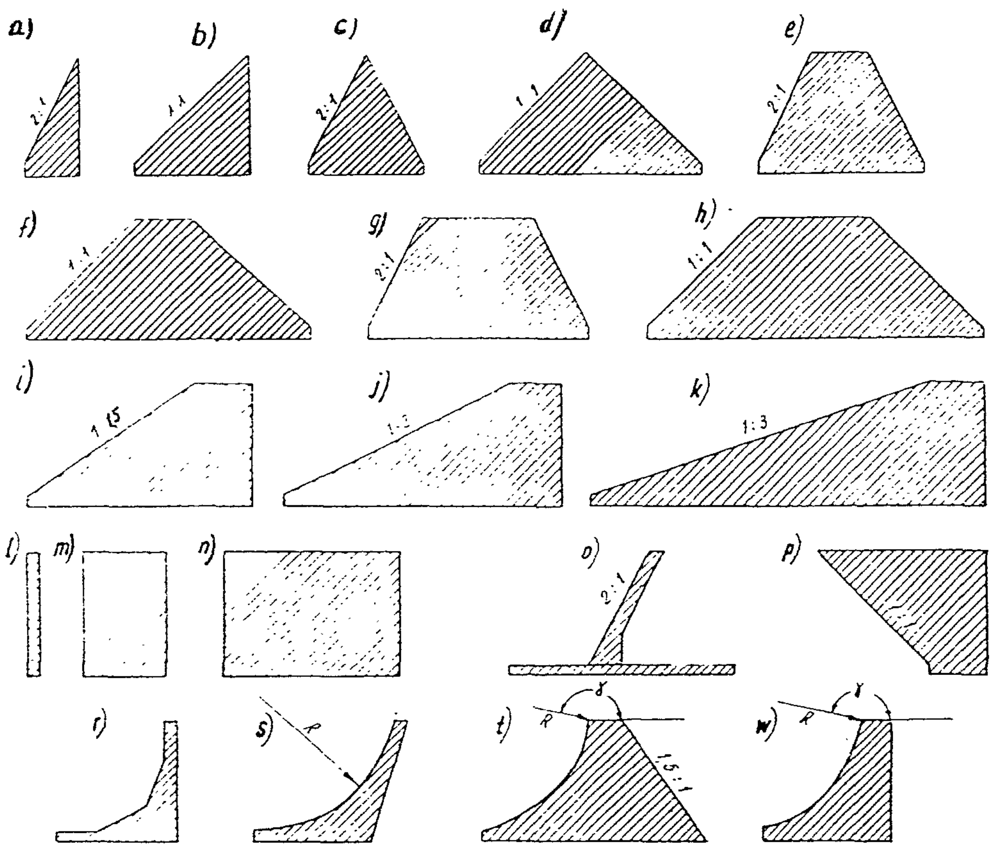


Figure 4.2.A. Cross-sections Tested in Odessa, Basiński (1963).

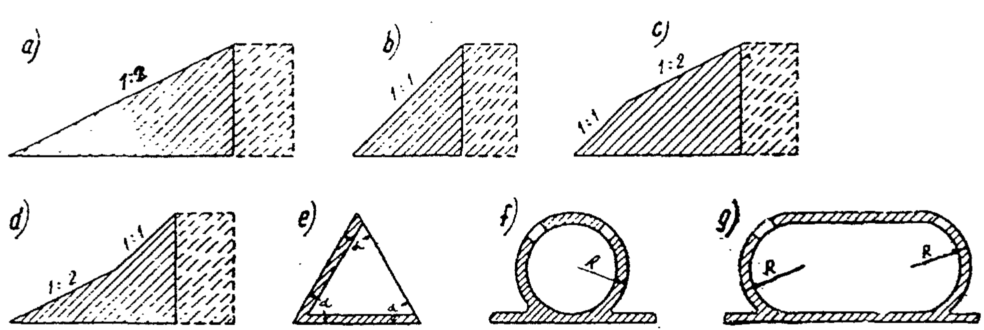


Figure 4.2.B. Cross-sections Tested in Poland, Basiński (1963).

When the structure length becomes greater than the distance offshore, the chance of tombolo formation increases. Locating the breakwater shoreward of the normal breakwater zone can also be conducive to tombolo formation. However, if this location is sufficiently shoreward of the breaker zone, a significant percentage of the total longshore transport will pass bayward of the structure, and the effect on the downdrift shoreline will be reduced. This shoreward location is also advantageous due to the lower quantity of stone required due to the shallower depth. Construction of the breakwater in this location using land based equipment is also possible, particularly if sand is placed in the nearshore zone to facilitate the movement of the equipment. Allowing this sand to remain in the area after construction helps develop the leeward beach planform. Adding additional sand to the area in the lee of the breakwater further minimizes any adverse downdrift effects.

- *Wave Overtopping*

  The basic low-crested design of the reef type breakwater allows part of the incident wave energy to be transmitted by overtopping. Wave overtopping is beneficial in preventing the cuspate spit from attaching to the breakwater and forming a tombolo. As a result, interruption of the longshore sediment transport system is minimized.

- *Breakwater Permeability*

  Another technique to limit the bayward growth of the cuspate spit and the resulting tombolo formation is to make the structure permeable. Part of the wave energy is then passed through the breakwater and limits the advacement of the spit. Eliminating the multilayer design incorporating a core section, used in typical breakwater design, increases the permeability of the structure.

Hueckel (1975) cites results of Russian investigations from which it follows that submerged breakwater should be sited at small distances from shore-line, parallel to it, on a depth of 1.5 to 2.5 m. They used to be rubble-mound structures of trapezoidal cross-section, with smooth flat seaward slopes of 1:3 and steeper on the landward side (1:2).

The surface layer of rubble-mound should consists of large hard rock or concrete blocks which would protect the structure from surf waves and erosion. The mound should be tightly connected to deeper layers. From the present experience, reflected in the subsequent parts of this report, it may be seen that the earlier practices have been incorporated into the thesaurus of contemporary coastal engineering.

Berm Breakwater

The berm breakwater can be regarded unconventional design, dealt with in our Chapter 6 but at the same time it has features in common with "ordinary" break-

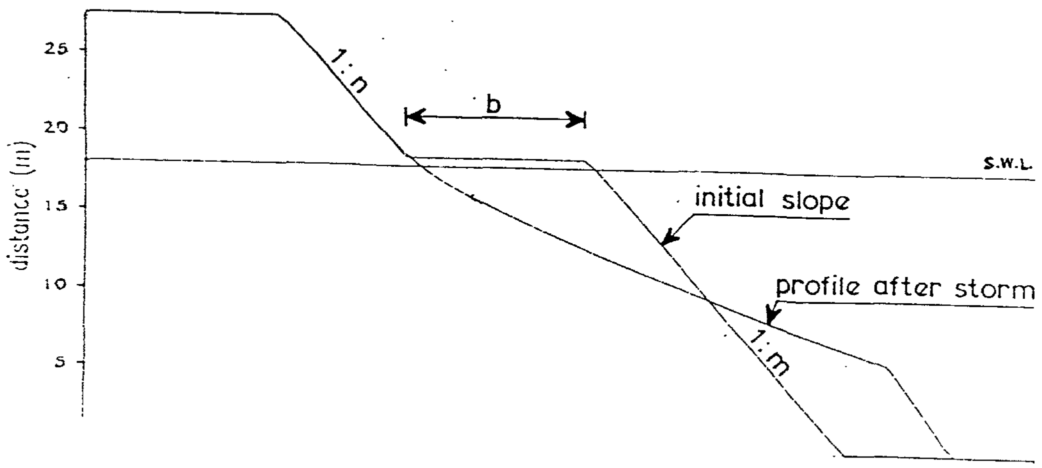


Figure 4.3. Geometrical Dimensioning of Berm Breakwater.

waters; hence it is briefly mentioned here although an example of design, both geometrical and structural seems more appropriate in Chapter Five, to which the reader is addressed.

At this point it is sufficient to emphasize the chief geometrical features. Displacement of armour stones in the first stage of its life time is accepted. After this displacement (profile formation) the structure will be more or less statically stable. The cross-section of a berm breakwater consists of a lower slope 1:m, a horizontal berm with a length b just above high water, and an upper slope 1:n (Fig.4.3). The lower slope is often steep and close to the natural angle of repose of the armour. This means that m is roughly between 1 and 2.

During the design of a berm breakwater the following aspects should be considered: - Optimum dimensions of the structure: m, n, b and crest height, obtained for chosen design conditions - Influence of water depth - Influence of stone class - Influence of wave climate - Stability after first storms.

The development of a computational model on dynamic stability has been summarized by van der Meer (1987). This model was used to design a berm breakwater. The optimum dimensions of the breakwater were calculated with respect to the minimum amount of stone required for construction. The 50-year return period design conditions were used for this procedure. Effects of water depth, stome class, and wave climate were investigated. Finally, the transition from dynamic to static stability was studied in more detail, cf. Ch.5.

Sills and Perched Beaches

Perched beach is a stepped beach. A beach sill, constructed parallel to the shoreline entraps sand brought in by wave action to form a gradual slope near the shoreline.

Some design considerations are presented in a Canadian handbook for coastline property owners and authorities, Ministry...Ontario (1987). The key elements that must be considered in the design are illustrated on the accompanying figure.

The **location** of the proposed shoreline will be chosen to reflect the width of beach required for recreational purposes, however, consideration of the anticipated limit of wave uprush should also be made. Waves, during design conditions, should not overrun the beach. Furthermore, the shoreline should not be located such that the design high water level (DHWL) exceeds two metres at the sill structure.

Once the desired location of the shoreline has been determined, the sill should be located thirty times the difference between the DHWL and the mean summer water level (MSWL) plus 10 metres lakeward of the proposed shoreline (i.e. 30 x (DHWL-MSWL) + 10). Where the natural slope of the beach is significantly steeper than 1:30 (vertical to horizontal distance) this distance can be reduced.

The **top elevation of the sill** should be approximately 0.3 m less than the MSWL or where the sill is located elsewhere than given by the guidelines given above, the top elevation should be equal to the DHWL minus the distance from the proposed shoreline to the sill divided by thirty (i.e., DHWL-(Distance from shoreline to sill) : 30).

A further consideration is **filling of the perched beach**. The beach shoreward of the sill can be allowed to accrete naturally, however, this would deplete downdrift areas of littoral drift. In addition, a stagnant pool of water could be created behind the sill as accretion process continues. The preferable way to create the beach would be to fill the area between the sill and the existing shoreline with imported beach fill (see beach nourishment in respective sections).

Beach sills will accelerate the beach filling process and are particularly valuable for steep beaches where extensive filling would be required in the nearshore prior to achieving significant results above the waterline. The cost of sill construction must be weighed against the extra measure of protection and recreation that will be derived from the new beach.

4.1.4 *Dykes, sea walls and revetments*

Dykes (see Pilarczyk, 1990)

Dike height

The height of a dike was for many centuries based on the highest known flood level that could be remembered. It is evident that in this way the real risk of damage or the probability of flooding were unknown. Little was known about the relation between the cost to prevent flooding and the cost of the damage that might result from flooding.

In the 20th century it was found that the occurrence of extremely high water levels and wave heights could be described adequately in terms of frequency resulting from probabilistic computations. However, the curves of extreme values, based on a relatively short period of observations, mostly have to be extrapolated into regions far beyond the field of observations with the risk for some uncertainties.

After the 1953 disaster, the frequency of the risk of flooding was studied in the Netherlands in relation to the economic aspects. It was eventually decided to base the design of all sea dikes fundamentally on a water level with a probability of exceedance of 10^{-4} per annum. In the Netherlands the storm-surge is mostly incorporated in the estimated water level. If it is not a case, the storm-surge should be calculated separately and added to design water level. Aside from the design flood level, several other elements also play role in determining the design crest level of a dike, viz.

- Wave run-up (2% of exceedance is applied in the Netherlands) depending on wave height and period, angle of approach, roughness and permeability of the slope, and profile shape,
- An extra margin to the dike height to take into account seiches (oscillations) and gust bumps (single waves resulting rom a sudden violent rush a wind),
- A change in chart datum (NAP) or a rise in the mean sea level,
- Settlement of the subsoil and the dike-body during its lifetime.

The combination of all these factors defines the freeboard of the dike. The recommended minimum freeboard is 0.5 m.

Dyke shape

The common Dutch practice is to apply the slope 1 on 3 on the inner slope and between 1 on 3 and 1 on 5 on the outer (seaward) slope. The minimum crest width is 2 m.

The water-side berm is a common element in the Dutch dike construction. It could in the past lead to a reduction in the expenditure on stone revetments (on a very gently sloping berm a good grass-mat can be maintained) and it produced an appreciable reduction in wave run-up.

The present practice aimed at a substantial reduction in wave run-up is to place the outer berm about the design storm water level. If the berm lies much below, the highest waves would not break on the berm and the run-up will be inadequately affected, the grass mat on the upper slope will be too heavily loaded by waves, leading to erosion.

For the storm flood berms at high design levels as in the Netherlands (having recurrence frequency of 10^{-4}) there are in general no problems with the growth of grass on the berm and the upper slope. However , there can be circumstances which also require the application of a hard revetment on the berm and even on a part of the upper slope i.e. when higher frequency of water level is assumed, leading to more frequent overwashing of the upper part by salt water due to the run-up or wave spray (a common grass mat can survive only a few salty events a year).

An important function of the berm can be its use as an access road for dike maintenance.

In general care should be taken to prevent erosion of the grass-mat at the junction with the revetment. The abrupt change in roughness may lead to increase of surface turbulence and more local erosion. It is advisable to create a transition zone by applying cell blocks, geogrids or other systems allowing vegetation.

Sea Walls

Section 2.4 provides details of the far-field effects of sea walls. Section 4.2 discusses structural design of sea walls.

A sea wall is usually meant a shore protection measure with essentially a vertical face (denoting either seawall proper or bulkhead). Sea walls may be employed to protect the eroding bluffs by retaining soil at the toe and by protecting the toe from erosion and undercutting.

The location of the proposed structure must be chosen to balance a number of concerns. Wave forces will be minimized if the shorewall is built as far landward as possible. Where a high steep slope exists on the shore, it will be desirable to construct the structure as far lakeward as practical as as this will improve the stability of the entire slope. In either case, a relatively flat area (five to seven metres wide) should be provided landward of the wall.

Ideally, the top elevation of the wall should correspond to the upper limit of wave uprush (see revetments for design chart). The construction of a wall to a lesser elevation will necessitate the design of the splash pad to withstand the wave action.

Cantilevered structures must penetrate the lakebed to a depth that will prevent overturning of the wall. The rule of thumb is to provide a penetration of 2 to 3 times the free standing height plus the anticipated scour depth (about one wave height below the existing lakebed) where no scour protection is required or provided for any reason whatsoever.

Goda (1985) provides some design principles, which are briefly summarised as follows.

(1) Determination of Crest Elevation

The planning of a sea wall requires comparative designs of several structural types of sea walls. For each design, the crest elevation must be determined according to the wave runup or overtopping characteristics. The crest elevation should be specified in order to give a height above the design storm tide sufficient to prevent wave *overtopping* during the attack of design storm waves.

The crest elevation of a sea wall is determined by one of the following two principles. One is to take the wave runup height as the reference and to set the crest of the sea wall higher than the runup height so that no wave ovetopping will occur. The other is to take the wave overtopping amount as the reference and to set the crest of the sea wall at such a height as to keep the overtopping below some maximum tolerable quantity.

Until recently, the estimation method for the overtopping rate of random sea waves had not been acknowledged by design engineers in general, in part due to insufficient design data. Thus the number of overtopping-based sea wall designs seems to have remained low in comparison to runup-based sea wall designs. However, it is expected that the principle of overtopping-based design will become popular in the determination of crest elevations of sea walls, by making use of hydraulic model tests with irregular waves (Goda, 1985).

(2) Tolerable Rate of Wave Overtopping

The amount of wave overtopping tolerable with regard to the structural integrity of a sea wall is greater than the tolerable amount for the protection of the land behind the sea wall. Even a seavall constructed with utmost care cannot escape the danger of structural failure if it is exposed to heavy wave overtopping for many hours. The mode of failure may be loss of earth-fill from the core of the sea wall by leakage, cracks and breakage of the armoring surfaces of the crown and back slope, or total collapse. The structurally tolerable limit of wave overtopping depends on the type of sea wall structure.

Goda (1985) analysed about thirty cases of coastal dykes and revetments damaged in the aftermath of typhoons, estimating the wave overtopping rate for each sea wall. This yielded an estimate of the maximum tolerable limit of wave overtopping rate as listed in Table 4.3.

From the viewpoint of structural safety, the limits given in Table 4.3 are applicable to sea walls built along embayments and exposed to storm waves a few metres high which continue for a few hours only, since most of the sea walls examined belong to this category. It is believed that the tolerable limit should be lowered for sea walls facing the ocean and exposed to the attack of large waves, or for sea walls subject to many hours of storm wave action.

Table 4.3. Tolerable limit of wave overtopping rate from the viewpoint of structural safety.

Type	Surface armoring	Overtopping rate (m^3/m·s)
Coastal dyke	Concrete on front slope, with soil on crown and back slope	less than 0.005
	Concrete on front slope and crown, with soil on back slope	0.02
	Concrete on front slope, crown and back slope	0.05
Revetment	No pavement on ground	0.05
	Pavement on ground	0.2

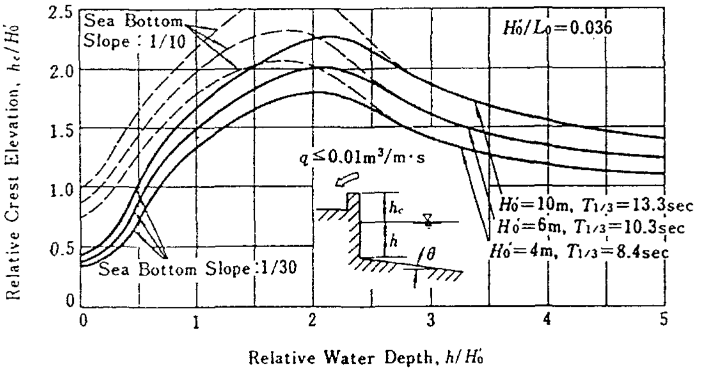


Figure 4.4. Crest elevation of vertical revetment for the condition of overtopping rate not greater than 0.01 $m^3/m.s$, Goda (1985).

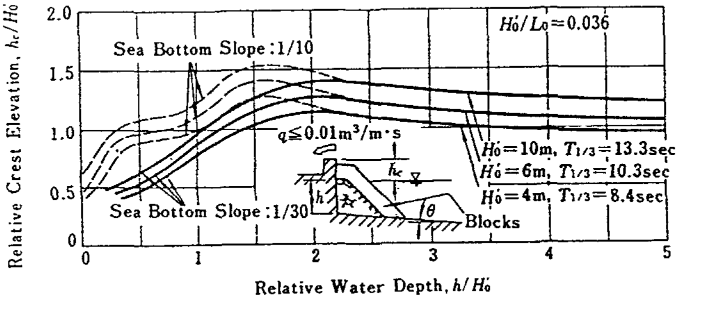


Figure 4.5. Crest elevation of block mound sea wall for the condition of overtopping rate not greater than $0.01m^3/m.s$, Goda (1985).

(3) Determination of Crest Elevation of a Sea Wall

Examples will be given of the decision process for the crest elevation as determined by the criterion of tolerable overtopping rate. By arbitrarily setting the tolerable limit at $q = 0.01m^3/m.s$, the required crest elevation for a vertical wall is illustrated in the following drawings.

The effect of the crown width of a block mound sea wall upon the overtopping rate has been tested by Goda (1985).

Fig.4.8 compares sea walls having front slope of 1:2 with vertical revetments. The model sea walls were set near the shoreline and the wave overtopping rate was measured for several crest elevations. From the resultant data on the overtopping rate, the design crest elevations of both types of sea wall for a given tolerable overtopping rate were estimated and compared. Although Saville (1956) has reported no effect of the front slope of a sea wall upon the runup height when the sea wall is located at the shoreline or on land, the effect of the front slope appears in the amount of overtopping of sea walls which have crests lower than the maximum runup height. As seen in Fig.4.8, a sloped sea wall requires a higher crest elevation than a vertical revetment for a given tolerable overtopping rate. The replacement of a smooth slope with a stepped one can reduce the required crest elevation to the extent that the crest elevation will be only 10 to 20 % greater than that of a vertical revetment, if the height of each step is about 30 cm for an incident significant wave height of about 5 m.

In the practical design of a sea wall, the crest elevation as calculated from the above considerations of wave runup and overtopping is usually given a small amount of additional rise to allow for unknown factors. If some ground settlement or subsidence is expected owing to the foundation characteristics, the estimated amount of settlement must be taken into account in the determination of the final crest elevation.[1]

Revetments

With reference to the possible adverse erosional effects of revetments discussed in Chapter Two, Dean (1986) argues that those effects could be mitigated. The mitigation would be the annual addition of sand to volumetrically compensate for that volume denied at the adjacent shorelines by the armouring.

This concept is illustrated in Fig.4.9, where installation of armouring without any mitigative sand placement will result in adverse effects to the shoreline, but with

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[1] With regard to the structural stability in terms of the near-field phenomena treated in Chapter Three, *inter alia* the perspective of vulnerability to wave attack, precautions should be taken against scouring of the seabed in front of a sea wall and leakage of filling materal in the core of a sloped wall or in the rear of a vertical revetment.

In the design of sea walls, the drainage system for the overtopped water should be well planned, because the overtopping amount by storm waves is quite large.

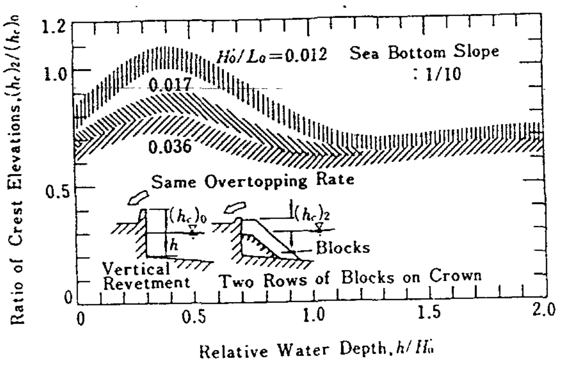


Figure 4.6. Ratio of crest elevations of block mounds to vertical revetments for the same overtopping rate (sea bottom slope of 1/10), Goda (1985).

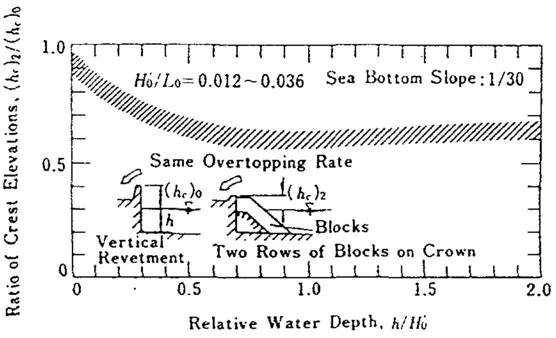


Figure 4.7. Ratio of crest elevations of block mounds to vertical revetments for the same overtopping rate (sea bottom slope of 1/30), Goda (1985).

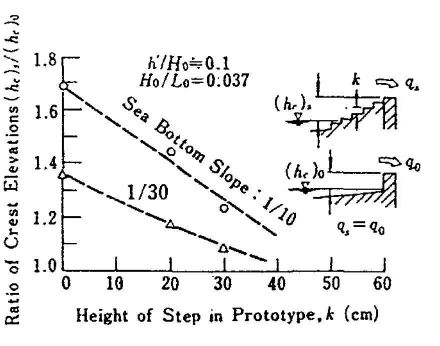


Figure 4.8. Ratio of crest elevations of sloped sea wall to vertical revetment, Goda (1985).

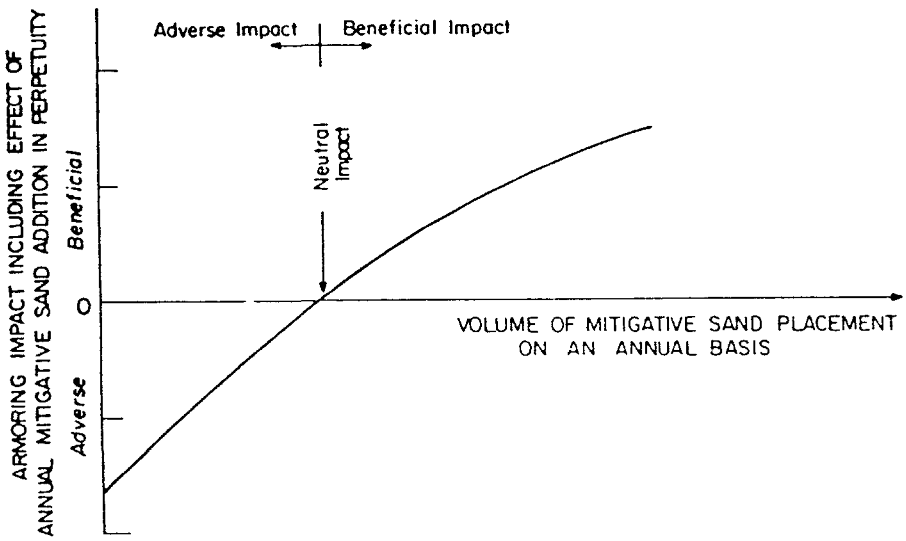


Figure 4.9. Effect of Annual Mitigative Sand Placement, Dean (1986).

increasing annual volumes of sand added, the combination of armouring placed plus mitigative sand added become a benefit. One should indentify the "neutral" point where the annual mitigative sand placement just offsets any adverse effects of the armouring. Two effects are considered by Dean (1986) (1) the reduction in sand supply through prevention of erosion, and (2) the blockage of sediment transport by a projecting revetment.

For the first case illustrated in Fig.4.10, the required annual mitigative sand placement, ∀, to achieve a neutral effect is

$$\forall_1 = (Z_u - Z_l)(R)(l) \tag{4.1}$$

For the second case Dean)1986) considers the profile in the storage area to be the same as that along the unperturbed beach. It is assumed that the updrift impoundment planform is linear and aligned with the incoming waves, see Fig.4.11. The additional annual volumetric storage rate, V_{2_a}, can be shown to be

$$\forall_{2a} = \frac{(B + h_*)}{\tan \Theta} bR \tag{4.2}$$

in which
B = berm height,
h_* = profile closure depth,
R = long-term erosion rate,
b = projection of armouring beyond unperturbed shoreline,
Θ = angle of the wave crest approach relative to the unperturbed beach.
Lacking specific information, a value of $\tan \Theta = 0.1$ appears reasonable. It is noted that the projection distance b increases with time in accordance with $b = b_o + Rt$,

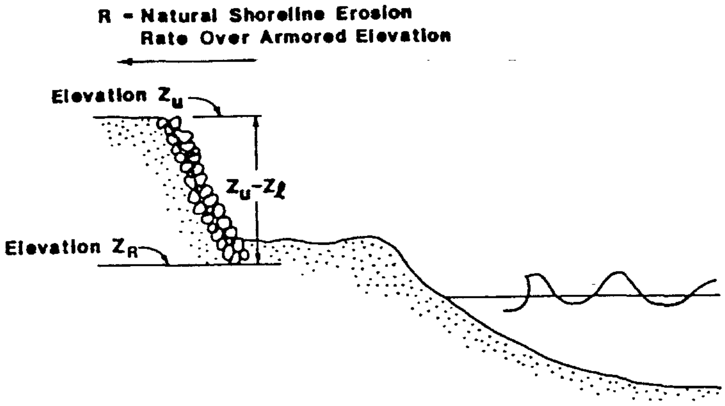


Figure 4.10. Principle of Armouring Mitigation by Prevention of Upland Supply by Erosion.

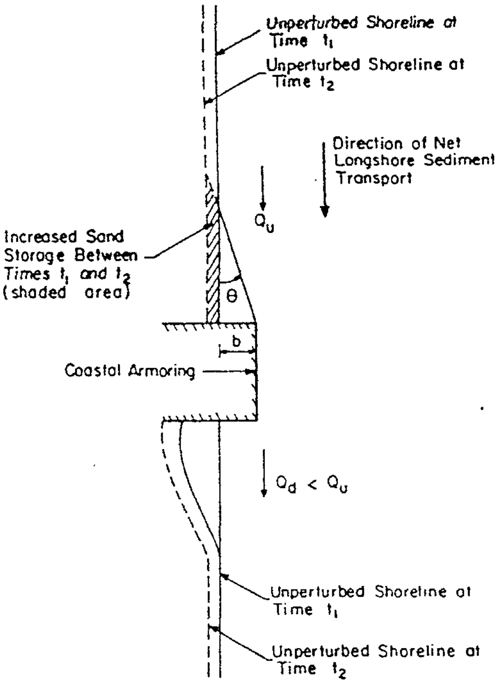


Figure 4.11. Sand Storage by Coastal Armouring, Dean (1986).

in which b_o is the projection at the initial time and t is the number of years into the future.

In another method Dean assumes that the profile modifications extend only out to the solid oblique line shown in Fig.4.11, which shows profiles of the unaffected and assumed affected profiles for the second method. Clearly, the second method represents and underestimate of the impounded volume whereas the first method is an overestimate. The equation for the annual rate of increased volume storage, $\forall_{2b}$, is

$$\forall_{2b} = \frac{1}{\tan\Theta}(B + \frac{3}{5}h')bR \tag{4.3}$$

in which h' is the depth that would be present at the toe of the sea wall if the sea wall were not present. The above equation incorporates the assumption of an equilibrium profile of the form $h = Ax^{2/3}$, in which h is the water depth at a distance x offshore and A is a scale parameter determined for the natural profile of inteerest. The parameter a has dimensions of $(length)^{1/3}$ and for fine to medium sands is on the order of $0.1m^{1/3}(0.15ft^{1}/3)$. Alternatively, h' can be estimated at a distance b along an unperturbed shoreline. As noted before, recognizing that the first and second method for estimating V_2 are too large and too small respectively, it recommended that an average of the two be used, i.e.

$$\forall_2 = \frac{1}{2}(\forall_{2_a} + \forall_{2_b}) = \frac{Rb}{\tan\Theta}[B + \frac{h_* + \frac{3}{5}h'}{2}] \tag{4.4}$$

4.1.5 *Headland control*

Silvester's (1980) general recommendations on the coast stabilization by use of headlands include the following:

- in designing headlands one should aim at providing the shortest possible shore-parallel (frontline) breakwaters (Toyoshima, 1974)
- the distance between headlands must not be too small, the optimum spacing of headlands being 10 lengths of the headland structure. Such spacing allows for the formation of an adequate crenulate shaped bay (cf. Ch.2)
- the distance from the headland structures to the existing shoreline should stimulate the prescribed coast evolution with headland control incorporated therein
- headland control should enforce bay stabilization without causing erosion outside the headland system. Coast stabilization by headland control provides means for forecasting extremum bay indentation, even in the absence of sediment supply.

It must be stressed that the concept of headland control cannot serve a universal remedy for any coastal erosion (accretion) problem. In the writers' opinion Silvester neglects at least a few factors that might modify significantly the headland control concept. For instance, bed evolution in the cross-shore direction cannot be considered at length. Transformations of the cross-shore profiles can be associated with local erosion and eventual net loss of seaward moving material. In plan view, constant wave obliquity may often prove an unrealistic assumption for natural environments with a wide two-dimensional (frequency- -direction) spectrum of incident waves. The variety of wave climates encountered under natural conditions is quite likely to bring about widening indentation of the curved coastline and eventual undermining of the headland structure if detached from shore.

In his inspiration by Nature, Silvester attributes all coastline curvilinearities to the interaction of headlands, coast and more or less regular waves (or swells). Many other factors, such as edge waves, are also felt to be responsible for the creation of crenulate forms.

Silvester (1984) agrees that substantial losses of beaches, in view of the erosion caused by handland downcoast of them, must be allowed for, and are in fact inherently built in the scheme. This voluntary retreat can barely be considered an advantage of the method.

The failure of a headland system due to outflanking, undermining and other modes of scouring at headland structures remains a potential hazard.

4.1.6 *Artificial beach nourishment*

The technology of artificial beach nourishment provides a viable and flexible coast management alternative (CUR, 1987). However, the transformation of shore and beach controlled by beach fills is very difficult to forecast in view of the variety of coastal factors involved. Therefore we shall confine ourselves to some general practical recommendations and address a more advanced reader to special modelling tools if site-specific problems must be solved. In addition to the general overview given in Chapter 2, provided below is a short review by d'Angremond, de Jong and van Oorschot (1988).

One drawback to the application of beach nourishment has often been the risk of underestimating or misinterpreting the quantities of sand to be transported. This poses a potential threat to the economics of the project, particularly in areas where dredging equipment is not readily available.

An answer to this is provided by the probabilistic approah to beach erosion problems.

Numerical morphological modelling and probabilistic calculations have greatly improved risk evaluation for artificial beach nourishment. Sensitivity analysis of the relevant parameters and simulation of various implementation scenarios provide

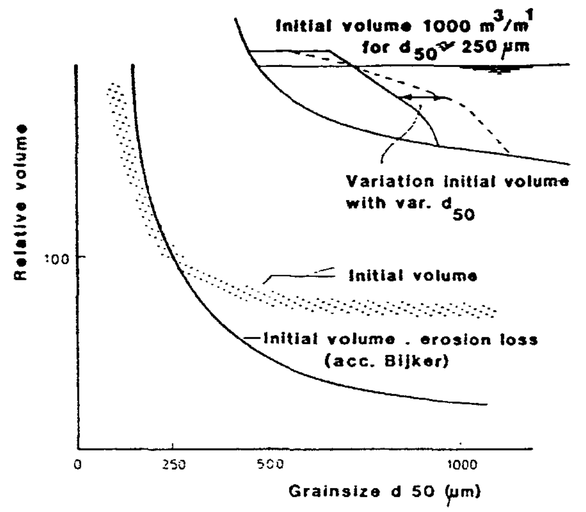


Figure 4.12. Effect of Grain Size on Volume and Erosion Losses, d'Angremond et al. (1988).

reliable tools for initial and maintenance costs.

Planning

In the design of beach nourishment projects, three different types of works may be distinguished:

- direct placement of the sand where it is needed,
- a stockpile of sand,
- continuous nourishment.

The three actually differ only in the design of the fill area. With regard to most other design elements they can be treated in the same manner.

Beach profile and fill material.

Once the layout and planning of the works have been done, the borrow area has to be selected. The most important parameter for the selection of the fill material is the grain size. No matter what the slope is directly after disposal, the sand will inevitably be repositioned by the waves according to the equilibrium profile. The relation between grain size and equilibrium beach profile is provide in a variety of models. If a certain width of dry beach is to be obtained, for either protection or recreational reasons, the total volume of sand that is initially required will strongly depend on grain size. Fig.4.12 shows the influence of this factor for an arbitrary but realistic beach profile. After disposal, loss of sand will occur due to sand--transport phenomena. From consideration of the variations in grain size in the different formulae, it may be roughly concluded that the transport rate decreases more or less proportionally with an increase in grain size.

Other parameters which may play a role in the selection of a borrow area are the density and shape of the material. The density is clearly important, but seldom varies. The shape has only a minor influence on resistance to erosion forces. Rounded grains may have a lower internal friction angle: on the other hand they will settle more easily and the drag forces will be smaller.

Preferably, to avoid rapid erosion of the new material the grain size of the fill material should be equal to or larger than that of the native sand. Yet, in order to avoid the development of unacceptably steep slopes and erosion of the down--drift beach as a result of the decreased mobility of the coarse sand, the difference should not be too large.

A borrow area should meet a number of requirements. The main one is that the area can supply sufficient quantities of sand of suitable quality. At the same time the sand has to be transported to the fill area economically. Therefore, the location and size of the borrow area must be decided on the basis of a soil investigation and on environmental aspects.

To avoid jeopardizing the coastal profile, the borrow area should not be too close to the shore. On the other hand, the price of the fill material will of course increase with distance from the reclamation area.

Reclamation area

When one considers reclamation on the beach, three important aspects emerge:

- the slopes of the reclamation,
- separation of the fill material,
- the accessibility of the reclamation site.

All three aspects are directly related to the amount of fines ($< 63\mu m$) in the fill material. In the first place, the amount of fines depends, of course, on the nature of the material in the borrow area, but it also depends on the dredging method. Direct pumping causes a higher percentage of fines in the dump area than does transportation with a hopper dredger, where the fines will flow overboard during loading of the hopper in the borrow area. The separation mainly depends on the execution of the works and the arrangement of the dumping area is particularly important.

In an open fill area, the further from the discharge point the smaller will be the mean grain size of the settled grains. While moving forward with the discharge point a layer of coarse material will cover the fine material. A part of the fines will flow into the sea. In a closed fill area a larger percentage of the fines can be caught at the low and far end of the reclamation. normally in the foreshore zone. This can result in particular areas which are most sensitive to erosion in the most exposed places. Generally the disposal slopes given in Table 4.4 may be expected.

Table 4.4. Disposal Slopes.

Grain size	Above water	Below water smooth sea	Below water rough sea
Fine ($60\mu m - 200\mu m$)	$1:50...1:100$	$1:6...1:8$	$1:15...1:30$
Medium ($200\mu m - 600\mu m$)	$1:25...1:50$	$1:5...1:8$	$1:10...1:15$
Coarse ($600\mu m - 2000\mu m$)	$1:10...1:25$	$1:3...1:4$	$1:4...1:10$
Gravel ($> 2000\mu m$)	$1:5...1:10$	$1:2$	$1:3...6$

Inventory of sand resources

The construction method will be greatly affected by the location and nature of the source of the sand. The basic possibilities are given schematically as:

- source offshore at considerable distance,
- source offshore at short distance,
- source on land,
- source in sheltered water (often inshore).

Inventory of equipment available

To cope with the particular conditions at a project site, including the borrow area and the reclamation area, an inventory is made of available dredging equipment with its specific capabilities and restrictions:

Work scenarios

When confronted with an actual beach nourishment project, one has to combine all data and site conditions in order to evaluate which combinations of equipment will lead to an optimal solution with respect to cost and risk. The availabity of sand sources combines with the particular properties of dredging equipment; this leads to a number of work scenarios, viz.

(a) Hopper (direct method).
(b) Hopper (rehandle method).
(c) Stationary dredger pumping directly ashore
(d) Stationary dredger loading barges
(e) Portable dredgers

Measurement

One of the most evident risks involved in beach nourishment is the volume of work involved. If the volume of work is uncertain, any contractor will allow a provision for this risk in his offer, however depressed the market. But determining the volume of work may be rather complicated.

In principle, there are several possibilities. A single "best" choice cannot be given as the optimum will largely depend on local conditions, but the main options are:

- through survey of the borrow area,
- through survey of the beach,
- measurement in barge,
- measurement in pipeline.

In most cases, the use of a trailing suction hopper dredger leads to the most economic solution for beach replenishment. Only if the borrow area is at a very short distance from the fill area, or if the borrow area is inaccessible to trailing hopper dredgers, may stationary equipment pumping ashore directly or loading barges become competitive.

The unit rate of a beach restoration operation varies only slightly with the total quantity involved. Only if volumes fall below 4...5 milion m^3 does the cost of mobilisation play a considerable role. In both cases (coarse sand and/or large pumping distance) the use of a booster station is very cost effective.

When using a trailing suction hopper dredger the unit costs are lowest for medium-sized sand (approximately $250\mu m$), but if the effects of erosion losses and required initial volume, both as functions of grain size, are also considered, the optimum shifts towards larger grain sizes (in the example described, from $250\mu m$ to $800\mu m$).

It can be shown that artificial beach replenishment is a technically and economically sound solution for beach protection. The initial capital requirements are lower than for appropriate fixed structures, which makes it a particularly attractive alternative for developing countries.

4.1.7 *Unconventional design*

Coastal structures are often constructed in combinations of different protection measures. Some of them are dealt with in our Chapter 6, from which more conclusions on the morhological dimensioning of single structural systems treated herein can be learned.

4.2 STRUCTURAL DESIGN (Dimensioning) of COASTAL STRUCTURES

4.2.1 *GENERAL*

Structural design should be centred about the following issues

- **overall stability** of structure
- **stability of structural constituents** within the structure
- **strength** of structural constituents

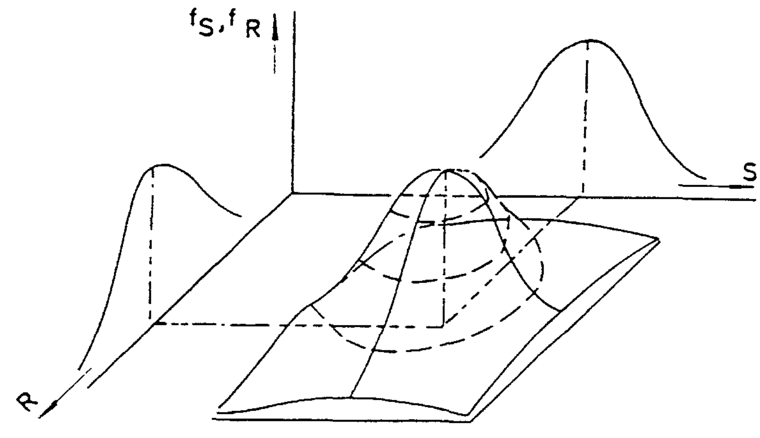


Figure 4.13. Joint Distribution of Loading and Strength.

The stability of entire structure and its individual constituents is endangered by the near-field failure modes discussed in Chapter Three. Accordingly, we are referring to pertinent subsections of that chapter in the present examination of design guidelines arising from those hazards, wherever appropriate and instructive.

Forces, stability and strength (to a lesser degree) are discussed concisely in Section 4.3. The reader is advised to consult it when embarking on structural design after preliminary assessment of near-field hazards dealt with in Chapter Three (and pinpointed in this section).

Examples of particular design features for different types of coastal defence measures are presented in Chapter Five.

Hence the designer, or a coastal manager, is recommended to proceed in the following steps

1. geometrical dimensioning based on evaluation of the far-field phenomena (examined in Chapter Two and summarised in Section 4.1)
2. corrective geometrical dimensioning, and structural modifications due to the near-field failure hazards (as in Chapter Three and Section 4.2)
3. structural design with regard to forces, stability and strength, as stipulated in Section 4.3.

In this section, in addition to the geometrical design in Section 4.1 we are adding quidelines on **structural dimensioning**, reflecting the near-field requirements imposed on entire structures and their constituents.

It is worthwhile to assume the following configuration of this Section:

1. Extension of Probabilistic Design
2. Peculiar Features of Coastal Structures

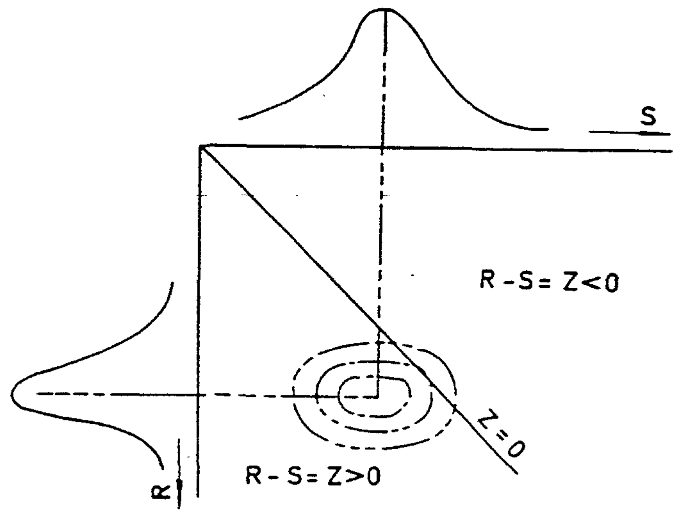


Figure 4.14. Isolines of $f_{R.S}$ and Partial Distribution Functions.

followed by

3. Forces and Stability of Structures and their Components

in Section 4.3.

This layout seems logical as many types of coastal measures have common properties, such as vertical walls, etc. which can be discussed jointly, while on the other hand, some individual features of particular structures and measures may require separate treatment. However, a certain overlapping in items (2) and (3) is hard to avoid. For instance, rock and armour units are widespread in a variety of structures, both logshore and cross-shore.

Accordingly the forces and general stability of such units are dealt separately in Section 4.3. On the other hand, the factors of gravity, interlocking, seepage, suffosion, and many other internal-stability type intertwine in a very special blend if the units are placed in e.g. flexible revetments. Hence it is appropriate to tackle these units and their controlling factors in more detail, which is done under item (2). Therefore the reader is advised to cross-reference rather than to confine himself to a single subsection.

4.2.2 *More on PROBABILISTIC DESIGN*

With reference to the background on deterministic and probabilistic design presented in Section 1.5 we are now adding more details, cf. Leeuwestein (1989), PIANC (1992). In extension of the probabilistic method the two-dimensional or joint probability density function of strength and loading can be used, viz. $f_{R.S}(R, S)$. As a result a probability is obtained for any combination of strength and loading (Fig.4.13). This function forms a more or less undulating surface above the two-dimensional domain, determined by the S and R along the coordinate axes in the horizontal.

The partial probability density functions f_R and f_S are shown in the planes $S = 0$, respectively. In general, S and R together contain n basic variables, resulting in an n-dimensional joint distribution function of these variables $(X_1...X_n)$. It is only for $n = 2$ that $f_{R.S}(R, S)$ can be transformed into $f(x_1, x_2)$, a 2-dimensional joint distribution function of basic variables x_1 and x_2.

In Fig.4.14 the contour lines of the joint probability density of S and R are given together with the partial distribution functions.

By defining the reliability function Z as:

$$Z = R - S \tag{4.5}$$

the state of failure of the structure under concern is determined by:

$$Z = 0 \tag{4.6}$$

In Fig.4.14 this failure condition appears as the plane $R = S$, having a 45-degrees intersection line in the horizontal plane.

The line $Z = 0$ forms the separation between the "safe" and the "unsafe" space determined by $(R, S, f_{R.S})$:

$$Z > 0; \quad safe \tag{4.7}$$

$$Z = 0; \quad failure \tag{4.8}$$

$$Z < 0; \quad unsafe, \quad structure \quad has \quad failed. \tag{4.9}$$

As such the line $Z = 0$ corresponds to the limit state of failure.

Because of $R = R(X_1, X_2....X_m)$ and $S = S(X_m...X_n)$, Z is a function of the strength variables $(X_1, X_2...X_m)$ as well as a function of the loading variables $(X_{m+1}...X_n) : Z = Z(X_1, X_2...X_m.X_{m+1}, ...X_n))$

Stability of a rock slope

The above considerations can be illustrated for the empirical damage relation for loose rock, exposed to breaking waves. The strength and loading can be written respectively as:

$$R = 6.2S_d^{0.2}(g/2\Pi)^{-0.25P^{0.18}}cot^{0.5}\alpha\ \Delta D \tag{4.10}$$

$$S = H^{0.75}T^{0.5}N^{0.1} \tag{4.11}$$

in which
S_d = dimensionless cross sectional damage
P = porosity parameter
D = stone diameter (usually nominal)
Δ = relative density
H = wave height (usually 1% wave)
T = wave period (usually reciprocal of peak frequency)
N = cumulative number of waves.

The static stability criterion is obtained by $S = R$, thereby replacing $S_d = 2$, which has been found to correspond to incipient of damage. Consequently, the failure function for a rock slope can be written as:

$$Z = 6.2 S_d^{0.2}(g/2\Pi)^{-0.25p^{0.18}} cot^{0.5}\alpha \Delta D - H^{0.75}T^{0.5}N^{0.1} \tag{4.12}$$

Stability of a block revetment

The stability of blocks in a revetment is a matter of wave- induced uplift pressures (loading) and block weight (strength). For simplicity, friction has been neglected in this approach. According to the theoretical solution, the strength and loading (in water head) can be written respectively as:

$$R = \Delta D \cos\alpha \tag{4.13}$$

$$S = [\frac{\lambda}{2\tan\alpha\tan\beta}(1 - exp[-\tan\alpha\tan\beta\phi_b/\lambda] + \lambda/2][1 - exp(-2z_1/\lambda)] \tag{4.14}$$

where
Δ = relative density of blocks
D = height of blocks
α = slope angle
β = steepness of wave front
λ = leakage length (permeability ratio of filter - and cover layer/blocks)
ϕ_b = height of wave front (height of impinging wave)
z_1 = internal phreatic water level relative to still water level in front of revetment.

Level III methods

It has been mentioned before that the most extensive probabilistic methods (level III) employ the exact probability density functions of load and strength. The determination of the joint probability density function (j.p.d.f) $f_{R.S}$ for R and S is ultimately based upon the partial probability density functions (p.p.d.f.) of the basic variables $X_1 to X_n$. This is schematically shown in Fig.4.15.

If the loading (S) and the strength (R) are independent (which is often a reasonable assumption) the joint probability density function (j.p.d.f) for R and S can be determined from the partial probability density functions (p.p.d.f.) of R and S:

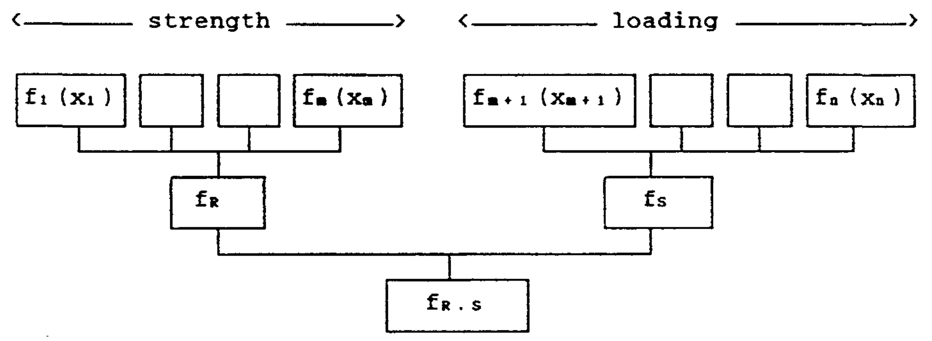


Figure 4.15. Determination of Joint Probability Density Function.

$$f_{R.S}(R,S) = f_R.f_S \tag{4.15}$$

The probability of failure of a structure is expressed by the content determined by the surface of the j.p.d.f. and the horizontal plane, under the condition $Z < 0$. The latter means that the plane $Z = 0$ cuts the part where $Z > 0$ off the "volume" contained by the j.p.d.f. and the horizontal plane. This can be written as:

$$\int_{Z<0}\int f_{R>S}(R,S)dR'dS' \tag{4.16}$$

or, when the assumption mentioned above is allowed (cases where the basic variables are not independent but correlated are discussed at the end of this section):

$$\int_{Z<0}\int f_R(R')\cdot f_S(S')dR'dS' \tag{4.17}$$

On the strength of $R = R(X_1, X_2, ...X_m)$ and $S = S(X_{m+1}...X_n)$
the p.p.d.f. f_R and f_s are determined by the basic p.p.d.f for the variables $X_1.X_2...X_m$ (strength) and $X_{m+1}...X_n$ (loading).

Ultimately, the resulting probability of failure can be determined from the corresponding n-dimensional probability density function as:

$$\int_{Z<0}\int\cdots\int f_1(x_1')\cdots f_n(x_n')dx_1'dx_2'\cdots dx_n' \tag{4.18}$$

Although some casses may allow for simplications of this integral, in general, the function $Z(X_1, X_2, ..X_m X_{m+1}....X_n)$ will have a rather complicated form, thus causing practical problems through the condition $Z < 0$ in the above given integration (eg. the given functions, applying to stability of rock and block revetments).

Besides, often the p.p.d.f. f_1 to f_n are not known to a satisfactory degree. This has led to a number of alternative simplified methods, which will be introduced below. Simplification of the computations involved with Eq.4.14 can be obtained by schematizations of either the p.p.d.f. (f_R and f_s or f_1 to f_n) or the function Z, or both. These possibilities are known under the methods that will be treated below.

Level II methods

The principal methods that are used to overcome the problems in the group of Level III methods can be employed in the category of Level II. The simplifications inherent in these methods make them quite attractive . Three major level II methods are used

- first order mean value approach (f.o.m.v.);
- first order design point approach (f.o.d.p);
- approximate full distribution method (a.f.d);

First order mean value approach

In this method the reliability function Z is approximated by means of linearization around the expected mean values of the basic variables x_1 to X_n. This is done by using Taylor-series expansion thereby neglecting all but the linear terms. Under the assumption of mutually independent variables X_1 to X_n the approximation for Z is written as:

$$Z(x_1, \dots x_n) = Z(\mu_{x1}, \dots \mu_{x_n}) + \sum_{i=1}^{n}[(x_1 - \mu_{x_1})\frac{\partial}{\partial x_1}(Z(\mu_{x_1}))] \tag{4.19}$$

For Guassian-distrubuted variables X_1 the linearized Z has also a Guassian distribution, which can be decrribed by a mean value and a standard deviation, written respectively as:

$$\mu_z = Z(\mu_{x_1}, \dots \mu_{x_n}) \tag{4.20}$$

$$\sigma_z = \sqrt{\sum[\frac{\partial}{\partial x_1}]^2\sigma^2_{x_1}} \tag{4.21}$$

With the mean value and the standard deviation of the function Z the reliability index β can be defined as:

$$\beta = \mu_z/\sigma_z \tag{4.22}$$

This is a the relative distance from μ_z to the limit state or $Z = 0$. Using the normalised Guassian density function of Z/σ_z, the probability of failure, $P_f = PrZ < 0$, can be found as the normalized distribution function $\Phi_N(-\beta)$, from standard tables. In the case of a strong nonlinear reliability function (Z) the errors in the computed probability $PZ < 0$ can be significant. This is due to the unreliability of the value found for σ_z. In such cases the use of a level III method will result in a better approximation than by the other alternatives.

First order design point approach

In this method the function Z is approximated by the same principle, however here the expansion of the Taylor-series is made arount the design point. The design point is defined as the point on the failure envelope (Z=0) where the j.p.d. has its maximum (see Fig.1.36.

This method requires an interative procedure in the case of the failure envelope. $Z(X_1, X_2, ...X_m, X_{m+1}...X_n) = 0$, is a nonlinear function (this will, unfortunately, often be the case).

Approximate full distribution method

Also in this method the design point is used for an expansion of the function Z, but additionally simplified descriptions are used for the p.p.d.f. The latter imply the approximation by equivalent normal distribution functions around the design point. These functions are defined by a mean value corresponding to the design point and standard deviation.

Some examples are elaborated by Leeuwestein (1989) using the probabilistic methods treated in this section; additionally the problem of correlated variables is also dealt with.

Correlated strength and loading variables

When the basic variables are correlated, additional and elaborative procedures have been developed. For a detailed discussion one is referred to Fiessler (1979). Within the first order mean value approach the correlated variables can be treated when the various correlations are contained in the correlation matrix C_x. By transformation of the variables x_i into y_i a corresponding reliability function Z_T, derivatives $\delta Z_T/\delta Y_1$ and standard deviations σy_1 are obtained. The following transformation is carried out: $y_1 = a(x_1 - \mu_z)/b$. The transformation parameters a and b are eigenxvalues of the transponed matrix C_x^T) and the square of the eigenvalue of C_x, respectively. The gradients $\delta Z_T/\delta Y_1$ are related to the original gradient as

$$\delta Z_T/\delta Y_1 = a.b.\delta Z/\delta x_1.$$

Risk analysis

No structure is built without consideration of the costs and benefits of the proposed alternatives. A popular method to compare alternatives is a cost-benefit analysis. After the generation of alternative structures, a decision has to be made on which structure is to be prefered. Apart from the investment, various structure-releated risks may be a particularly important cost factor. This transfers the cost-benefits analysis into a risk-benefit balance to be made for the alternatives. This necessitates the determination of risk-related costs. In this subsection, therefore, a short introduction will be given on the determination of the risk level. According to a

Top level Total failure

Top 1 level Partial failure Partial failure

Sublevel f1 f2 f3 f4 failure 5 failure 6 failure 7

Basic level $S_5 > R_5$ $S_6 > R_6$ $S_7 > R_7$

$i = 1$

Figure 4.16. Failure Systems.

very useful definition, risk is the product of failure probability (p_f) and the consequences of failure (C_f). Only when the consequences (C_f) can be capitalised, the risk can also be capitalised as an expected risk (R_c) during the lifetime (N years) of a structure:

$$R_c = \sum_{n=1}^{N} \frac{p_f C_f}{(1+r)^n} \tag{4.23}$$

In this equation r represents the real interest rate. Comparison of the assessed risk level to the acceptable risk can be used as a criterion to select the best design from a number of concepts.

First, a value for the acceptable risk should be adjusted. Social and ethic aspects play a major role in the adjustment of this risk level so therefore only some general remarks can be made.

Acceptable risk

The first factor of the risk, the probability of failure (p_f), can be defined rather objectively for a designed structure by means of a failure tree. An objective quantatitive definition of the consequences (C_f), however is not so easy. The consequences may vary by dimension (eg. dollars, in the above equation) and may be difficult to relate explicitly to the structure in concern. Also a generally agreed assignment of a quantitative measure to consequences may be impossible. Examples of possible dimensions are:

- cost of repair;
- loss of human life;
- human injuries;
- loss of property;
- loss of investments;
- loss of (expected) future income;
- loss of natural environmental values.

Usually an acceptable risk level is proposed by the project manager or by the society. In practice, adjustment of an acceptable risk level is made on social and political grounds, often mixed with ethical considerations. Any optional structure should thus answer that risk level. The most common practice is to express the risk as a capitalized cost during the lifetime of the structure. The way imponderabilia such as loss of human life and environmental impact can be treated as a special problem. Some rather controversial solutions to this problem have been found.

Risk due to environmental impact can, to a certain extent, be found by using the principle of "substitution project cost". Accounting for human life by using the value of the national product rate, however, is a far less agreed solution.

Risk level assessment

A systematic method to include a likely failure mechanisms into one value for the failure probability (p_f) is the use of a failure tree. In Chapter 1 the fault tree concept has been introduced. Using the results of a fault tree analysis, a risk assessment can be carried out, which subsequently may enable a risk (cost) - benefit evaluation of the design. All possible failure mechanisms are covered by the fault tree. The top of the tree is the event of total failure of the structure, whereas on the lower levels partial failure events and minor failure events are ranged (see Fig.4.16) At the base of the tree (the roots) one finds all possible initial failure mechanisms. These initial or basic mechanisms derive their occurrence from fundamental failures, that are determined from relations of the type $R < S$. The total risk is defined as the product of the probability of total failure (p_t) and the (economic) consequences of total failure (C_t):

$$r_t = p_t C_t \tag{4.24}$$

Similarly, the risks corresponding to the head (on level i), of a k-th partial tree with m different submechanisms (on level i-1), can be expressed as:

$$r_{i.k} = \sum_{i}^{m} p_{i.k.m} C_{i.k.m} \tag{4.25}$$

In general, risks will accumulate with the level. However, loss of functions, not being directly accounted for by the mechanisms considered, will entail an extra risk. Two obvious examples of these indirect risks relate to loss of serviceability and environmental damage, Leeuwestein (1989).

In such cases risk will increase more than due to simple accumulation of direct risks.

An (imaginary) example of risk assessment using a fault tree has been elaborated in Fig.4.17. The semicircular symbol defines an "or" gate, which means that the subordinate systems exclude one another and ranges indicated with "p.m." have ben acknowledged but have further been disregarded.

The example concerns the failure of the front slope of a sea wall.

Consequences (costs, indicated in \$) have been assigned to any of the failure events.

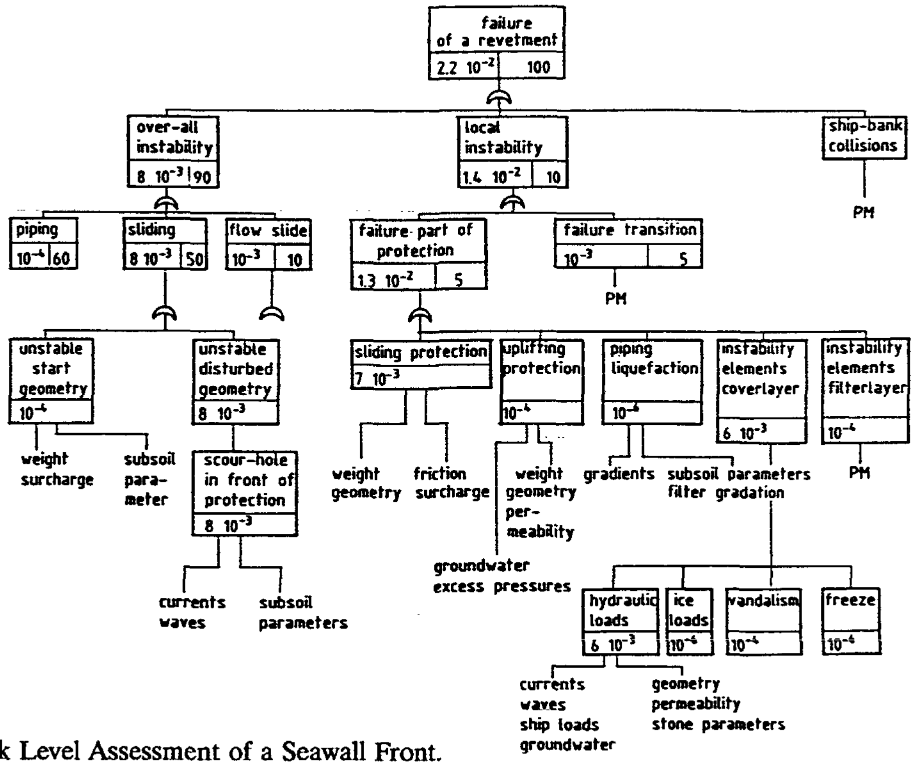


Figure 4.17. Risk Level Assessment of a Seawall Front.

In general the consequences are relatively small for the basic events increase rapidly with higher failure level[1].

4.2.3 *Some STRUCTURAL DESIGN Guidelines*

General

For the sake of clarity we are repeating a few lines identifying the general layout of design, and of our proposals in this document. **Structural** design should be centred about the following issues

- **overall stability** of structure
- **stability of structural constituents** within the structure
- **strength** of structural constituents

The stability of entire structure and its individual constituents is endangered by the near-field failure modes discussed in Chapter Three. Accordingly, we are refer-

[1]Cost of repair and/or replacement of (parts) of the rock slope protection system should be accounted for in the consequences. When the probability is expressed in $[y^{-1}]$ the total risk of failure for the protection system amounts to $1.10^{-1}.100 = 10$ $[\$y^{-1}]$.

Partial failures have a risk of 0.540 (piping), 0.005 (sliding), 0.250 (instability of armour stones) and 0.20 (failure of transitions) respectively (all in $[\$y^{-1}]$. Thus a (preliminary) estimate can be made of the cost-sensitive of design aspects ring to pertinent subsections of that chapter in the present examination of design guidelines arising from those hazards, wherever appropriate and instructive.

Forces, stability and strength (to a lesser degree) are discussed concisely in Section 4.3. The reader is advised to consult it when embarking on structural design after preliminary assessment of near-field hazards dealt with in Chapter Three (and pinpointed in this section).

Examples of particular design features for different types of coastal defence measures are presented in Chapter Five.

For instance, erosion processes undermine the stability and integrity of a groyne or breakwater. Therefore we endeavour to provide in **this section** some tips how to eliminate or alleviate the adverse effects.

CROSS-SHORE STRUCTURES: GROYNES...

In this subsection we propose a few general guidelines concerning cross-shore as well as shore-parallel structures, and resulting from the near-field requirements elucidated in Chapter Three. By and large, a structure as a whole, and not its particular constituents, is dealt with. As already mentioned, more details on particular design features of all types of structures are given in Chapter Five.

Groynes suffer from frequent failures and damage, particularly if they are permeable and constructed of timber piles.

Bakker et al (1984) stipulate that in the case of a groyne system in tidal seas the first check point should be an evaluation of the risk of washing out of piles by shoreward motion of tidal channels.

One might face situations that even the maximum practical pile length will not be enough to guarantee constructional stability.

Hence the depth of pile should make allowance for possible washing of soil and scouring. The allowance should be 1 m in the subaeral part, 1.5 m in the central part and 1 m at heads. The total depth of pile at head is 5 m below sea bed[2].

It is worthwhile to repeat the quotation from Bakker et al (1984)

---

[2]Bakker et al. claim that the top of pile in the central part of groyne should be elevated by not more than 0.20 m above mean sea level. To improve visibility one may elevate single piles some 0.60 m above SWL, with spacing of 10-15 m. The top of the most remote (extreme seaward) pile should be 0.8 m below SWL. The transition to the central part of groyne is done with slope 1:6. The sub-aeral section of groyne is recommended to create a straight line, 1:25 at top, with elevation above beach below 0.2 m. It must be however stressed that numerous investigations on the operation of the groynes have exposed unfavourable effects of the subaeral part due to beach erosion during stroms. Accordingly this segment is abandoned in recent projects.

One may also design uniform slope of 1:25 all over entire length of groyne. This line must start at the extreme seaward pile (-0.8 m below SWL); no pile must protrude by more than 0.2 m above SWL in the central segment or above natural beach.

Based on theoretical and experimental evidence positive effects can be expected; especially in areas with a large tidal range, piles have better effect during a longer part of the tidal cycle than stone groynes with flat berms.

Most of the negative experience can be attributed to constructional failures, which in the future can be avoided. For instance, in Oostkapelle and Europort the piles were too short at the site where the washing-out occurred, because at these locations no heavy erosion had been foreseen. It seems to be of much importance that the screens are well incorporated in the dunes, implying some temporary removeal of sand. At present, mussles are regularly removed from the screens.

Although an objective financial balance of the use of pile screens is hard to make (as the effects are not clear and a financial value can hardly be assigned), the feeling exists, that this balance is positive and that, for instance, it has been a wise decision to protect the Schouwen area with pile screen instead of impermeable groynes.

Pile screens form a much more flexible construction than stone groynes: piles can be added, lifted or removed, and in the case of very heavy erosion one can make - if absolutely necessary- stone berms in a later stage, thus postponing the expense. Furthermore a combination of pile screen and sand supply is feasible.

LONGSHORE STRUCTURES: OFFSHORE BREAKWATERS, SEA WALLS...

Breakwaters

Along the lines of this subsection, **scour** appears as one of the most important factors of geometrical and structural design, cf. Sec.3.1. Reference to other failure modes and their design implications is made in Sec.4.3. Some side issues are noted below.

Bibliographical sources give very reliable information as regards design, but in most cases they only deal with effects connected with wave motion: transmission, reflection and breaking.

Breaking kinematics are very different in the two cases of submerged and emerging breakwaters: for low submerged barriers (i.e. height from the sea bottom less than about 1/3 of the water depth) one usually faces spilling breakers, while for the higher ones one has plunging breakers. Instead, in the case of emerging breakwaters, it is well known that breaking very often occurs with a 'surging' pattern, confirmed in many laboratory tests. The least stable zone of a submerged rubble mound breakwater is always the edge between the top face and the face sloping towards shore.

The behaviour of the foundation is much less explored. Hence a lot of effort should be put in examination of all possible failure mechanisms, and design ofrespective counter-measures.

Tremendous diversity of armour units have been devised. Their applicability is discussed, in terms of stability, in Sec.4.3. It is clear that the usual formulae must be checked and adapted to the case of submerged or emerging breakwater designed. It must also be remembered that not only wave height but also breakwater submergence plays an essential role in the type of breaking. When the breakwater is made of non-interlocking armour units, the usual stability formulae proposed for the case of emerging breakwaters seem to cause overevaluation of the weight of the armour units for submerged breakwaters.

Perched Beaches/Sills

Structural design of the sill should follow procedures set out for revetments using the water depth at the seaward side of the sill in the design.
The sill structure must be designed to retain sand shoreward of the sill. Where the structure is porous, it will be necessary to incorporate proper filters. Even with a non-porous structure, filters may be required beneath the scour pad. Typically, a synthetic filter cloth is used for this purpose. One should consult suppliers for the most appropriate material for the particular application.
The structural design of the scour pad should follow guidelines set out for revetments. The design should, however, use the shallowest slope (i.e. 1:5) possible or economically feasible. As a guideline, the width of the scour pads should be equal to the design wave height used in structural design of the sill. It should, however, be no less than one metre in width.

The beach sill must be designed to withstand both wave and ice action. A variety of materials can be used to construct this sill including wood piling with lagging, steel sheet piling, armour stone or concrete-filled fabric bags.
Gabion baskets are not recommended for beach sill structures as the wire mesh of the baskets are easily damaged by ice and abrasion. Unseen in the water, these wires pose a hazard to beach users.

Sill structures will be subjected to significant ice forces, cf. Sec.4.3. Rock structures will withstand ice forces better than steel piles and other slim structures. In addition, vertical walls will tend to create more reflection than rougher, sloped concrete or stone structures. Increased reflection will increase the potential for scour at the sill structure.

Sea walls

As noted at many places of this document, sea walls may be employed to protect the eroding bluffs by retaining soil at the toe and by protecting the toe from erosion and undercutting.

A sea wall may be either a thin structure penetrating deep into the ground (i.e.

sheet piling) or a gravity structure resting on the surface (i.e. armour stone wall). The thin structures depend on ground penetration to retain the soil behind the wall whereas gravity structures depend on the weight of the structure itself to resist movement of the soil behind the bulkheads.

The location of the proposed structure must be chosen to balance a number of concerns. Wave forces will be minimized if the sea wall is built as far landward as possible. Where a high steep slope exists on the shore, it will be desirable to construct the structure as far seaward (or lakeward) as practical since this will improve the stability of the entire slope. In either case, a relatively flat area (five to seven metres wide) should be provided landward of the wall.

Ideally, the top elevation of the wall should correspond to the upper limit of wave uprush (see revetments for design chart). The construction of a wall to a lesser elevation will necessitate the design of the splash pad to withstand the wave action.

Cantilevered structures must penetrate the sea bed to a depth that will prevent overturning of the wall. The rule of thumb is to provide a penetration of 2 to 3 times the free standing height plus the anticipated scour depth (about one wave height below the existing lakebed) where no scour protection is provided.

Anchored or braced sea walls use an embedded anchor to withstand overturning forces. In this case, penetration should normally be 1.5 to 2 times the free standing height above the anticipated scour depth. The anchors for such a wall usually consist of a row of piles or line of heavy constituents with large surface areas placed a distance behind the wall. This distance should be at least 1.5 metres greater than 1.7 times the total height of the structure (i.e. free standing height plus penetration); cf. Ministry...Ontario (1987). Where practical, the anchor should be located in native, undisturbed soil. The anchor should be placed so that its base is a least two metres below the top level of the wall. The connection between the wall and the anchors must be made of a corrosive resistant materials.

The wall itself must be structurally designed to withstand the full force of waves breaking against it. In addition, it must be designed to resist sliding and overturning. Where the lakebed material is sufficiently firm (preferably bedrock), the sea wall can be constructed as a gravity structure, that is, a structure designed to withstand movement by essentially the weight of the structure itself.

As with the design of the revetment, the scour pad should be designed with a width equal to two times the design wave height but should be no less than one metre in width. The size of stone may be calculated using the procedure set out in the revetment section but utilizing the shallowest slope (i.e., 1:5). It is generally accepted that 300 mm diameter rip rap provides a good materials for many purposes.

The splash pad should be designed as a revetment if water is expected to overtop the crest of the wall. For overtopping wave conditions, the width of the pad should be based on runup conditions. Otherwise, the splash pad should be equal in width to the design wave height but no less than one metre.

Another concern in designing a sea wall is the provision of adequate access for maintenance. A relatively flat area (i.e., less than 1:10 slope) should be provided for a width of five to seven metres behind the splash pad.

Provision for the drainage of the material behind the wall is also important. Surface drainage should be collected and conveyed to the beach in a controlled manner to prevent excessive saturation of the backfill material. Drainage must also be provided through the structure itself from a weeping system location landward of the wall.

For **sea-walls** in a **tidal region**, fronting deep water, the following approximate zones can be distinguished:

I The zone permanently submerged (not present in the case of a high level "foreshore");

II The zone between MLW and MHW; the ever-present wave-loading of low intensity is of importance for the long-term behaviour of structure;

III The zone between MHW and the design level; this zone can be heavily attacked by waves but the frequency of such attack decreases as one goes higher up the slope;

IV The zone above design level, where there should only be wave run-up.

DIKES

The degree of wave attack on a dike (or another defence structure) during a storm surge depends on the orientation in relation to the direction of the storm, the duration and strength of the wind, the extend of the water surface fronting the sea-wall and the bottom topography of the area involved. For coastal areas there is mostly a certain correlation between the water level (tide plus wind set-up) and the height of the waves, because wind set-up and waves are both caused by wind. Therefore, the joint frequency distribution of water levels and waves seems to be the most appropriate for the design purposes (also form the economical point of view). Respective forces in different zones are analysed in Section 4.3.

The gradient of the dyke bank may not be so steep that the whole slope or the revetment can loose stability (through sliding). These criteria give, therefore, the maximum slope angle. More gentle (flatter) slope leads to a reduced wave-force on the revetment and less wave run-up; wave energy is dissipated over a greater length. By using the wave run-up approach for calculations of the crest height of a trapezoidal profile of a dike for different slope gradients, the minimum volume of the embankment can be obtained.[3]

[3]However, this does not necessarily imply that minimum earth- -volume coincides with minimum costs. An expensive part of the embankment comprises the revetment of the waterside slope and the slope and the slope surface (area) increases as the slope angle decreases. The optimum cross-section (based on costs) can be determined when the costs of earth works per m^3 and those of

The water-side berm is a common element in the Dutch dike construction. It could in the past lead to a reduction in the expenditure on stone revetments (on a very gently sloping berm a good grass-mat can be maintained) and it produced an appreciable reduction in wave run-up.
It is advisable to create a transition zone by applying cellular, hollow or any other blocks or units, geogrids or other systems allowing vegetation. The forces exerted on such units, and pertinent stability and strength criteria are presented in Section 4.3.

REVETMENTS

A. GENERAL DESIGN REQUIREMENTS

Revetments constitute a common constituent of numerous coastal defence measures. Their armour units are similar to those used in other versions of coastal design. Therefore this considerable space has been devoted to revetments and their components.

The primary **function** of a slope revetment structure is to protect an existing or newly built slope against the erosive action of waves and/or currents. This function is secured by the design outlined in Fig.1.31.

Referring to Fig.1.31, the innermost layer of a revetment is the **base layer**, consisting of sand, clay, peat, erosive "rock" or similar materials. In general, this base layer is not a part of the protection structure but rather a constituent that **has to be protected.**

This base does not necessarily have to be a layer, but can also be the core of the structure that has to be protected.

On top of this base layer, one or more **intermediate layers** are generally found (an exception is the situation where the revetment's toplayer is placed directly on the clay layer of the base). The function of each intermediate layer is in all cases a combination of one or more of the separate functions mentioned in the following:

- Separation of the base layer, or another underlaying intermediate layer, from the layer(s) laying above. This function is essential when the hydraulic loads on the interface, between two layers can be sufficient to induce washing out of the fines. Such situations arise in the presence of a granular sublayer on top of a core (base) of sand and the presence of a geotextile on the interface of a granular filter and a very open toplayer, with holes bigger than the size of the filter material.

revetment per m^2 are known. Careful attention is needed however, because the revetment costs are not always independent of the slope angle, e.g. for steep slopes the heavy protection is necessary while for the mild slopes the (cheaper) grass-mat can provide a sufficient protection.

Another point of economic optimization can be the available space for dike construction or improvement.

- Reduction of the hydraulic loads on the underlaying layers of the structure. The design of an intermediate layer can in this situation be such that in principle the underlaying layer can be washed out through the intermediate layer, but the design aims at such a reduction of the loads that no erosion will take place during design conditions.

- Reduction of the hydraulic loads on the toplayer. In this situation the design of the intermediate layer will be such that the hydraulic internal loads against the downside of the toplayer are reduced as a result of the proper composition of the intermediate layer.

- The function of the intermediate layer as a flat working floor for placement of regular blocks in a revetment, and the function as a layer of filling material for the placing of a block revetment with irregular thickness (natural stones like basalt) should also be mentioned.

It is the task of a responsible designer to combine the different functions for the intermediate layers as much as is relevant and possible. In this respect the designer should be aware of the consequences of the substitution of one type of material by another. He should be aware that each of the functions of the initial intermediate layer can also be performed by the proposed alternative.

For example, the substitution of a granular filter layer by even a properly selected geotextile will in general replace the separation function of the granular filter, but the load reduction through the granular filter as a result of damping in the relative thick layer will in general not be achieved by the geotextile. The designer should be aware of this possible discrepancy that could easily lead to failure of the structure.

On top of the intermediate layer the **toplayer** can be found. Of course the primary function of the toplayer is to protect the base of the structure from erosion by waves or currents. A secondary, but necessary, function of the toplaiyer is to avoid instability of the intermediate layers by reduction of the hydraulic loads and separation.

Especially the ratio of the permeability of toplayer and sublayer k/k', is decisive for the pressure generation in the revetment, and consequently for the stability of the toplayer.

B. TOPLAYER

STONE AND ARMOUR UNITS, AGGREGATES, MATTRESSES, ETC.

In addition to the discussion in Section 4.3, on forces and stability of stone and armour units of general designation, we will comment on these units, and similar measures, in connection with revetments, i.e. particular area of their application in the form of placed blocks, rip-rap, mats, mattresses and the like.

The methods to calculate the internal and external hydraulic loads have been presented elsewhere in this document.

The stability of the revetment toplayer can be determined if the resulting loads are compared with the strength of the toplayer. This fundamental approach is only available for placed block revetments. For all other types of flexible revetments purely empirical design methods are used. A fundamental approach to toplayer design can be splitted up into strength (I) and stability (II) computations.

(I) STRENGTH OF TOPLAYER (placed blocks)

The strength R (= resistance) of the toplayer of a block revetment has been defined as the maximum allowable upward pressure difference across the toplayer. A minimum value for the strength, R_0, is obtained by considering only the own weight of the structure

$$R_0 = \Delta D \cos \alpha \tag{4.26}$$

in which:
R_0 = permissible water pressure difference (m),
Δ = relative specific mass = $\rho_s/\rho - 1$ (-),
ρ_s = specific mass of block (kg/m^3)
ρ = specific mass of water (kg/m^3)
α = slope angle (degrees).
In addition to the weight of the blocks the strength of the toplayer is increased by:
(a) the friction forces between the blocks as soon as the loads exceed R_0 and
(b) the inertia forces as soon as the loads exceeds the sum of R_0 and the friction forces.

FRICTION FORCES

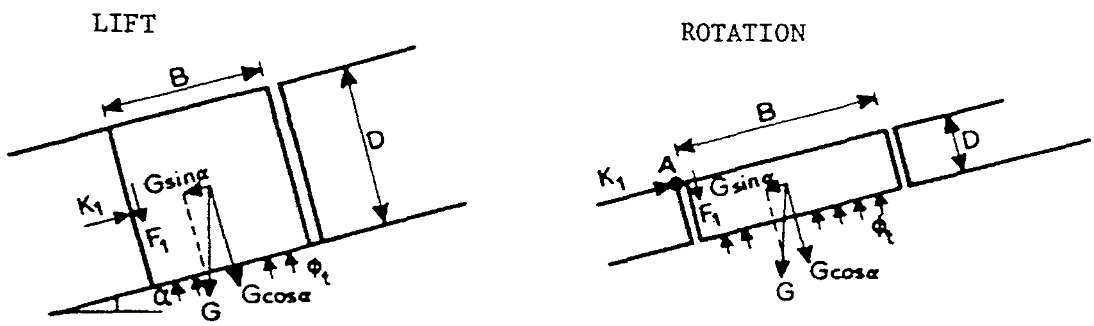
It is known from prototype and large scale model tests on placed block revetments that large friction and clamping forces can develop between initially loose blocks. It concerns, however, a highly stochastic phenomenon. There always remains a chance that loose blocks are present in a revetment. Therefore only the minimum friction forces between such loose blocks and the block one row lower on the slope have been taken into account (see Fig.4.18)

Depending on the block dimensions the friction forces will be mobilised directly when potential instability occurs, or only after a small initial rotation of the critical block.

Elaboration of the mechanics of the various possible equilibrium stages of forces on the block leads to a set of analytical expressions for the strength increase due to friction. Each valid for a specific combination of block dimensions.

The influence of friction on the strength is quantified by multiplication factor r_{s1}. The value of r_{s1} depends on the ratio of block thickness D and block width B.

$$-For \quad D/B > f_b ::: r_{s1} = 1 + f_b.\tan\alpha \tag{4.28}$$



$$W - G\cos\alpha = f_b G\sin\alpha \quad \text{or} \quad W - G\cos\alpha = D/BG\sin\alpha \quad (4.27)$$

for $D/B > f_b$ for $D/B < f_b$

in which:

W = upward hydraulic force = $S_0.BL.\rho g(N)$,
G = submerged block weight = $\Delta DBL.\rho g(N)$,
K_1 = contact force (N),
F_1 = friction force = $K_1.f_b(N)$,
f_b = friction coefficient (-).

Figure 4.18. Principle of Friction Forces on a Loose Block.

(the block slides upward, perpendicular to the plane of the slope)

- For $D/B > f_b$ and $D/B < 2.\tan\alpha$:

$$r_{s1} = 1 + D/B\tan\alpha \tag{4.29}$$

for initial rotation, or

$$r_{s1} = \frac{1 + f_b^2 B/D\tan\alpha}{1 + \frac{1}{2}f_b^2} \tag{4.30}$$

for sliding after initial rotation; whichever r_{s1} proves greater.

For $2\tan\alpha < D/B < f_b$ one has

$$r_{s1} = 1 + D/B\tan\alpha \tag{4.31}$$

(Partial initial rotation)

A graphical presentation of the above equations for $f_b = 0.6$ has been given in Fig.4.19.

INERTIA FORCES

When the loads on a block are sufficient to overcome the weight and the friction forces, the block will be lifted from its place. However, to accelerate the block inertia forces on the block and its added mass of water should be overcome. This

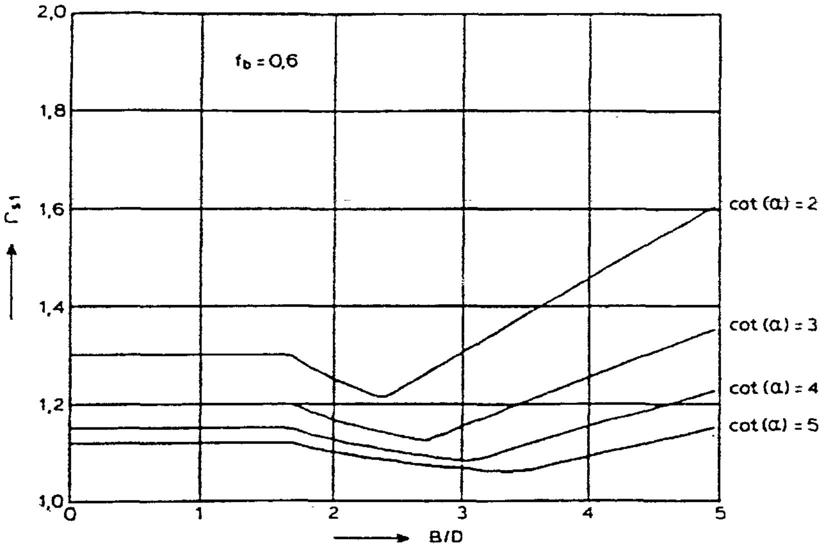


Figure 4.19. Strength Growth Factor due to Friction.

means that, if some block motion is accepted under design conditions, then the effective strength of the toplayer is larger than the sum of weight and friction.

The influence of inertia on the strength is quantified by the multiplication factor r_{s2}. In Fig.4.19 the resultant load on a block is assumed to be a parabolic function of time:

$$F = F_e + 4\hat{F}(t/t_o - (t/t_0)^2) \tag{4.32}$$

in which:
F = resultant load (N),
F_e = load at incipient motion (N),
$\hat{F}$ = maximum of $F - F_e(N)$,
t_0 = duration of excessive load(s), cf. Fig.4.20.

For the moving block the following equation of motion is valid:

$$F - F_e = M.\ddot{x} \tag{4.33}$$

in which:
M = mass of block plus added mass of water = $(\rho_s + \rho).B.L.D(kg)$
$\ddot{x}$ = block acceleration (m/s^2).
(Note that the volume of added water is estimated to be equal to the block volume).

Substitution of Eq.4.33 in Eq.4.32 and introduction of the boundary conditions,viz for $t = 0$ ::: $\ddot{x} = 0$ and $x = 0$ and for $t = t_u$ ::: $\ddot{x} = 0$ and $x = \epsilon.D$ leads to a solution, from which the total strength R of the toplayer can be computed as a product of the own weight R_0 and a multiplication factor r_s according to:

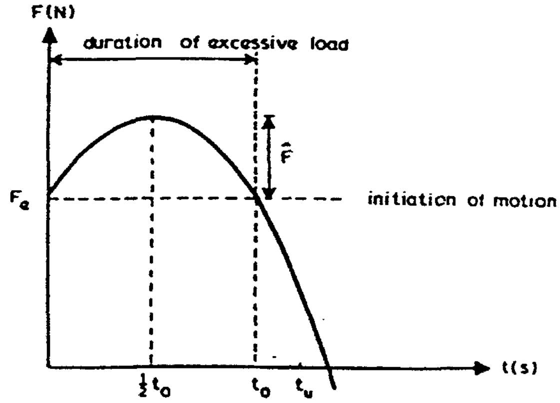


Figure 4.20. Parabolic Representation of Excess Load.

$$R = R_o r_s \tag{4.34}$$

in which:

$$r_s = r_{s1} = r_{s2} - 1 \tag{4.35}$$

(II) TOPLAYER STABILITY

In dimensioning of a stable toplayer a distinction has been made between structures under attack of currents and waves, Pilarczyk (1988).

The simple method derived by Hudson lays the foundations for rubble revetment computations. In the dimensionless form Hudson's solution can be presented as

$$\frac{H}{D_w}\Delta = (K_D \cot\alpha)^{1/3} \tag{4.36}$$

in which H = wave height
$D_w = (G_{50}/\rho_s)^{1/3}$ = nominal rubble diameter (m)
K_D= 3.0 for roughly equal size of rubble elements and K_D = 2.2 for riprap (nonuniform rubble)
α = revetment slope angle
G_{50} = 50% value of the rubble mass distribution curve
$\Delta = \frac{\rho_s - \rho_w}{\rho_w}$=relative density
$\rho_w; \rho_s$ = water and rubble density, respectively.

Since revetments, and in particular riprap, are subject to direct impact of waves and the hydrodynamical instability factors, it seems appropriate to present some methods for revetment computations, although this topic is dealt with separately elsewhere in this study. Some tentative guidelines at this point will make easier, *i.a.* the preliminary design of shore- -parallel structures located close to shoreline.

Van der Meer (Pilarczyk 1990a) modified Hudson's formula by referring to his laboratory tests on revetment stability in the case of stone rubble subject to irregular waves. Wave period has been included through the parameter:

$$\xi_z = \tan\alpha \frac{1.25T_z}{\sqrt{H_s}} \tag{4.37}$$

in which
H_s = significant wave height
T_s = wave period
g = acceleration due to gravity.

Van der Meer's method has been transformed into a table shown in Fig.4.21 For clearly breaking (plunging) waves; $\xi_b < 2.5 \div 3$; the analytical relationships can be given the form

$$H_s/\Delta D_{n50} = 6.2P^{0.18}(S_d/\sqrt{N})^{0.2}\xi_m^{-0.5} \tag{4.38}$$

in which
P = rubble permeability
N = numbers of waves
$S_d = \frac{A}{D_N^2}$
A = area of eroded profile (m^2)

while for surging waves ($\xi_m \geq 3$) one has

$$H_s/\Delta D_{n50} = 1.0P^{0.13}(S_d/\sqrt{N})^{0.2}\sqrt{\cot\alpha}\,\xi_m^P \tag{4.39}$$

As mentioned above, more information on the two methods of computations for revetments (rubble and bituminous), and results of the computations, can be found in the study by Pilarczyk (1985, 1990a and Appendix 5-1).

Currents Alone
A general formula for the strength of units subject to attack by flowing water reads

$$\Delta_m D_n = \phi K_T \frac{0.03}{\Psi_{cr}} K_h \cdot K_s^{-1} \frac{u^2}{2g} \tag{4.40}$$

where:
D_n = thickness of protection unit; for rock one assumes$D_n = (\frac{M_{50}}{\rho_s})^{1/3}$; M_{50} = 50% value of the mass distribution curve of stone; for mattresses(gabions, sand-mattresses etc.) one takes D_n = d = average thickness of mattress.

Δ_m = relative density of protection unit
for rock $\Delta_m = \Delta = \frac{\rho_s - \rho_w}{\rho_w}$
for mattresses $\Delta_m = (1-n)\Delta$
where n = porosity of stone or sand fill, and can be taken approximately as 0.4; for common quarry stone and sand as fill-material for mattresses one has $(1-n)\Delta \approx 1$
$\bar{u}$ = mean velocity; if K_h is neglected ($K_h = 1$), one takes $u_b = u$ = bottom velocity;
K_h = depth factor;
$K_h = \frac{2}{(log12h/k_r)^2}$ for logarithmic velocity profile and $h/k_r > 5$
where $k_r = D_n$ for relatively smooth units (i.e. concrete blocks) and $k_r = 2D_n$ for relatively rough units (i.e. rock)

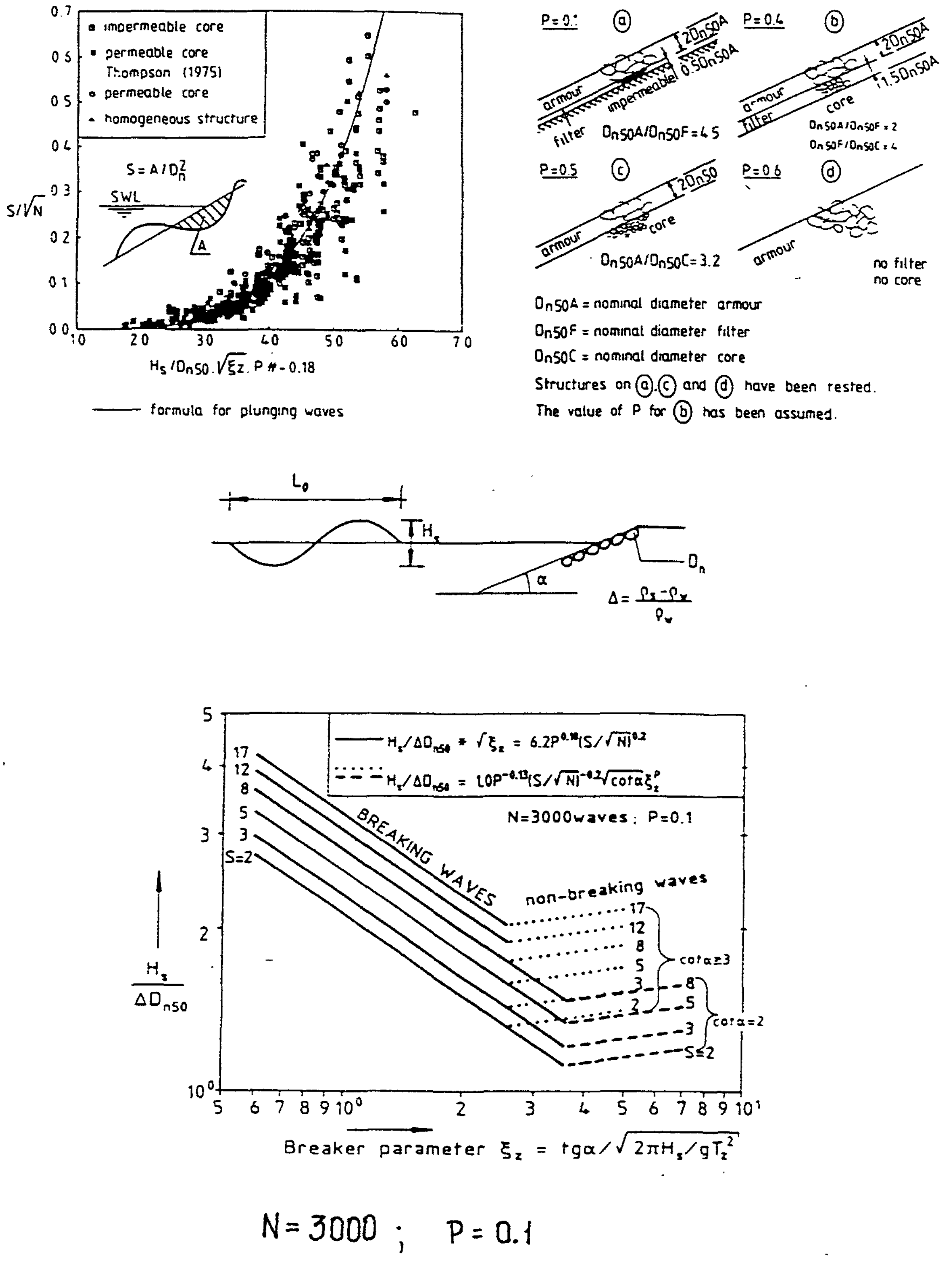


Figure 4.21. Van der Meer's formulae on rip-rap stability for N=3000 waves and an impermeable core (P=0.1).

$K_h \cong (\frac{h}{D_n})^{-0.2}$ for weakly developed velocity profile
K_s slope factor

$$K_s = \cos\alpha(1 - \frac{tan^2\alpha}{\tan^2\Theta})^{0.5} = (1 - \frac{\sin^2\alpha}{\sin^2\Theta})^{0.5} \tag{4.41}$$

For a bed slope in the flow direction with angle (β), i.e. bottom protection beneath a hydraulic structure, the following slope factor holds: $K_s = \frac{\sin(\Theta-\beta)}{sin\Theta}$

Ψ_{cr} = critical shear-stress parameter: $\Psi_{cr} = 0.03$ for rock and $\Psi_{cr} = 0.06...0.10$ for gabions

α = slope angle

Θ = angle of internal friction of granular material

K_T = turbulence factor; K_T = 2/3 for low turbulence and uniform flow; $K_T = 1.0$ for normal turbulence in rivers and $K_T = 2.0$ for high turbulence, local disturbances and outer bends of rivers. $K_T = 2$ is only to use when due to the difficulties in defining the local mean velocity the average mean velocity is being applied.

ϕ = stability factor.

For practical applications the following values of the stability factor ϕ are recommended:

ϕ = 1.25 for exposed edges of loose units
ϕ= 1.0 for exposed edges of block-mats and/or mattresses
ϕ = 0.75 for continuous protection of loose units
ϕ = 0.50 for continuous protection of blocks-mats and/or mattresses.

Examples of the exposed edges are: bottom protection at the scour hole (special hazard in the case of two-directional current i.e. ebb and flood), edges of toe-protection, transitions between adjacent revetment-systems, connections between mats or mattresses.

Practical approach to Wave Attack

Placed/pitched stones and blocks

One has the following relationship (Pilarczyk, 1989):

$$\phi_f \frac{\cos\alpha}{\xi_z} \leq \frac{H_s}{\Delta D} \leq \phi_o \frac{cos\alpha}{\sqrt{\xi_z}} \tag{4.42}$$

This refers to free blocks and stones in older types of revetments in which use has been made of natural friction and interlocking of units

As earlier, one employs

$\xi = \tan\alpha \frac{1.25T_z}{\sqrt{H_s}}$
D = thickness (m)
T_z = mean wave period (s)
H_s = significant wave height (m)

The following figures are suggested as follows

Table 4.5.

Φ_f	Φ_o	Type of revetment
2.0	2.5	poor quality (irregular) pitched stone
3.0	3.5	good quality (regular) pitched stone
3.5	4.O	natural basalt and less permeable (closed) blocks
4.0	5.0	relatively permeable blocks (open area 5...0 %)
5.0	6.0	closed blocks on good quality and smooth clay - surface

Notes: 1. The open blocks in line 3 have more reserve stability than closed blocks;
2. Lines 1 through 4 hold for granular sublayers;
3. Blocks on geotextile and sandy subsoil hold only up to $H_s = 1.2\ m$ (max. $H_s = 1.5\ m$) due to danger of liquefaction.

Block-mats and grouted/interlocked systems

One takes

$$\frac{H_S}{\Delta D} \leq \phi \frac{\cos\alpha}{\sqrt{\xi_Z}} \tag{4.43}$$

with Ψ= 3.5 for blocks connected to geotextile by pins
Ψ = 4.0 for grouted blocks connected by geotextile
Ψ = 4.5 for cabled closed blocks
Ψ = 5.0 cabled upon blocks; grouted basalton $\Psi \geq 6.0$ for grouted cabled blocks; mechanically interlocked blocks.

The above remarks for Φ_f... hold.

The edges of adjacent blocks-mats, if not properly connected, should be treated as free blocks (see case a), with $\Phi_f \approx 3$).

In all cases, the experience and sound engineering judgement play an important role in applying these design rules; otherwise mathematical or physical testing can provide an optimum solution.

Loose aggregates & riprap on less permeable subsoil (i.e. sand)

In this case one has

$$\frac{H_S}{\Delta D_n} \leq \frac{\phi}{\sqrt{\xi_Z}} \tag{4.44}$$

$\cot\alpha > 2$; ξ_Z¡ 3 (breaking waves); $N \leq 3000$ waves.
where $D_n = (M_{50}/\rho_s)^{1/3}$; $M_{50} = 50\%$ exceedance of mass of stones
ϕ = 2.25 for incipient movement
ϕ = 3.0, for maximum tolerable damage in a 2-layer system.

A more sophisticated general approach for loose materials has been provided by van der Meer (198). He has considered plunging breakers with $\xi_z \leq 2.5$; duration of wave attack, damage level and permeability of underlaying layers, and has put forth

$$\frac{H_S}{\Delta D_n} < 6.2 \cdot p_b^{0.18} \cdot (S_b^2/N)^{0.1}/\xi_z^{\frac{1}{2}} \tag{4.45}$$

where:

P_b = permeability factor related to the core of the protected body; being P_b = 0.1 for practically impermeable cores (i.e. sand-/clay-body) and P_b = 0.5 for permeable (granular) core (i.e. in breakwaters)
N = number of waves
ξ_Z= breaking parameter
D_n = nominal diameter
Δ =relative density of stone
s_b = damage level.

The physical meaning of S_b is the number of cubic stones with a side of 1 x D_n, eroded over a width of $1 \times D_n$. The 'no-damage' criterion is taken generally as S_b between 1 and 3 (stones eroded).

Grouted aggregates (bitumen grouting)
One takes

$$\frac{H_s}{\Delta D_n} = \Psi_u \frac{\Phi}{\sqrt{\xi_z}} = \Psi_u \frac{2.25}{\sqrt{\xi_z}} \tag{4.46}$$

with ϕ_u = upgrading-factor due to grouting; being $\phi_u = 1.50...1.65$ in the case of pattern grouting (about 60% of the total surface is filled).

Gabion (rock) mattresses

The existing stability criteria for necessary thickness (d) of gabion result from small-scale tests, viz.

for sliding:

$$\frac{H_s}{\Delta d} < 2(1-n)\cot\alpha; \quad for \quad cot\alpha < 3 \tag{4.47}$$

for wave resistance

$$\frac{H_s}{\Delta d} < 4(1-n)(\cot\alpha)^{1/3}; \quad for \quad \cot\alpha > 3 \tag{4.48}$$

The latest equation gives, versus Hudson's K_D = 3.2 for loose materials, an upgrading of stability by a factor of $4 : (3.2)^{1/3} = 2.7$.

Transformation into a more general form provides:

$$\frac{H_s}{(1-n)\Delta d} \leq \Psi_u \cdot \phi\xi_z^{-0.5} = \Psi_u \cdot \frac{2.25}{\sqrt{\xi_z}} \tag{4.49}$$

The value of the upgrading factor is recommended as $2 \leq \Psi_u < 3$ (max).
By introducing for $\Psi_u = 2; d = 1.8D_n$ and n = 0.4 one obtains

$$\frac{H_S}{\Delta D_n} \approx 2.16\frac{2.25}{\sqrt{\Psi_Z}} = \frac{5}{\sqrt{\Psi_Z}} \tag{4.50}$$

in which (D_n = size of fill-rock).

This means that the level of wave loading in the basket is 2.16 times higher than acceptable loading for incipient movement of loose rock. Using more than 2 layers in a system with a finer stone leads to heavier movement of stones causing deformation and extra loading on the wires. Therefore it is recommended to calculate the size of stone as if it were in a 2-layer system even if more layers are applied.

Open stone asphalt (Fixtone)

One has Δ = 1.15

The existing criteria are summarized as follows:

1° U_{max} = 6...7 m/s (maximum velocity in respect to surface erosion)

2° thickness d= $C.H_S$ with C = 1/6 - on filter cloth, and C = 1/10 - on sand-bitumen filter

3° From Delta Flume: $\frac{H}{\Delta d} \cong 8.4$ - on sand asphalt and 6.7 on filter cloth and sand; with the no-damage criterion of $\xi = 1.2$

In particular, assuming $H = 1.5H_S \rightarrow H_S/\Delta d = 5.6$ and 4.5 respectively one has

Ad. 1° : $\quad U = \Psi\sqrt{gH}\xi^b; \quad 0.25 \leq b \leq 0.50; \quad \Psi = 1$

$$H = \frac{U^2}{\Psi^2 g\xi^{2b}} = \frac{6^2...7^2}{1 \cdot 9.81\xi^{2b}} \cong 4.5\frac{1}{\xi^{2b}} \tag{4.51}$$

For $\Psi = 1 \& U_{max} = 7m/s$ this is equivalent to $H \cong 4.5\ m$ or H_s = 3.0 m.

Ad. 2°: $\frac{H_S}{\Delta d} = 1\Delta C = \frac{0.87}{C}$ = 5.22 on filter cloth and sand or 8.70 on sand bitumen filter.

In conclusion one may postulate that the criterion

$$\frac{\phi}{\xi_z^{2/3}} \leq \frac{H_S}{\Delta d} \leq \frac{\phi}{\xi_z^{0.5}} \tag{4.52}$$

with $\phi \cong 5$ seems to be a responsible choice. Because of possible liquefaction the open stone asphalt on geotextile placed on sand this criterion is recommended only up to $H_s < 2\ m$. For $H_S \geq 2$ m the sand-bitumen filter is recommended. Because of the limitation due to the surface erosion this system may be applied up to H_S = 3 for a less frequent loading up to H_S = 4 m.

Fabric (geotextile) mattresses

The stability criterion for fabric mattresses of thickness d filled with sand or sand cement attacked by wave is derived from some (limited) tests and the recent knowledge on revetments principles, viz.:

$$\frac{H_S}{\Delta d} \leq \phi(1-n)\xi_z^{-0.5} \tag{4.53}$$

or

$$\frac{H_s}{\Delta_m d} = \frac{\phi}{\sqrt{\xi_z}} \tag{4.54}$$

for ξ_z¡ 3 (breaking waves)
For granular fill-materials one has $\Delta_m = (1-n)\Delta \cong 1$,
where Δ = density of fill material and n = porosity (m = 0.4).

The value of ϕ depends on the ratio of the permeability of the mattress and the permeability of the subsoil. For values of this permeability ratio much smaller than 1, uplift pressures become important. A safe value for ϕ is 2.

For nearly equal permeabilities ϕ-values of 2 (to 4) may be applied. Local uplifting may then occur. If the permeability of the mattress is much greater than that of the subsoil, the uplift pressures are of minor importance, then a stability factor ϕ equal to 4 is considerated to be safe.

However, in the latter case the internal instability of the subsoil might become the decisive factor. For the wave height $1.2 < H_S < 2.0$ and sandy subsoil special measures against sliding and/or liquefaction should be taken (extra compaction, extra thickness, granular sublayer 0.2 m in th case of sand-cement, etc.).

Special attention should also be paid to the construction and the transitions which need special measures, expecially against overtopping and/or scour at the toe. High deformation of the mattress may be expected. Above the wave height of 2.0 m the application of the fabric mattresses should be supported by large-scale tests or prototype studies.

In view of the uncertainties involved the use of the ϕ-value higher than 4 is not recommendable. Moreover, the permeability of the mattress should be treated as an integrated permeability of the mattress, and not only that of the components involved (geotextile, fill-material...). Sand and/or gravel are very suitable as fill material, possibly mixed with cohesive additives. Mixing the fill with seed can promote vegetation.

C. SUBLAYERS or INTERMEDIATE LAYERS

It will have become clear from the foregoing material that a large variation exists in types of intermediate layers; each type having its own characteristic behaviour and its own mode of failure. As specified earlier, the following distinction is made

- granular materials;
- sheet-type materials (like all kinds of geotextiles) and
- cohesive sublayers.

In the following paragraphs design procedures will be presented for the granular materials and for the geotextiles, as far as the separation function is considered. So the only mode of failure treated is the granular failure of the structure. Attention has been given to

- The erosion of granular filter material through the holes in the toplayer.
- Internal instability of a granular layer.
- Erosion of a granular base layer through a geotextile.

All other modes of failure for the sublayer will be ignored in this document. No generally applicable design rules are available. Still the designer should be aware of the other failure modes.

WASHING-OUT OF GRANULAR SUBLAYER THROUGH THE TOPLAYER

From the designer's point of view it can be atttractive to minimise the leakage factor of a slope revetment . In this way the resulting forces on the toplayer will be reduced. One of the possibilities to reduce the leakage factor is to increase the permeability of the toplayer, for example by the application of a toplayer with large holes in the blocks. In case the holes in the toplayer are larger than the grains in the under laying filter layer, this sublayer may be washed out through these holes. Two different mechanisms can be distinguished

1. Erosion by outflowing water through the holes by the flow perpendicular to the toplayer
2. Erosion by the external water flow up and down the slope by wave impact, runup and rundown.

The first mechanism will take place during the moment of maximum rundown; the pressure gradient over the slope induces an outgoing water flow that may be able to transport the individual grains of the filterlayer.

The second mechanism can occur during or just after the moment of wave impact or during the running down of water on the slope. This downrush generates eddies in the holes in the toplayer, by which the individual grains are eroded from the sublayer. Of course, also a combination of both mechanisms is possible.

For larger holes, the flow velocities of the outflowing water are reduced as a result of the large cross-section and the reduction of the gradient over the toplayer. In this case mechanism 1 is unlikely to occur, while mechanism 2, as a result of the larger hole diameter is more likely to arise.

INTERNAL INSTABILITY OF SUBLAYERS

A granular filter is called internally stable when the fine fraction of the filter cannot be washed-out between the larger grains. The mechanism of internal instability is often referred to as suffosion or internal erosion. Internal instability can only occur within materials with a very wide gradation (gentle sieve curve). This type

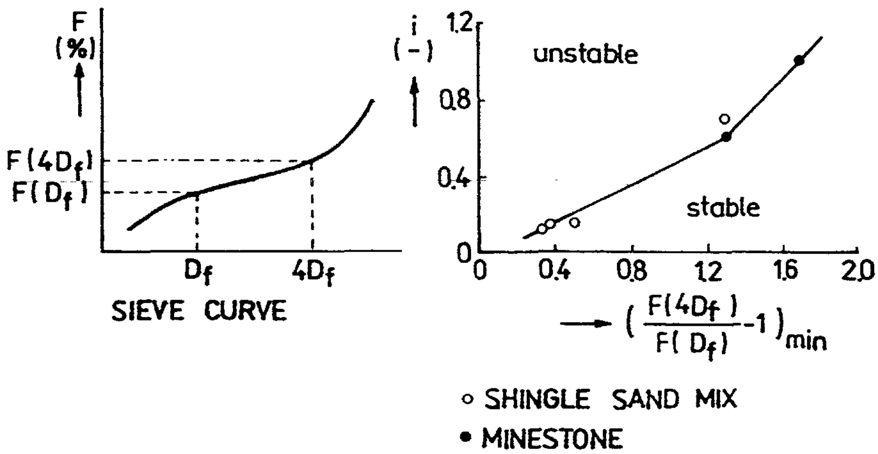


Figure 4.22. Internal Stability of Granular Material.

of material is used more and more widely in slope protections, where industrial waster materials (slags, silex, minestone) become a cheap alternative for natural granular materials.

Internal instability is a dangerous process for slope revetments. A gradual increase of the permeability of the filter layer leads to

- higher internal pressures in the filter layer, endangering the stability of the toplayer

- Instability of the interface between filter layer and base material, which can lead to undermining of the entire structure.

Therefore internal instability should be avoided or the consequences should be taken into account in the design. Kenney and Lau (1985) have developed a method to evaluate the internal instability as a function of the hydraulic gradient. The ratio of the large and the fine portion of the most gentle part of sieve curve (on linear scales) appears to be decisive for internal instability. In practice internal instability occurs only within the 20 % (m/m) of the finest fraction of the filter material. Therefore this percentage makes up the basis of our analysis.

The method is illustrated in Fig.4.22.

Independent of the hydraulic gradient, suffosion will not take place for $[F(4D_f)/F(D_f) - 1]_{min} > 2$

EROSION OF THE BASE LAYER INTO THE GRANULAR SUBLAYER

Erosion of the base material into the filter layer can easily be avoided with a geometrically closed filter layer. This means that it is physically impossible that a grain from the base material is transported into the voids of the filter layer.

According to Huis in't Veld et al (1984) the following strict rules apply to different types of materials

- For wide graded filters: $D_{f15} < 5.D_{b85}$
- For very fine base material: $log(D_{f10}/D_{b10} - 2) < 1.9/log(10^6 D_{b10} - 1)$

in which:
D_{fx} = grain size of filter exceeded by x % (mm) material (m)
D_{bx} = grain size of base exceeded by x% (mm) of material (m)
n = porosity of filter (-).

Geometrically open filters can also be stable but then the hydraulic loads should be taken into account. Klein Breteler (1988) developed a set of modern empirical **filter rules**, where the filter material (D_{f15}) is selected on the basis of the hydraulic gradient perpendicular to the slope plane (i), the base material (D_{b50}), the filter porosity (n) and the slope angle (α).

The formulae for filter design are quite complicated. Therefore a graphical presentation is given in Fig.4.23, for a base of normal sand (specific weight $\Delta_b = 2,650\ kg/m^3$) and water viscosity $\nu = 1.2.10^{-6} m^2/s$.

For placed block revetments one has $i = \tan\alpha$. For the calculation of hydraulic gradient parallel to the slope plane, a distinction should be made between the downward and the upward gradient:

- $i_{down} = \sin\alpha$
- i_{up} can be calculated with a rather complicated analytical formulae, that has been presented in a graphical form in Fig.4.24.

EROSION OF THE BASE LAYER UNDERNEATH A GEOTEXTILE

Instead of using a stable granular filter layer the application of a geotextile as a separation layer can be considered. For geometrically closed geotextiles the following rules quarantee that practically no sand from base will be washed out:

- For stationary hydraulic loads: $\Phi_{90}/D_{b90} < 1.0$ (for "woven" geotextiles)
  $\Phi_{90}/D_{b90} < 1.8$ (for "non-woven" geotextiles)

- For dynamic (cyclic) hydraulic loads: $\Phi_{98}/D_{b15} < 1.0$ (if all grains should be halted)
  $\Phi_{98}/D_{b85} < 1.0$ (if some washing out is acceptable leading to a natural filter in the upper base layer).

In general the latter criterion is sufficiently rigorous.

Studies by the Dutch Association "Kust- en Oeverwerken" (1979) have shown that

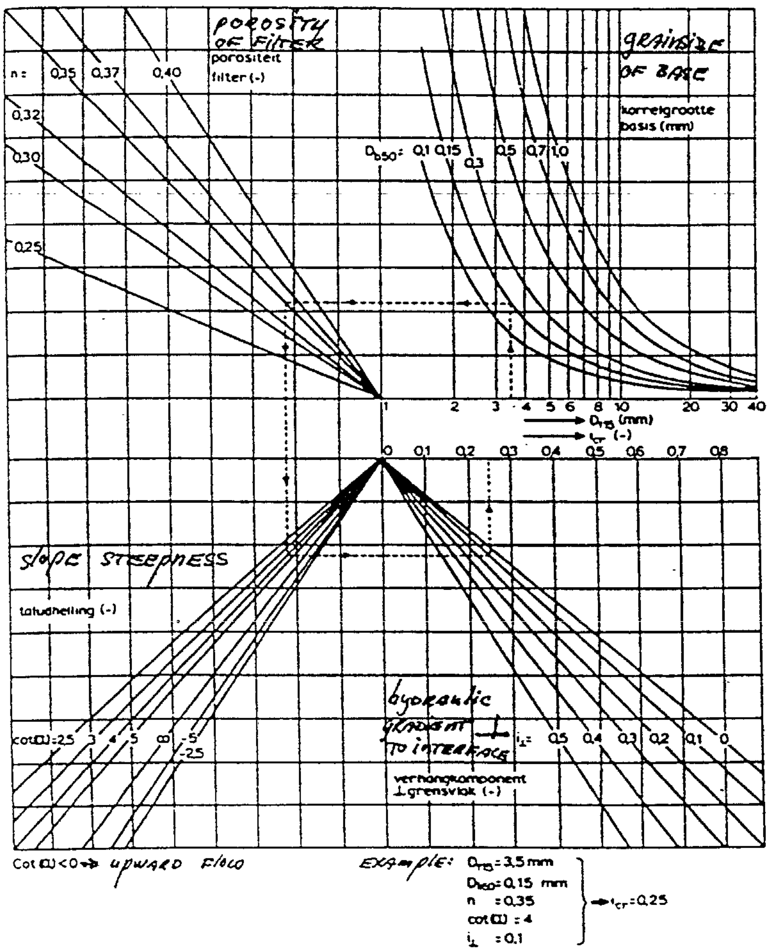


Figure 4.23. Design Graph for Granular Filter Layers.

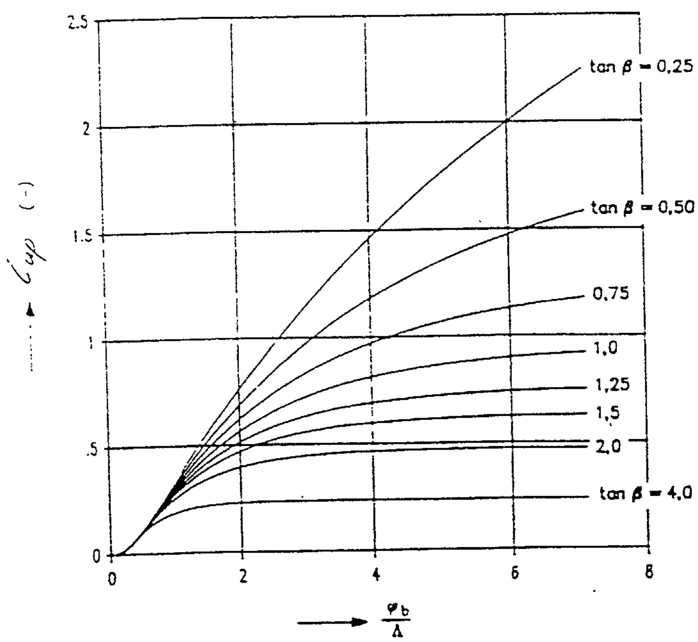


Figure 4.24. Maximum Parallel Upward Gradient in Filter.

Table 4.6

D_{b50} mm	c -	m -	Ψ_s -	Φ_i (°)
0.1	1.18	0.25	0.11	60
0.15	0.78	0.20	0.073	60
0.2	0.71	0.18	0.055	60
0.3	0.56	0.15	0.044	55
0.4	0.45	0.11	0.038	55
0.5	0.35	0.07	0.036	55
0.6	0.29	0.04	0.035	55
0.7	0.22	0	0.034	50
0.8	0.22	0	0.034	50
1.0	0.22	0	0.035	50

geotextiles that are too open, in terms of the traditional strict design rules, often perform perfectly well in practical situations. Therefore recently Klein Breteler (1988) developed a new selection criterion for geotextiles in which not only the grain size of the sand from the base, but also the hydraulic gradient, i_{cr} or the filter velocity v_{fcr} is taken into account.

The critical filter velocity can be computed with the following formula:

$$v_{fcr} = \Theta[12(\frac{D_{b90}}{O_{90}})^4 \frac{T_g}{D_{b90}}(\frac{w}{k_g})^{\frac{1}{2m}} + \frac{n}{e}]\sqrt{\psi_s \Delta_b g D_{b50}} \tag{4.55}$$

in which:

Θ = coefficient; $\Theta = 0.8$ for bottom protections; $\Theta = 0.5$ for block revetments (-)
V_{fcr} = filter velocity for incipient motion (m/s)
D_{b90} = 90% grain size of base material (m)
Φ_{90} = 90% opening size of geotextile (m)
T_g = thickness of geotextile (m)
Ψ_s = Shields parameter (see Table 4.6) (-)
Δ_b = relative mass of sand ($= p_s/P_w - 1$)
g = acceleration of gravity (m/s^2)
k_g = permeability of geotextile ($v = k_g.i^m$) (m/s)
m = power in formula for k_g (see Table 4.6) (-)
n = porosity of filter (-)
e = $f.Re^m$ (see table 5.1) (-)
Re = Reynolds number ($= v_{fcr}.D_{f15}/\nu$) (-)
w = fall velocity of sand (m/s).

The critical gradient i_cr can be computed from the critical filter velocity by the following formula:

$$i_{cr} = 160\frac{\nu}{g}\frac{(1-n)^2}{n^3 D_{f15}^2}v_{fcr} + \frac{2.2}{gn^2 D_{f15}}v_{fcr} \tag{4.56}$$

The critical gradient should be compared with the actual gradient.

The method is applicable to structures with slope angles $\cot\alpha > 3$ and a base of fine sand ($D_{50} = 0.1...0.2mm$).

4.2.4 *Scour protection*

Toe protection is supplemental armouring of the beach or bottom surface in front of a structure which prevents wave from scouring and undermining. Factors that affect the severity of toe scour include wave breaking (when near the toe), wave run-up and backwash, wave reflection, and grain size distribution of the beach or bottom materials.

Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Toe scour is a complex process. Specific, generally valid guidance for scour prediction and toe design based on either prototype or model results has not yet been developed, but some general guidelines can be ventured; they are exemplified in Chapter 5.

The principal methods of reducing or preventing the scour of bed materials involve reducing the hydrodynamic forces on the bed, and/or isolating the bed materials from those forces.

The main measures adopted may be summarised:
(a) reduce forces by reducing reflections, often by the construction of an energy-absorbing mound or berm;
(b) isolate the problem area close to the structure by a scour control blanket, often of rock fill, preformed flexible mats or gabion mattresses;
(c) reinforce the bed foundation material by full, partial or local grouting using cement or asphalt material.

The most common measure for scour protection is the provision of a scour protection blanket of rock fill.

In the design of new or rehabilitated structures the use of an absorbing mound or berm is also frequentaly considered. The design methods available therefore address the size of rock needed for stability, and the extent of the protection. Three methods are available, for which two are based on regular waves only.

The stability of rock armour on a toe berm in front of a vertical wall, hence subjected to relatively severe reflections, has been studied by Brebner & Donnelly (1962) under regular waves, and by Wu & Jensen (1983) under random waves.

Brebner & Donnelly's results have been used in the Shore Protection Manual (SPM) to give values of a stability number $N_s = H/\Delta D_n$. It is recommended that the value of H selected should lie between $H_{0.01}$ and $H_{0.1}$, depending on the severity of the exposure. They relate values of N_s^3 to a relative depth given by

h_1/h_s, and this is shown in Figure (7-120) of SPM.

Wu & Jensen (1983) combine the results of two series of model tests in a single diagram giving values of stability coefficient, $N_s = H_s/\Delta D_n$, for zero damage, against a relative water depth, h/L_p, for contours of h_i/L_p. Hales & Houston (Hales 1985) considered the stability of a rock blanket extending seaward from a permeable rubble slope on a 1:25 slope foreshore. They tested with regular waves to determine the conditions at which the rock in the scour blanket was just stable. To these conditions they fitted a mean trend given by:

$$\frac{H_b}{\Delta D_n} = 28.5(\frac{b}{L_s})^{2/3} \tag{4.57}$$

where H_b is the breaker wave height
L_s is the wave length in shallow water, given by $T\sqrt{gh_s}$ in this instance
B is the seaward extend of the toe protection.

They also suggest that a more conservative line can be given by:

$$\frac{H_b}{\Delta D_n} = 17.5(\frac{B}{L_s})^{2/3} \tag{4.58}$$

The data on which these equations are based is summarised in Hales (1985).

The maximum scour force occurs where wave downrush on the structure face extends to the toe and/or the wave is breaking near the toe (i.e. shallow water structrures). These conditions may take place when the water depth at the toe is less than twice the height of the maximum expected unbroken wave that can exist in that water depth. The width of the apron for shallow-water structures with a high reflection coefficient, which is generally true for slopes steeper than about 1 on 3, can be planned from the structure slope and the expected scour depth.

The maximum depth of a scour trough due to wave action below the natural bed is about equal to the maximum expected unbroken wave at the site. To protect the stability of the face, the toe soil must be kept in place beneath a suface defined by an extension of the face surface into the bottom to the maximum depth of scour. This can be accomplished by burying the toe, where construction conditions permit, thereby extending the face into an excavated trench the depth of the expected scour. Where an apron must be placed on the existing bottom or only can be partially buried, its width should not be less than twice the wave height. One is addressed to Engineer Manual 1110-2-1614 and Shore Protection Manual (both published by U.S. Corps of Engineers) where th possible configurations are shown.

The wave force will be dissipated on the structure face and a less apron width may be adequate, but at least equal to the wave height (minimum requirement).

Since scour aprons generally are placed on very flat slopes, quarrystone of the size (diameter) equal to 1:2 or even 1:3 of the primary cover layer probably will be the heaviest required unless the apron is exposed above the water surface during wave action. Quarrystone of primary cover layer size may be extended over the toe apron if the stone will be exposed in the troughs of waves, especially breaking waves.

The minimum thickness of cover layer over the toe apron should be two quarry-stones.

Quarrystone is the most favourable material for toe protection because of its flexibility. If geotextile is used as a secondary layer it should not be extended over the whole width of the apron to provide the flexible edges (at least 1 m) against undermining or it should be folded back, and then buried in cover stone and sand to form a Dutch toe.

The size of toe protection against waves can also be roughly estimated by using the common formulae on slope protection and introducing mild slopes (i.e. 1 on 8 to 1 on 10) and local wave height.

Toe protection against currents may require smaller protective stone, but wider aprons. The necessary design data can be estimated from site hydrography and/or model studies.

Special attention must be given to sections of the structure where scour is intensified; i.e. to the head, areas of a section change in alignment, the channel sides of jetties, and the downdrift sides of groyns. Where waves and resonable currents ($u > 1m/s$) occur together it is recommended to increase the cover size at least by a factor of 1.3.

Note that the conservatism of the apron design (width and size of cover units) depends on the accuracy of the methods used to predict the waves and currents action and to predit the maximum depth of scour. For specific projects a detailed study of scour in the natural bottom and near similar existing structures should be conducted at a planned site, and model studies should be considered before embarking on the final design.

In all cases, the experience and sound engineering judgement play an important role in applying these design rules.

4.3 FORCES ON AND STABILITY OF STRUCTURES

4.3.1 General

As already stressed in this chapter, the general layout of design encompasses structural design dealing with the overall stability of structure, the stability of structural constituents, and the strength of structural constituents. The stability of entire structure and its individual constituents is endangered by the near-field failure modes discussed in Chapter Three and examined concisely in Section 4.2, with regard to design guidelines arising from those hazards, wherever appropriate and instructive. In addition, Subsec.4.2.3 also contains a review of the dynamics of units placed on slope revetments. Therefore it overlaps to a certain degree with our treatment of the stability of (other as well as similar) units in Subsec.4.3.4.

Forces, stability and strength (to a lesser degree) are discussed in this section. **Examples** of particular design features for different types of coastal defence measures are presented in Chapter Five.

It seems appropriate to recall that coastal structures must fulfill their defence functions, i.e. provide some desirable topographic features, from the macroscopic (far-field) perspective. On a microscopic (near-field) scale, the structures must be stable and reliable enough to ensure the far-field effectiveness. In this section we analyse the important factor of the behaviour of structures, **forces** exerted by the ambient medium. These forces, together with other factors, such as scour, contribute to **stability** of structures and their units. The forces themselves are dealt with in Subsec.4.3.2 while the stability is discussed in Subsec.4.3.3.

4.3.2 *Forces on coastal structures*

The forces exerted an coastal structures depend primarily on two groups of properties associated with: (1) marine environment (2) structures themselves. The first group incorporates wave climate i.e.various aspects of transformation, including crucial issues of wave breaking, reflection, overtopping and runup. The second group embodies properties of layout and configuration of structures, such as existence of vertical or composite walls, curvilinearities, porosity, spacing of structure's members, etc. A variety of design combinations arise if one considers each type of structures at various distances from shoreline, not to mention secondary effects.

Design formulae and procedures have evolved from some simple concepts dating back to the 19th century. Yet there is no unique formula, or a set thereof, accepted universally throughout the world. Intercalibration of different formulae would be a useful exercise. In its absence, in this report we confine ourselves to a shear presentation of different procedures, varying from well-known SPM proposals, through important Japanese practice to the least known Soviet standards. The review is certainly incomplete, and a lot of room may be filled in if necessary, although it must be remembered that forces are <u>not</u> the primary topic in our project, no matter how important the real sources, components or outcomes of any coastal phenomenon.

Reference is made to our Part Two, where the summary and excerpts from different sources have been presented.

Partenscky (1988) illustrates the history of forces on vertical walls (Fig.4.25 and Fig.4.26), draws attention to the effect of air cushion under breaking waves (Fig.4.27), compares different theories, with his laboratory measurements (his page 252) and eventually proposes a new pressure distribution on vertical walls

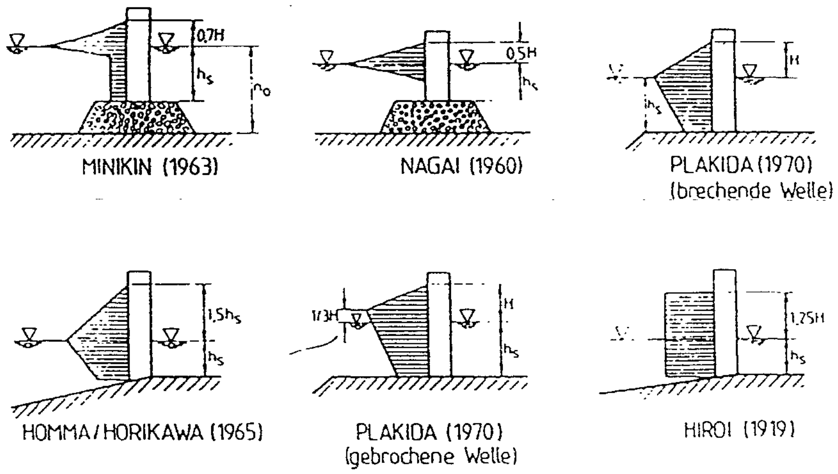


Figure 4.25. Pressure Distribution on Vertical Wall due to Breaking Wave; historical perspective by Partenscky (1988).

(Fig.4.28), with the following set of characteristic pressure ordinates

$$p_c = \frac{\rho x_m}{t_1}\sqrt{gh_s} \approx \frac{\rho H}{t_1}\sqrt{gh_s} \tag{4.61}$$

$$p_0 = \frac{2\pi \rho H^2 c}{t_1 L} \tag{4.62}$$

$$p_s = \frac{2\pi \rho H^2 c}{t_1 L coshkh_s} \tag{4.63}$$

which remains sensitive to evaluation of the wave impact time t_1.

The current Japanese design practice for vertical walls is described extensively by Goda (1989), cf p. 107–123. He also emphasizes the importance of impulsive forces and recommends a procedure for their inclusion (p. 133–138).

The Polish guidelines for maritime structures have assumed the following configuration:

- Vertical Wall Under **Nonbreaking** Waves
  high (non–overtopping) or low (overtopping)
  on rubble mound or without it
- Vertical Wall Under **Breaking** Waves
  divisions as above
- Inclined and Curvilinear Walls
- Single Cylindric Bodies

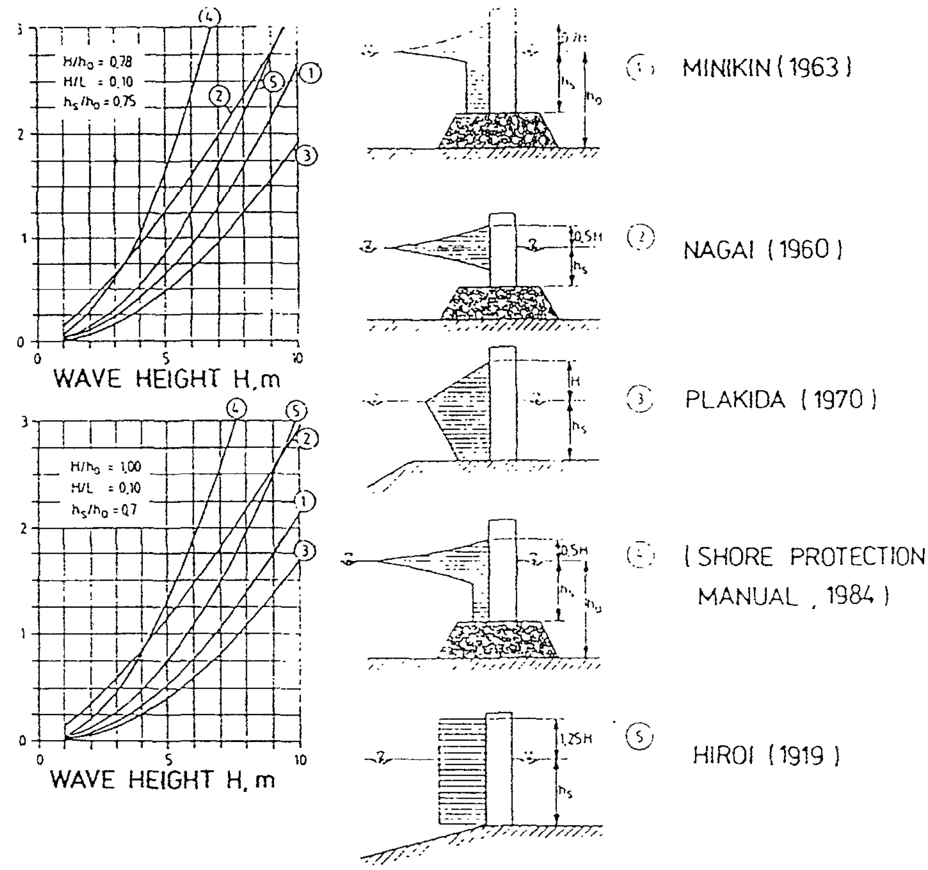


Figure 4.26. Horizontal Force per 1 m (left) and Pressure Distribution (right) in Retrospective; Partenscky (1988).

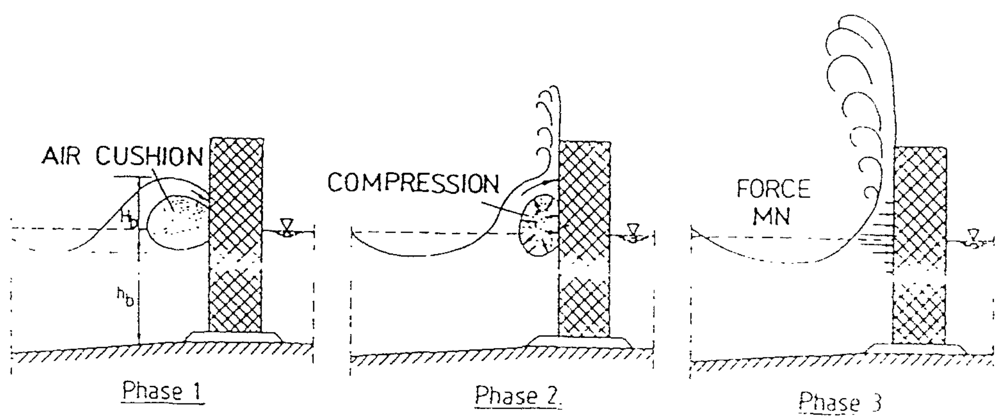


Figure 4.27. Wave Breaking on Vertical Wall; Air Entrapment; Partenscky (1988).

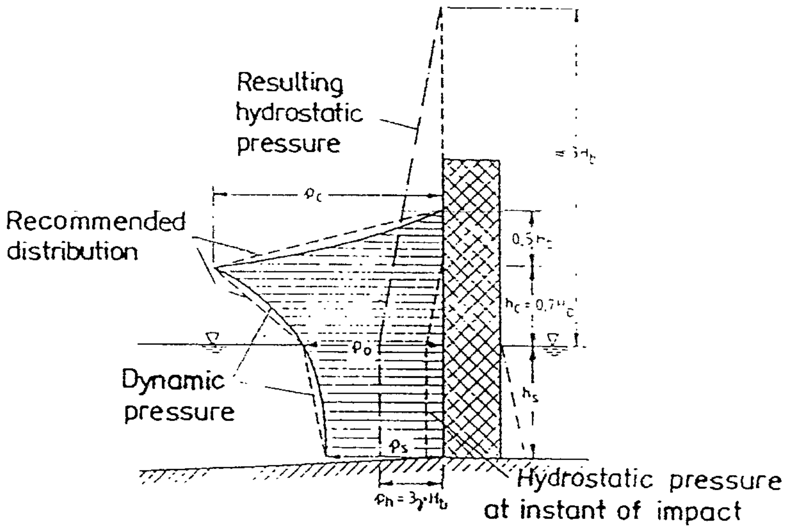


Figure 4.28. Pressure Distribution due to Breaking Wave; Partenscky (1988).

- Spacial Coastal Structures

The formulae for nonbreaking wave are based on the Miche theory in the form proposed by Lundgren, while those for breaking waves refer to the Minikin method.

Comparison of dynamic breaking wave forces on vertical walls predicted by Minikin and Goda methods is provided by Chu (1989). Normalized wave forces computed by both methods are plotted as a function of water depth to deep-water wavelength ratio, seabed slope, and superstructure and foundation dimension ratio. The results exhibit distinctly different characteristics of the two predictive methods. The Minikin method is more conservative than the Goda procedure for large depth to wavelength ratio. With long wave or shalow water depth, the Goda method predicts conservative forces.

The reference to the Shore Protection Mannual is noteworthy.

The Polish quidelines for shore protection structures are based on the Soviet Standard SNiP 2.06.04.-82* (Appendix 4-1).

The latter assumes the following categories:

- Vertical Walls

  Offshore structures; Surf-zone structures; Seawalls

  Nonbreaking or Breaking Waves

- Submerged Breakwater; the same divisions

- Impervious Groynes

- Slope Revetments

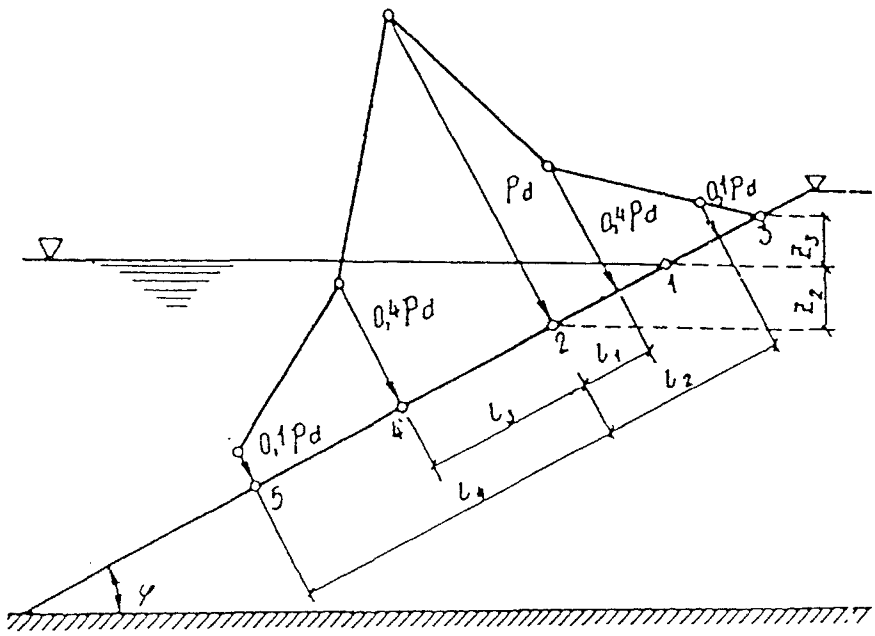


Figure 4.29. Maximum Wave-Induced Pressure on Slope Revetment, in Soviet Standards SNiP 2.06.04.-82*.

An example is provided in Fig.4.29 which illustrates how the loading on slope revetments differs from other approaches. The maximum pressure reads

$$p_m = k_1 k_2 k_3 \gamma H \tag{4.64}$$

the ordinate z_2 under p_m being

$$z_2 = A + \tan^2\beta(1 - \sqrt{2\cot^2\beta + 1})(A + B) \tag{4.65}$$

for

$$A = H(0.47 + 0.023\frac{L}{H})\frac{1 + cot^2\beta}{cot^2\beta} \tag{4.66}$$

$$b = H[0.95 - (0.84cot\beta - 0.25)\frac{H}{L}] \tag{4.67}$$

and z_3 being found from wave runup formulae; with the following lengths $l_1 = 0.0125L_r$, $l_2 = 0.0325L_r$, $l_3 = 0.0265L_r$, $l_4 = 0.0675L_r$ in which L_r is a runup-associated length given by

$$L_r = l \cot\phi(\cot^2\phi - 1)^{-1/4} \tag{4.68}$$

$$k_1 = 0.85 + 4.8\frac{H}{L} + cot\beta(0.028 - 1.15\frac{H}{L}) \tag{4.69}$$

where

k_2, k_3, k_4 = coefficients given in attached tables (App.4-1).

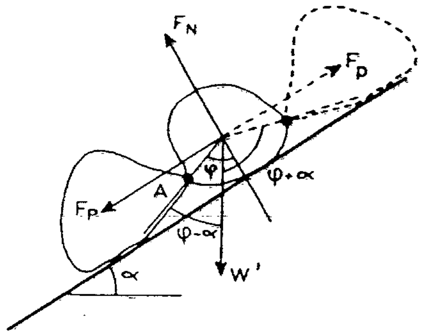


Figure 4.30. Forces on Rock Placed on Slope.

The problem of the forces induced by waves breaking on a sloping surface is attractive scientifically and important practically. It has been dealt with recently within PIANC Working Group 21, concerned about flexible revetments in the marine environment. Here are some highligths.

Regular waves, breaking on a slope excert *cyclic* hydraulic loads. On the basis of physical model tests in wave tanks good knowledge has been acquired of the relevant load phenomena within a wave cycle. For certain specific wave conditions it has been possible to simulate the entire wave induced water motion with numerical models. The practical use of these numerical models is still very limited. Therefore, numerical stability calculations (e.g. STEENZET) completely relay on the wave induced pressures on a slope that are measured in physical model tests. Systematic studies have revealed that small scale tests are sufficient to measure the relevant details of the water motion for slope renvetment stability analysis.

For different types of revetments, different moments or periods from the wave cycle are decisive for the stability of the toplayer.

For rip-rap structures, the decisive combination of normal, F_n, and parallel forces, F_p, is shown in Fig.4.30. It appears that the most dangerous combination of drag and lift forces on the individual elements developes during the rundown of the waves. For placed block revetments Den Boer (1983) analysed the wave- -induced pressures on a slope and distinguished 8 types of resultant loads during one wave cycle that can lead to failure of the toplayer (see Fig.4.31). Later investigations in the large flume of DELFT HYDRAULICS (Burger 1983, 1984) have clearly shown that the most critical failure mode for the toplayer is the uplifting and later ejection of individual blocks are a result of the combination of relatively high water pressures in the filter layer (internal load) and low water pressures on the outside of the slope (external load).

It has become clear that the most critical resultant upward load occurs at the moment of maximum wave rundown at a position just above this level. This resultant load is due to a combination of retarded drainage of the filter layer in combination with the phenomenon that the next wave approaching the slope influences

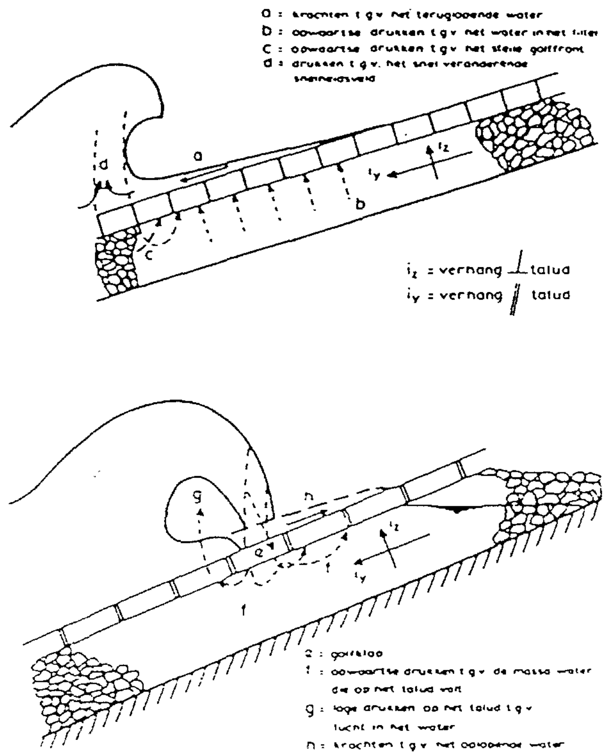


Figure 4.31. Revetment Subject to Breaking Wave.

the internal loads just a moment before the external loads are influenced. It is this perception of loads on a block revetment that forms the basis of the proposed design method (PIANC WG21)

. A lot of effort has been invested in numerical methods to describe and compute the water motion under breaking waves on a slope. The main objective was to compute the water motion for complex slope configurations, with berms and varying slope angles, where empirical methods would fail. The most succesfull attempt has been made by Klopman (1987, 1988) with a modification of the model proposed by Vinje and Brevig (1981). Very succesfull simulations of the water motion have been performed but further improvement of this method towards a versatile, reliable, predictive tool, appeared to be unrealistic. For this reason this development has been stopped in 1988.

At present, the most promising numerical method for the simulation of the complicated water motion under breaking waves on a slope is the "Marker and Cell" method. It will take at least a few years untill the development of this method can give practically valuable results.

A much simple approach towards a computation of the relevant wave loads is to abandon a full description of time and place dependent wave pressures on the slope, but to concentrate only on the moment of critical wave loads. Tests have shown that for placed block revetments the most critical load situation occurs at

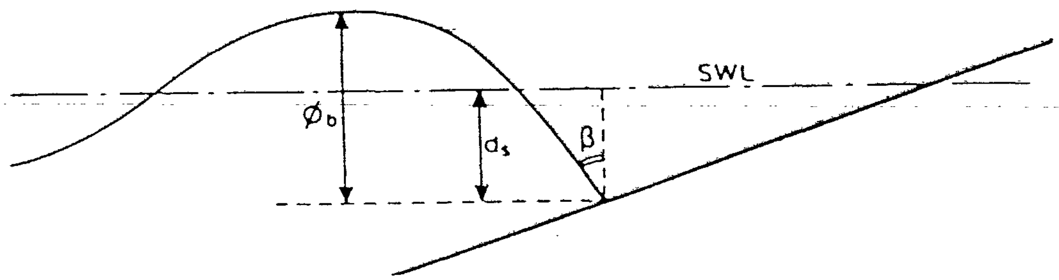


Figure 4.32. Critical Wave Position on Slope.

the moment of maximum wave rundown. Te Duits et al (1988) have indicated that a sufficient description of the critical wave front is given by the parameters ϕ_b, β and ds as shown in Fig.4.32

. A thorough analysis of wave pressure measurements in a small scale model with varying slopes between $1/2 < tan\alpha$ ¡ $1/4$ and varying wave steepness between $0.01 < H/L_o < 0.07$ has led to the following predictive formulae for ϕ_b, β and ds:

$$\frac{\phi_b}{H} = 0.36\frac{\sqrt{\tan\alpha}}{\sqrt{H/L_0}} \quad for \quad \frac{\tan\alpha}{H/L_0} \leq 37 \tag{4.70}$$

$$\frac{\phi_b}{H} = 2.2 \quad for \quad \frac{\tan\alpha}{H/L} > 37 \tag{4.71}$$

$$\tan\beta = \frac{0.17}{\sqrt{H/L_0}} \tag{4.72}$$

$$\frac{d_s}{H} = -0.11(\frac{\tan\alpha}{H/L_0})^{0.8} \quad for \quad \frac{\tan\alpha}{H/L_0} \leq 26 \tag{4.73}$$

$$\frac{d_S}{H} = -1.5 \quad for \quad \frac{\tan\alpha}{H/L_0} > 26 \tag{4.74}$$

in which
H = incoming wave height (m)
L_o = deep water wave length (= $g.T^2/(2\pi)$) (m)
T = wave period (s)
g = gravitational acceleration (m/s^2)
α = slope angle (^{o})
ϕ_b, β, d_s = see Fig.4.32.

With these formulae the critical external wave loads on the slope can be calculated with sufficient accuracy to perform as boundary conditions for the internal water motion and pressures in the sublayers of the revetment structure.

Ice Loading

In addition to the ice effects assessed by the Soviet standard attached in Appendix 4-1 it is worthwhile to examine the western procedure put forth in the aforesaid PIANC WG21 document.

External ice loads on revetments may be divided into three classes:
I. Horizontal loads caused by sgtatic pressure or dynamic impact from moving ice
II. Vertical loads caused e.g. by tidal motion, or by ride up of ice
III. Thermal expansion forces.

Horizontal loads (class I) are caused by surface drag from wind and currents, and occur mainly at structures located on the leeward side of the ice sheet. Their magnitude is limited by the following factors:
- the sum of drag force components acting in the direction of the structure
- the crushing strength of ice (or structure) along the line of contact
- the flexural strength of ice in case of ride-up on a sloping structure
- friction and adfreeze strength between structure and ice

Vertical loads (class II) result when the ice is lifted or pushed above or beneath its normal floating level. Their magnitude is limited by
- the dry weight of ice
- the buoyancy lift of submerged ice
- the flexural stiffness of the ice sheet
- adfreeze or friction between ice and structure

Thermal forces (class III) are mainly expansion type forces, since contraction usually results in cracks and moderate contraction pull. A special type are the forces caused by the formation of ice in cavities and cracks within the structure.

Ice loads may be particularly serious to revetments, by first causing local deformation of the cover layer or rearranging groups of cover elements, followed by dislocation of the cover elements by the ice itself or by late wave action. Internal damage to filter layers may also occur.

Ice loads depend largely on environmental parameters, e.g. air and water temperatures, radiation, water chemistry, waves and wind.
Empirical formulae, such as those given below, should be used with caution, but will usually be sufficient for verification of the problem.
If preliminary investigations indicate that ice loads may be important for the stability of the structure, specialist's assistance is recommended.

Drag forces from wind and currents

Drag forces are calculated from the formula

$$F_d = fA\rho u^2 \tag{4.75}$$

in which
f = skin friction factor (water/ice or air/ice)
A = area of ice sheet
ρ = density of fluid (water or air)
u = velocity of fluid (water or ice)

Where both wind and currents occur, calculation must be made for both, and the resulting forces summed as vectors. Typical values of f are for smooth ice:
Wind drag :$f \approx 0.002$
Water drag :$f \approx 0.005$
For rough ice the f-values may be more than doubled. The velocities should be referred to 1 m below the ice for current and 10 m above the ice for wind.
F_d may give the limiting load for interaction from small ice areas.

Crushing strength of ice

The crushing strength of ice may limit the ice loads, particularly on protruding structures of limited size and having near vertical sea front. The crushing load on a vertical structure reads

$$F_c = \sigma_c h_1 bk \tag{4.76}$$

in which
σ_c = crushing strength
h_1 = ice thickness
b = with of contact zone
k = 0.2 - 1.0, depending on structural geometry etc.

Typical field values for σ_c range from 1 to 3 MPa for ice near $O°$C, rising with decreasing ice temperature.

Ride-up forces on slopes

Fig.4.33 shows the general situation.

For 2-dimensional cases one has

$$F_h = b\sigma_t(\rho_w g h_1^5/E)^{1/4}C_1 + zh_1\rho_1 gbC_2 \tag{4.77}$$

in which
F_h = horizontal force
σ_f = flexural strength of ice (0.5 - 1.5 MPa)
ρ_w = density of water
ρ_1 = density of ice ($915 - 920 kg/m^3$)
E = modulus of elasticity of ice (8,500 - 9,500 MPa)
z = maximum ride-up level or bank height,
h_1 = ice thickness

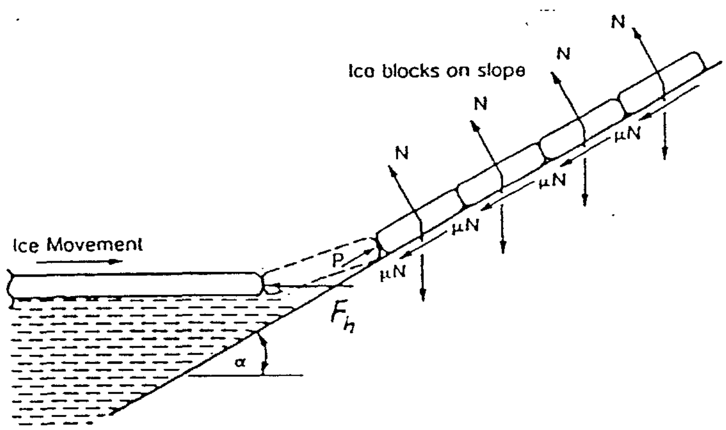


Figure 4.33. Ice on Sloping Surface.

b = width of contact zone
C_1 and C_2 are functions of slope and friction given in Fig.4.34 and Fig.4.35.

Figure 4.36 shows data for thick lake ice with σ_f = 0.7 MPa. [1]

For three-dimensional cases, the situation is more complicated.

Adfreeze Forces

If the ice in contact with a revetment remains stationary long enough for the ice to freeze to the revetment elements, adfreeze forces will result whenever the ice sheet starts to move again. The motion may be parallel to the slope due to dynamic ice thrust, or vertical due to changes in the water level.

For certain types of sloping structures, the force required to break the adfreeze bond can be greater than any other force on a single element, and therefore alone represents the design condition for the structure.

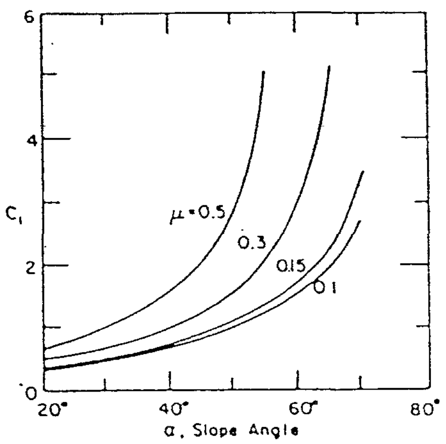
Referring to Fig.4.37, the force F_{ha} required to break the bond, for instance on an element of width b will be

$$F_{ha} = \frac{\sigma_a h_1 b}{\tan\alpha} \tag{4.78}$$

where
h_1 = ice thickness,
α = slope angle
δ_3 = adfreeze bond strength per unit area.

Little data is available on adfreeze strengths. Adhesion strengths published by Michel (1970) are in the range 140 to 1050 kPa for fresh ice. Sackinger and Sackinger (1977) have published adfreeze strengths for sea ice on steel. They found that salinity and temperature affected adfreeze strengths. A maximum value

[1] Figures 4.33...4.36 have been copied from Croasdale, 1980



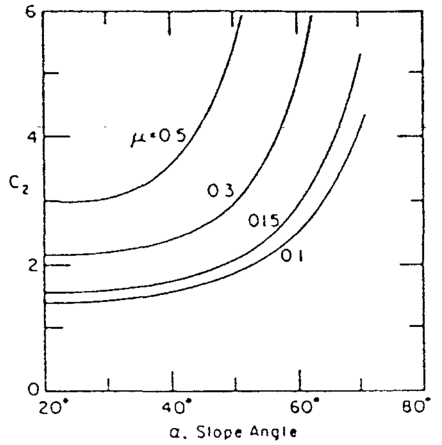
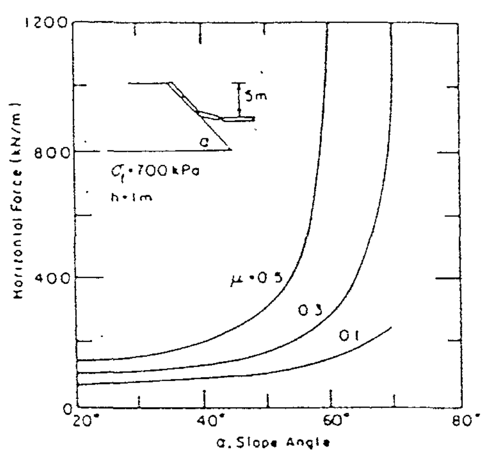


Figure 4.34. C_1 versus Slope Angle and Friction Factor (left).
Figure 4.35. C_2 versus Slope Angle and Friction Factor (right).



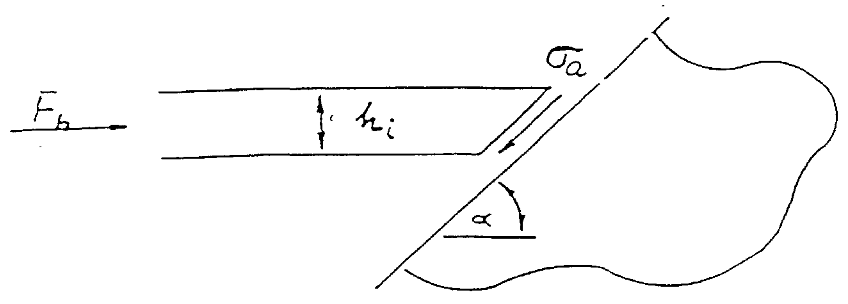


Figure 4.36. Horizontal Ice-Induced Force versus Slope Angle and Friction Factor (left).
Figure 4.37. Adfreeze Bond (right).

of 1590 kPa was measured at a temperature of −23°C for ice with a salinity of 0.4 parts per 1000.

Vertical forces

For a vertical wall, the vertical up/down design load a solid ice cover will be the minimum of the weight/buoyancy load due to the stiffness of the ice, given by

$$F_v = \rho_w g \Delta_z l b \tag{4.79}$$

and the adfreeze bond load being

$$F_{va} = \sigma_a h_1 b \tag{4.80}$$

in which
Δ_z = vertical motion
l = characteristic length of ice sheet = $[\frac{Eh_1^3}{12g\rho_w(1-\nu^2)}]^{1/4}$
ν = Poisson's ratio (0.31 - 0.35).

During strong horizontal pressure against a vertical structure, vertical friction loads may result from water level variations also when no adfreeze bonds are present:

$$F_{vt} = F_d\mu \quad or \quad F - c\mu \quad \text{(lowest value)} \tag{4.81}$$

where
μ = friction factor (0.1 - 0.5)
If $F_{vf} > F_{va}$ then F_{vf} should replace F_{va} in the above analysis.

Vertical ice loads on slopes may have different origin:
The weight of piles up ice above the water level may be significant, but depends on local conditions.
Weight caused by the stiffness of the ice sheet during sinking water level can be found by the formula for F_v ahove.
During rising water level, a lift force of magnitude F_v will appear only if the ice sheet has points for upward attack on the slope.
Adfreeze forces may in some cases lift separate revetment elements out of their position.

Thermal Forces

Thermal expansion of an ice sheet occurs mainly under two situations:
- rapidly rising air temperature
- snow fall on snow-free ice during low temperature.

In the latter case the snow will insulate against heat loss, and the average temperature of the ice will approach the water temperature.
If free to expand, the linear expansion ΔL os given by

$$\Delta L = \Delta T \alpha_1 L \tag{4.82}$$

where
ΔT = average temperature change through the ice
α_1 = linear expansion coefficient $\approx 5.10^{-5}$ per °C
L = linear dimension of the ice sheet.

The linear expansion may cause ride up of similar effects as wind- or current-driven ride up, but it is limited to the distance L each time. By repetition, considerable pile up of ice on the shore may result during a season, however.

If prevented from expanding by solid structures on two sides, the ice will build up pressure against its constraints. The magnitude of expansion thrust varies with many factors, including the local topograph.

Reported values of maximum thrust are within the range 100 - 500 kN/m for 0.5 m thick ice, increasing slowly for larger thickness. The thermal thrust will be quicklwy reduced as soon as the structure yields.
Freezing of water between the elements of a structure causes expansion and may gradually wedge the elements apart, finally leaving the elements free for dislocation by waves etc.

Ice Load Codes and Other References

Various countries or institutions have established codes for predicting ice loads on vertical structures. Similar codes for sloping structures are not known to be available, with the only exception of the Soviet standards given in **Appendix 4-1**.

The knowledge on ice is still developing. It is therefore recommended to consult the most recent literature and handbooks.

4.3.3 *Overall Stability of Structures*

As pointed out throughout this document, an optimum structure should satisfy simultaneously two requirements, i.e.
(1) offer favourable morphological effects and
(2) possess satisfactory stability and durability at possibly low cost.
The anticipated morphological effects include shape and volume of beach forms, shoreline sinusoity, type of shore profile etc. or, most generally, long-term stability of defended coast. This class of problems is very wide, and is treated elsewhere. Attention in this Section is focused on the other aspect, i.e. stablity of coastal structures and their units.

We have already identified the three classes of stability problems. For example, in the case of a mound, the toe structure, the wave screen, the crown and inner slope must also remain stable. Furthermore, the foundation must be able to carry all loads as well as provide protection against bottom material scour to avoid hazards to the stability of the toe structure.

The **overall stability** of a structure can be exemplified by a vertical wall (Fig.4.38).

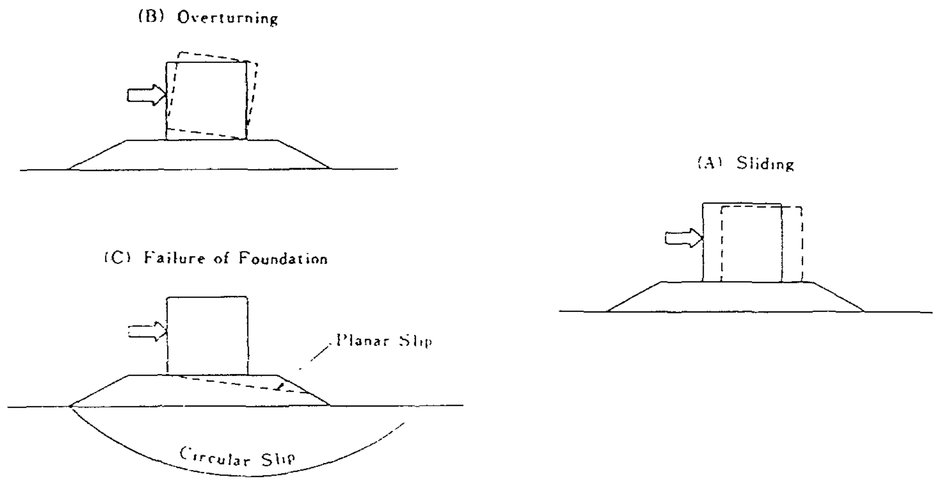


Figure 4.38. Modes of Overall Stability Loss for Vertical Breakwater.

One can see that the major, classical modes of overall stability encompass sliding, overturning and slip of foundation.

All potential modes of the loss of overall stability are checked by reference to *standard methods of mechanics* and the strength of materials. For conservation of overall stability one requires that respective passive forces and moments resist the active counterparts arising due to the impact of the coastal environment. One takes into account all possible effects, e.g. not only the wave-induced pressure but also uplift forces (example in Fig.4.39), friction and shear, etc. Details are beyond the scope of this document.

In the case of rubble mound, **overall stability** is mainly concerned with:

(1) Sliding of the armour layer as a whole or
(2) mass slides penetrating deeper in the mound;
(3) toe failure; and
(4) mass breakdown.

Sliding of the armour layer as a whole is usually a result of a lifting of the armour decreasing or cancelling entirely the friction forces between the armour and the first sublayer. The lifting may be a combination of forces by up- or downrushing water, hydrostatic pressures from the water standing in the mound, which varies highly with the permeability of core and sublayer materials. It is counteracted by gravity including friction forces.

Mass departure of blocks from the armour layer by jumps and/or rollings may take place as a result of such combined forces, which first force a single unit out. Neighbouring units which are resting on (exert pressures on) said unit may then start moving too in a kind of "attempt to close the gap". The result of such movements will, however, in most cases not only be a "healing of the wound",

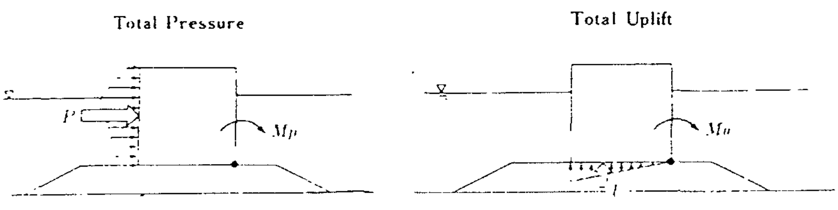


Figure 4.39. Example of Loading in Overall Stability Computations.

but a decrease of ties between blocks making them more vulnerable to extraction by external forces. The departure of one unit may therefore be followed by the departure of a great many other units.

Toe failure causing breakdown of the lower slope expanding upwards, causing a mass failure of the armour. Such incident may start as a failure of the mattress below the toe.

Toe failure is often hard to observe because a severe toe damage usually expands upward in the mound leaving no direct evidence of its occurrence. The reason for the failure is either scour by currents or by deep downrushes or by both. Such scours may be prevented by a proper mattress, which could be built of rock, willow (Holland) or of synthetic material, in all cases loaded down with rock. The magnitude of the mattress depends on the local bottom condition. The softer the bottom the larger and the stronger the mattress has to be. Model experiments may be of guidance on design of a proper mattress.

Mass breakdown by heavy overwashes of the crown of the mound, peeling off layer by layer, and washing most of the material down on the inside of the mound are sometimes the reason for massive failure of mound breakwaters. It has been a habit to save on materials in the crown and in the inside slope. This often proved to be a poor practice. Storm waves are in most cases accompanied by high tides. Together they cause high uprushes and possible overwashes, which could be fatal to the breakwater if crown and inside slope blocks were too small. To improve such situations, still using blocks of more modest size, grouting by asphalt may be used, avoiding the rigidity of cement mortaring. Concrete caps on the top of crown blocks sometimes failed due to uplift pressures. Vent holes may mitigate the problem, but they are seldom kept clean and fail to work.

4.3.4 *Stability of Units*

General

The stability of units can be well illustrated by a rubble mound where it depends heavily on gravity forces. The latter are exerted directly on the unit and by friction forces from the neighboring units. Ability of units to stay in place depends highly upon their geometry. For the same slope blocks may show a great variety in being extracted by hydraulic forces, because their ability to "interknit" or "intertangle" varies greatly. So do the "squeezing forces" by gravity depending upon friction forces in all directions. While intertangling is "a geometrical condition" that does not depend upon slope the squeezing forces are highly slope-dependent. One may, therefore, present the stability in its single components as shown in Fig. 3.29 demonstrating schematically "the stability" as function of slope and the three components of stability: gravity directly, squeezing and intertangling. Gravity is responsible for squeezing by friction forces. Intertangling is a geometrical property. The three components make up "the stability of the unit" disregarding all structural aspects. Various block materials, however, have strongly varying characteristics.

The optimum stability slope, therefore of necessity will come out differently for the three types of armour. It is in this respect interesting to note that Losada gives the following optimum slope angles for these units:

Optimum Stability Slope

Type of Armour Unit rip-rap	$\cot\alpha_{opt}$
quarry stones	5.00
parallelopipedic blocks	4.00 to 6.00
stabits	3.00 to 6.00
tetrapods	2.00 to 2.50
dolos	1.75 to 2.00

The hydraulic stability of revetment units depends not only on the slope angle α but also on the friction and serration (intertangling) of the units. These effects are outlined in Fig.4.40.

Structural unit stability may be considered not only as the ability of the unit to stay in place in the mound but as the structural stability of the unit, which must not repute, crack or break under any load, static or dynamic or both, it will be exposed to.

The forces working for stability of a unit include gravity forces, directly and by friction. These forces may in blocks of special geometrical shape be assisted by joining of blocks in an interknitting or intertangling making the armour a kind of a "mattress". This, however, may be dangerous practice if the single elements (leg or arm) of a block are not strong enough to resist the forces by pressure momentum, shear or combined, which they are exposed to. The numerous failures of breakwaters with armour of multilegged blocks have proven the inadequacy of the structural stability of such blocks, which simply was not considered in the design.

Stability Formulae for Units

Most formulae in contemporary use have been based on the semi-empirical ap-

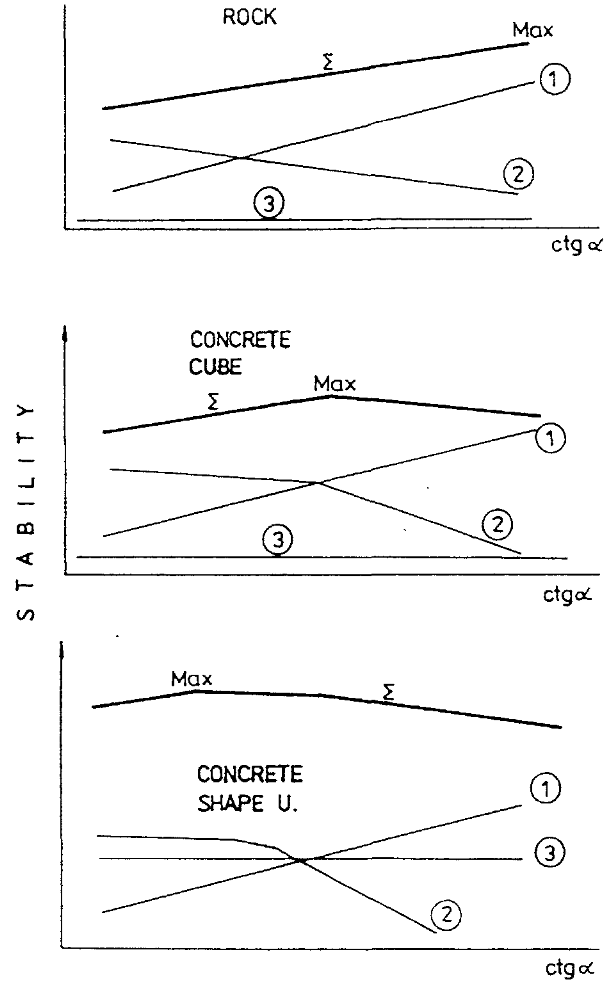


Figure 4.40. Hydraulic Stability of Rubble Mound, Kozakiewicz (1987)
1 = effect of weigth; 2 = friction between units; 3 = intertangling of units; α *= slope angle*

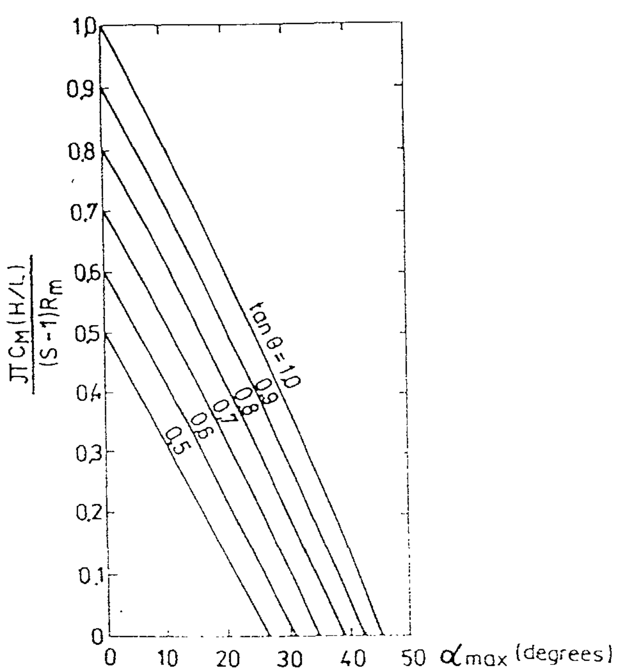


Figure 4.41. Steepest Allowable Slope of Mound, Walton and Weggel (1981).

proach of Irribaren ("the first and probably the best" in Per Bruun's opinion). Some of the versions have been presented for revetments in Subsec.4.2.3.

The weight W of a stable unit reads

$$W = \frac{C_D^3 \rho_s H^3}{8(\rho_s/\rho - 1)^3(\mu \cos\alpha - \sin\alpha)^3} \tag{4.83}$$

in which

c_D = drag coefficient
μ = friction factor, blocks versus mounds
α = slope angle of riprap facing offshore

Subsequent developments include primarily the Hudson formula

$$\frac{H}{(\rho_s/\rho - 1)D_n} = (K_D \cot\alpha)^{1/3} \tag{4.84}$$

in which
H = wave height
$D_n = (W_{50}/\rho_s)^{1/3}$ = nominal rubble diameter
K_D= stability factor
G_{50} = 50% value of the rubble mass distribution curve.

The distinction between the two formulae, Irribaren's and Hudson's, consists in K_D in the Hudson formula, which varies from 2.2 (nonuniform rubble) to 3.2 (roughly equal size of rubble elements). In both approaches the incident wave is regular, at right angle, nonbreaking and nonovertopping.

More recent modifications account for wave steepness, H/L; cf. Walton and Weggel (1981) and Carver and Davidson (1982), wave period, cf. Pilarczyk (1985) and wavelength, cf. Chen, Kao and Tang (1986).

Walton and Weggel (1981) based their considerations on the earlier findings of Ahrens and McCartney, Thomsen, Bruun, Losada and others, and proposed the following formula, which incorporates the effects of inertia, lift and drag, reflected in the parameter of steepness:

$$W = \frac{N \cdot \rho_s \cdot g \cdot H^3 \cdot \left(1 + \frac{C_L}{C_D}\tan\theta\right) sin^3\psi|sin^3\psi|}{\left[(\rho_s/\rho - 1)(\tan\theta\cos\alpha - \sin\alpha) - C_M \cdot \pi \left(\frac{H}{L}\right)\cos\psi\right]^3} \tag{4.85}$$

in which
c_L, c_D, c_M = lift, drag, and added-mass coefficient, respectively
θ =angle of internal friction
ψ = wave phase angle

N_s= stability number.

The stability parametr N is linked with K_D as follows

$$K_D = \frac{\tan\theta\cos\alpha - \sin\alpha}{N_s\cot\alpha} \tag{4.86}$$

For typical values of $\rho_s = 2650kg/m^3$, $\tan\theta = 1$, maximum $h/L = 0.05$ and $C_m = 1.5$ (for a hemisphere on a plane surface) one comes to the maximum stable slope 1 : 1.5. Graphical representation of the stability is given in Fig.4.41.

Carver and Davidson (1982) emphasized the effect of wave breaking. Upon analysis of doloses and rock units they postulated a fuctional (graphical) dependence of N_s on relative wave height $\frac{H}{h}$, wave steepness H/L, Ursell number $\frac{L^2H}{h^3}$, breakwater slope angle α and shape of unit. They have concluded i.a.

a . Effects of H/d, L^2H/d^3, and H/L are more pronounced for dolos armor.

b . In general, minimum stability for each armour type occured for the larger values of H/h $(H/h > 0.90)$, intermediate values of H/L $(0.06 \leq H/L \leq 0.085)$, and larger values of L^2H/h^3.

c . Linear Hudson-type data fits generally give a reasonable approximation of N_s as a function of cot α; however the influences of H/h, H/L and L^2H/h^3 are strong enough to merit their consideration in final selection of armour unit weight.

Pilarczyk (1983, 1985), Meer & Pilarczyk (1987) and Van der Meer (1988) have seriously improved the Hudson formula by including the effect of irregular waves (see 4.2.3). Aside from the controlling quantities identified earlier, such as $\frac{H_s}{(\rho_s-\rho)D_w}$, $\cot\alpha$, $\frac{S}{\sqrt{N_s}}$ (S=damage level, N=number of waves) and porosity P, they also singled out the important factor $\xi_z = \frac{tan\alpha}{\sqrt{2\Pi H/T^2}}$ and formulated stability criteria for a variety of designs and configurations such as slope revetment and breakwater revetment, plunging and surging etc. Pilarczyk (1988) presented a comprehensive summary of his findings, from which excerpts are presented in 4.2.3.

The highest discrepancy between the theory and practice is observed if the Hudson method is applied to long waves, as noted by Chen, Kao and Tang (1986). A unit which is stable under a short wave becomes unstable if the length of the wave increases, for the same wave height. The method proposed by Chen, Kao and Tang links the weight of units with wave length, so that the original method by Irribarren becomes modified. Wave length is found to be more important than wave steepness. Respective nomographs are shown in Fig.4.42.

Kobayashi and Jacobs (1985) provide stability analysis with the effect of drag, lift

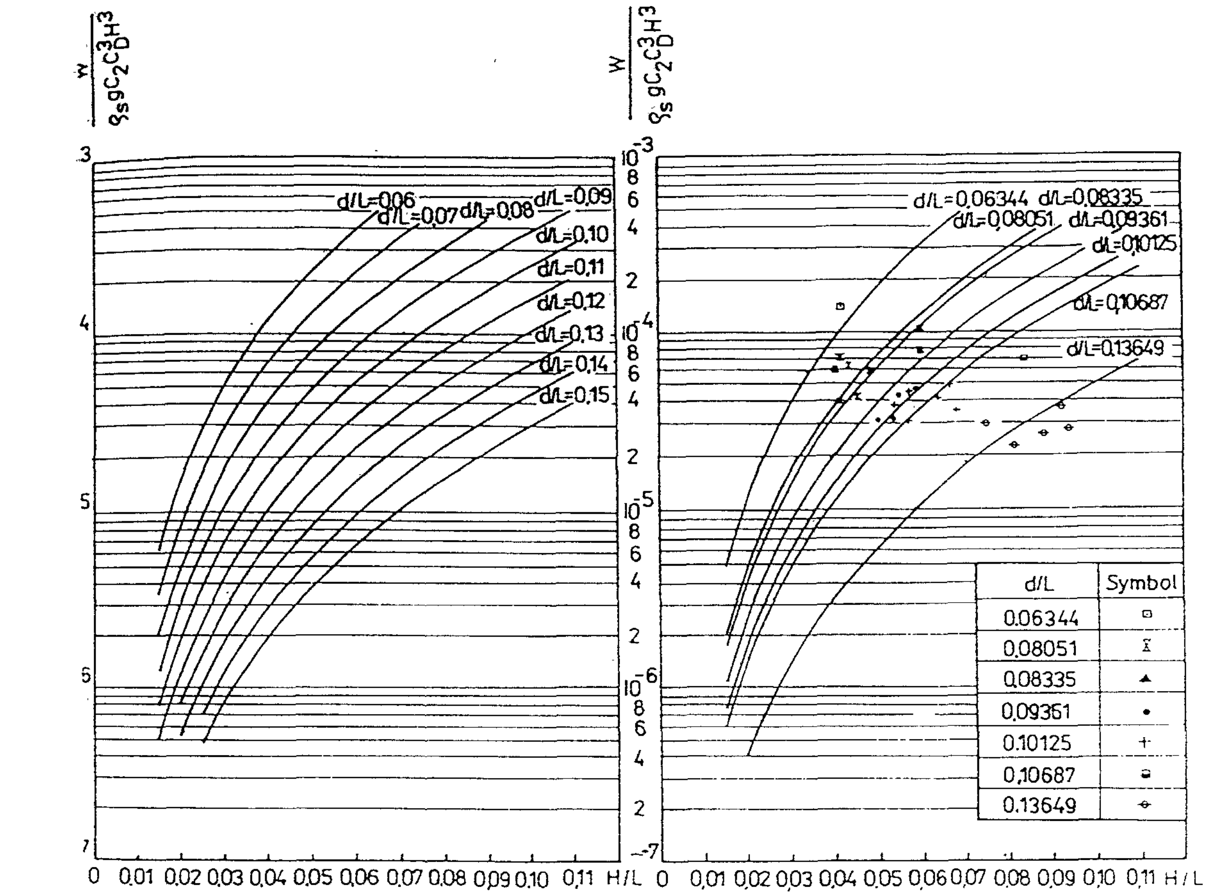


Figure 4.42a. Weight of Units by Chen et al.(1986)
d *stands for depth of water; instead of usual* h.

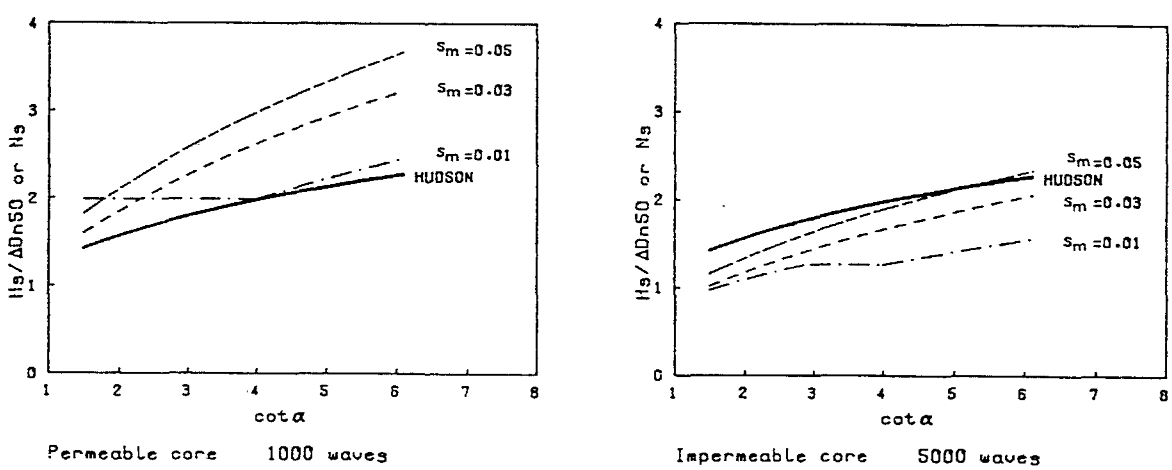


Figure 4.42b. Van der Meer's approach.

and inertia forces which may vary spatially along slope and in time during rush-up and rush-down periods.

Unit stability with regard to sliding down the revetment or in the lift phase is given in the following form:

I: $F_R \geq F_D + F_z + w_s \sin\alpha$
II: $w_s \cos\alpha \geq F_L$
in which
F_R = friction force
F_D, F_I, F_L = drag, inertia and lift forces, respectively.

Kobayashi and Jacobs (1985) provide the following stability criteria:

$$N_S \leq N_R = A\left(\frac{R}{H}\epsilon\right)^{-1}[u(t,x)]^{-2}[\cos\alpha(\tan\theta - \tan\alpha) - \frac{C_M}{S-1}\epsilon \times \times \tan\alpha F(t,x)] \quad (4.87)$$

$$N_S \leq N_L = B\left(\frac{R}{H}\epsilon\right)^{-1}\cos\alpha[u(t,x)]^{-2} \quad (4.88)$$

$$A = \frac{8C_3^{2/3}}{C_2(C_D + C_L \cdot \tan\theta)}; \qquad B = \frac{8C_3^{2/3}}{C_2 \cdot C_L} \quad (4.89)$$

in which
$N_R(x,t)$ and $N_L(x,t)$ - stability numbers associated with the stability against downward sliding rolling and that against upward lifting,
C_2 *and* C_3 - area and volume coefficients of an armour unit,
u(x,t) - horizontal velocity component,
R - wave runup,
ε - nonlineartity factor.

The stability number N_s is related to the stability coefficient in Hudson's formula by
$K_D = N_s^3 \tan\alpha$
Despite many merits, the method proposed by Kobayashi and Jacobs does not account for permeability of structure or wave overtopping. This is considered a limitation, particularly for slope revetments.

Another method of stability analysis has been presented by Breteler et al. (1988). The slope revetment *of place blocks*, or particularly a unit becomes unstable if the force S generated by wave exceeds the resistance force R linked to unit stability,$S \geq R$. The analysis includes distribution of pressures across and along revetment. The quantity R is computed as the difference of pressures above and below revetment unit; it is directed downwards:

$$R = \Delta D \cos\alpha \Gamma_s \quad (4.90)$$

in which
$\Delta = \frac{\rho_a}{\rho} - 1$
ρ_a = specific weight of block
α = slope angle
Γ_s = additional effect of friction and inertia;
$1.1 < \Gamma_s < 1.3$ for $\frac{1}{4} < tan\alpha < \frac{1}{2}$ and normal size of blocks.

The maximum value of the parameter S is determined by the pressure difference (as for R); it is however directed upwards (Wolsink equation):

$$S = \left[\frac{\lambda}{2\tan\alpha \cdot \tan\beta} \cdot (1 - e^{-\tan\alpha \cdot \tan\beta \cdot \phi_b/\lambda}) + 1/2\lambda\right] \cdot (1 - e^{-2\phi_b/\lambda})\Gamma_b \quad (4.91)$$

in which
$\lambda = \sqrt{k \cdot b \cdot Dk'}$
k,k' = permeability of filter and unit of revetment
b = thickness of filter layer
D = thickness of revetment unit
β = angle of wave front with regard to vertical
Φ_b = pressure height at wave front
Γ_D = correction factor accounting for flow in filter layer.
Since the Breteler method is based on simple hydraulic characteristics it is a convenient tool in design stability analysis of revetments.

The basic criteria for slope revetments illustrated in Shore Protection Manual (1984) are given by Galvin (1988). His method of design is based on estimates of lift forces and pressures acting on revetment units and caused by waves of a design storm. The concept is therefore analogous to that of Breteler et al (1988). The balance of the weight, lift and hydrostatic forces provides the following critical thickness of revetment units:

$$t = \frac{\rho}{\rho_a - \rho} \frac{H_b}{\cos\alpha}\left(\sigma + \epsilon\frac{d}{H_b}\right) \quad (4.92)$$

in which
ε = attenuation factor that accounts for leakage and head loss
$0 < \varepsilon < 1$
σ = ratio of wave trough to wave crest elevation under breaking wave
d_s = surcharge of pore water from run-up.

Approximate computations by Galvin (1988) show that the slope angle $\alpha = 26.6°$ (1 : 2) corresponds to the critical thickness of revetment units of the order $t \cong 0.5H_b$. Recent improvements in the design of rubblemound breakwaters are oriented towards optimum selection of slopes and thereby the maximum stability and effectiveness. Ryu and Sawaragi (1986) have proposed a method which accounts for irregular waves, wave breaking and effect of wave period. The following equations yield optimum design and stability:

- uniform slope:

$$W = \left[\frac{\rho_w \cdot g(4.55Q_p + 14.79}{(\rho_r \cdot g)^{1/3}(D + 23.0)} \frac{\tan\alpha}{\tan\theta}\right]^{3/2} \cdot H^3_{1/3} \tag{4.93}$$

- optimum composite slope:

$$W = \left[\frac{\rho_w \cdot g(5.55Q_p + 17.86)}{(\rho_r \cdot g)^{1/3}(D + 35.8)} \frac{\tan'\alpha}{\tan\theta}\right]^{3/2} \cdot H^3_{1/3} \tag{4.94}$$

in which
ρ_r = density of rubble
$\tan'\alpha$ = inclination of imaginary composite slope
Q_p = spectrum peakedness parameter
D = allowable damage, %. The optimum design with minimum wave reflection, run-up and maximum stability is estimated for relative depth $h_b/H \cong 0.5$ and berm width $l_b/L \cong 0.25$.

The optimum design of composite slope revetment provides reduction in wave reflection by 50% (or even more), in wave run-up by 10%, accompanied by increase in stability by 50%, for the same volume of revetment. Ryu and Sawaragi draw further attention to adequate identification of statistical wave parameters for irregular waves. They rightly assume that this is one of the most important factor in design of revetments. It should be emphasized that wave height is in the third power, so that any inaccuracy becomes highly critical.

The Soviet practice reflected in the standard SNIP P-57-75 (and later SNiP 2.06.04-82*) provides quadratic relationship of the unit weight and wave height: one assumes

$$W = \frac{m\phi \cdot \gamma_b \cdot H^2 \cdot L}{\Delta^3\sqrt{1 + m\alpha^3}} \tag{4.95}$$

for elements on the depth $z < 0.7H$ and

$$W_z = \omega \exp\left(\frac{7.5z^2}{H \cdot L}\right) \tag{4.96}$$

for deeper elements $z > 0.7H$ in which
m_ϕ = experimental factor depending on type of blocks and their placement; it can vary from 0.0034 to 0.025; and
$m_\alpha = \cot\alpha$.

Kozakiewicz (1987) compared three methods of tetrapod design (Hudson, Losada et al. and Soviet standards). He has found out that the Hudson method yields the heaviest blocks. The values obtained by different methods are highly scattered. If economy and safety require precise assessment of block stability, one must resort to hydraulic experiments. Generators of irregular waves are very helpful in the simulation of real waves.

The expected occurrence of currents along a proposed coastal structure should affect the design of that structure mostly in regard to selecting the size of cover layer stone for the toe apron.

The minimum weight of stable quarrystone necessary to resist a current can be determined from, Eckert (1983):

$$W = \frac{V^6 w_r w_\omega^3}{15.23 \cdot 10^5 (w_r - w_\omega)^3 (\cos\theta - \sin\theta)^3} \tag{4.97}$$

where:

W = minimum weight of quarrystone, lbs.

V = velocity acting directly on the stone, feet per second

w_r = unit of weight of rock, pounds or cubic feet

w_ω = unit weight of water, pounds per cubic feet

θ = angle of the toe apron slope with the horizontal in the direction of flow.

The weight of quarry stone (W) is the minimum weight that will be stable. A range of quarry stone weights may be specified with this weight as the lower limit. This quarrystone layer should have a minimum thickness of twice the maximum stone diameter.

Rubble-mound structures require toe aprons for two purposes. First, to prevent scouring of bottom sediments from under the primary armour causing a slip, opening of the armour, and wave attack on the under layer. This can be protected by a relatively narrow apron. The second function of a toe apron is to prevent formation of a scour hole adjacent to the structure which can effectively decrease the slope stability of the entire structure.

Design criteria for toe apron design must address both the width of protection necessary before the wall and the size of armour units needed in the toe spron to resist the expected wave or current forces.

Vertical-faced walls gravity retaining walls require **toe apron** to prevent undermining at its toe. A rule of thumb is to extend aprons a distance beyond the wall face equal to their height to prevent settlement due to a bearing capacity failure or tipping of the wall due to loss of toe support.

It may be said *in passing* that the major problem encountered in designing **toe apron** for current induced scour is usually the lack of good current velocity data. It is difficult to measure the current velocity acting on the stone, particularly during storm conditions when scour is most likely. When currents occur in conjuction with waves, the scour effects are partially additive. A general design guideline in such cases is to increase the cover layer quarrystone weights found for wave scour by 1.5.

The first step in design of any toe apron to resist wave-induced scour is the selection of a design wave. This design wave should be selected to recognize both the criticality of the structure, i.e., the impact of its failure, the reliability of the wave data, and the potential difficulty or relative economy of repair versus initial overdesign and construction. The guidance of the SPM (1977) for highly critical structures is to use the maximum depth limited wave, where wave data indicates that such waves can be reasonably expected, or if the site is not depth limited, use the average of the highest one percent of all waves at the site. For less critical structures a lesser wave, i.e., the average of the highest ten percent of all waves, may be used.

The most widely used method of designing in the sixties of the size of toe apron units for vertical walls was that suggested by Brebner and Donnelly in their study of rubble-mound foundations for vertical breakwaters. This method has been included in the SPM (1977) and is the most rational approach to toe apron design short of hydraulic modeling. The database of the Brebner and Donnelly work is confined to laboratory studies of random placed rubble of a uniform size and subangular shape. The study developed a stability number N_s which can be correlated with values of the depth ratio d_l/d_s and used as a logical design parameter for computing the mean weight of toe apron cover stones (W) by the equation given in Fig.4.43.

According to Eckert (1983) one has:

$$W = \frac{w_r \cdot H^3}{N_s^3(S_r - 1)^3} \tag{4.98}$$

In this equation w_r is the unit weight of quarrystone, S_r the specific gravity of the quarrystone, H the design wave height. Using the mean weight (W) found by this method, the cover layer stones should be specified as 0.5 to 1.5 W.

Brebner and Donnelly found that the top width of the toe protection (H) does not substantially affected the hydraulic stability of the cover layer. The SPM suggests taking B as $4d_s$ unless the geotechnical criteria require a side toe apron. The stability of armour units for rubble mound formations of composite breakwaters has been investigated under the action of irregular waves, by Tanimoto at al. (1982). The results of this study provide the design informations of armour units for rubble mound foundations of composite breakwaters. The stability number of the armour units are greatly influenced by the configuration of rubble mound foundation and wave conditions. The stability numbers for quarry stones and concrete blocks were formulated by two parameters of $h'/H_{1/3}$ and K on the basis of irregular wave tests. The stability number is proposed as:

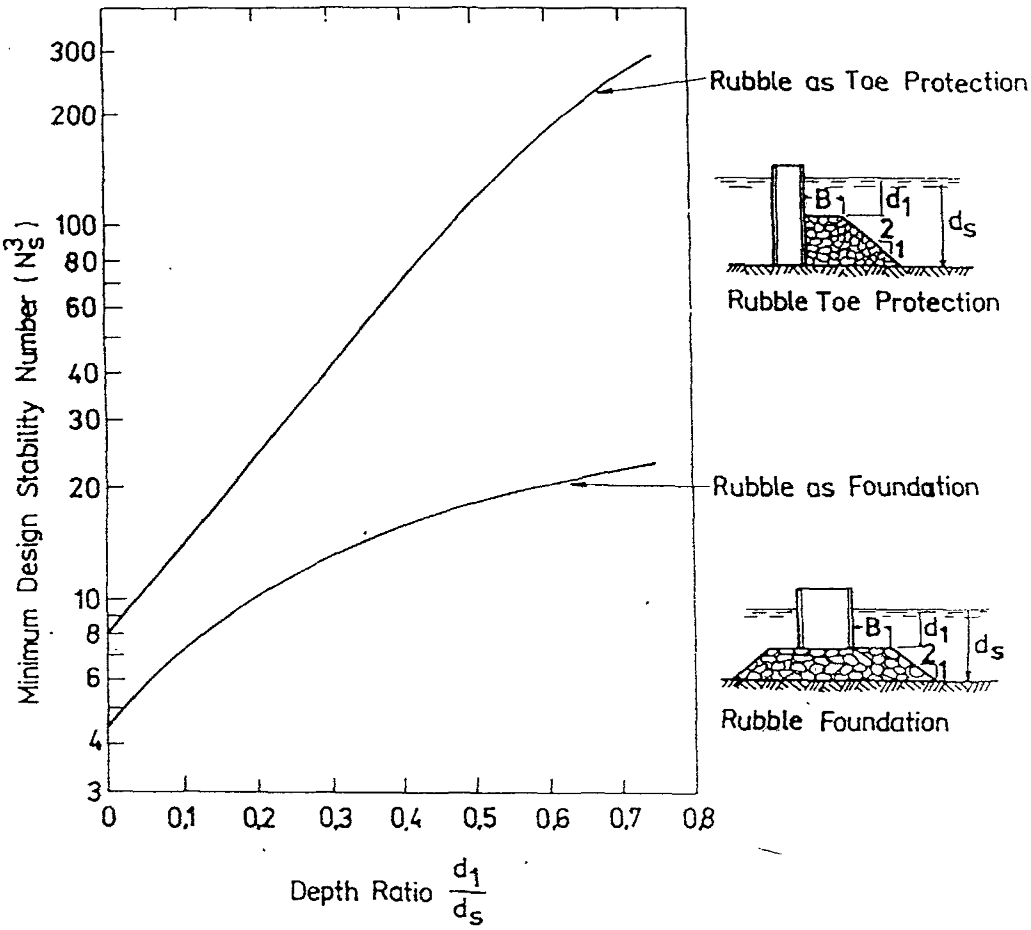


Figure 4.43. Stability Number by Eckert (1983).

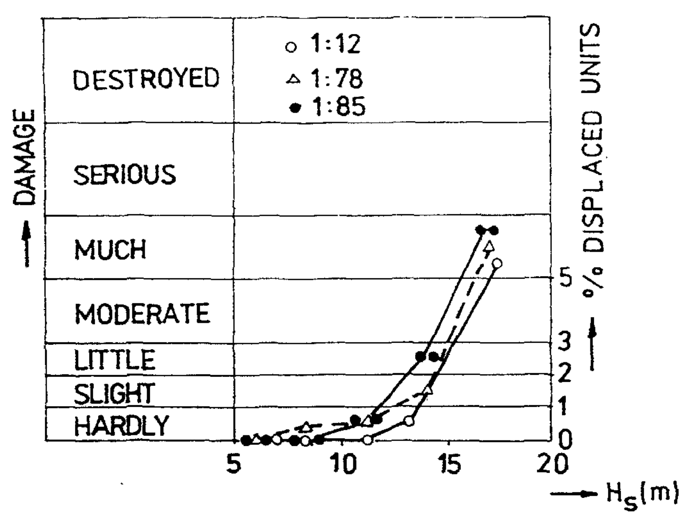


Figure 4.44. Effect of Significant Wave Height on Damage of Slope Armour Units, Mol et al.(1983).

$$N_s = \max\left\{1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left[-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\cdot\frac{h'}{H_{1/3}}\right]\right\} \tag{4.99}$$

The design mass of quarry stones reads

$$W = \frac{\rho_r}{N_s^3(S_r-1)^3}H_{1/3}^3 \tag{4.100}$$

where:

h'- depth at the crest of rubble mound foundation exluding the armour layer

κ = a parameter for the combined effects of the relative water depth and the relative berm with of the rubble mound foundation to the wavelength

$H_{1/3}$ - design significant wave height.

Generally, the damage of quarrystones placed in two armour layers progresses rather gradually. The application of the proposed method to the test results demonstrates that the damage of quarry stones is at most 3.5% at the design condition. On the other hand, the damage of the concrete blocks placed regularly in single layer progresses drastically in the case of high rubble mound foundations. Therefore, it is not recommended to apply the concrete blocks in single layer to high rubble mound foundations.

Broderick L. (1983) has found from his laboratory tests with irregular waves that when converting monochromntic wave riprap design criteria for spectrum use, the 5% wave height is more appropriately used in design calculations than significant height. He has also showed dependance of irregular waves on wave period.

The change in the riprap surface between the damage survey and the reference survey typically consists of an erosion zone and an accretion zone. The volume per unit length of the erosion zone is used to quantify the extent of damage D (D is made dimensionless by using the median stone size W_{50} to characterize the size of the riprap). Dimensionless damage D' is given by

$$D' = \frac{D}{(W_{50}/W_r)^{2/3}} \tag{4.101}$$

where
W_r is the unit weight of riprap, $2,700kg/m^3$.
The quantity D' is related to stability number N_s as follows

$$D' = aN_s^b \tag{4.102}$$

$$N_s = \frac{H}{\left(\frac{W_{50}}{W_r}\right)^{1/3}\left(\frac{W_r}{W_w}-1\right)} \tag{4.103}$$

in which

H = damage-causing wave height

W_w = unit weight of water

a,b = dimensionless regression coefficients.

The laboratory tests by Baird, Scott and Turcke (1986) have pointed out to the importance of temporal variation of forces and moments about revetment units. The durability of unit depends on its position on the revetment. One must account for this position (below or above water level), and include the various relationships between forces and moments. Some units must be overdesigned while other will be underdesigned. Adequate revetment design requires an analysis of hydraulic stability of the entire structure and units. Each layer of breakwater must be analysed separately.

Scale effects must not be forgotten in the analysis of hydraulic tests. These effects are discussed by Shimada, Fujimoto et al. (1986). The scale effects are found negligible for Reynolds numbers above $4 \cdot 10^5$.

However, no serious scale effects with respect to the stability of the Sines breakwater (Portugal) have been identified by Mol, Linteringen et al. (1983) who looked for possible scale effects in the Delta flume with irregular waves up to 3 m. In Fig. 4.44 the results obtained for different scales are compared. The scale effects in stability tests must obviously include viscocity, surface tension and mechanical effects. Bruun and Kjelstrup (1983) ascertain that scaling of tests on mound structures is difficult to evaluate, as single elements and their combinations behave differently. They propose a procedure consisting of four steps:

a large scale tests, preferably 1 : 5 or 1 : 10. Final tests should be run in large scale.

b proper scaling of core and sublayer materials to produce a more correct water table movement.

c three-dimensional tests of wave bvreaking and flow at the surface of the mound.

d structural tests and analysis of resistance against shocks and fatigues.

Basic information from the field is of course very important.

A variety of armour units for mounds and revetments have been devised and patented, cf. e.g. Fig.4.45 and Fig.4.46.

Dolos is believed to be one of the most economic blocks and therefore a lot of attention has been paid to the analysis of its effectivenes. Dolos is one of the designs offering a high hydraulic stability for armour units. However, as any type of unit, it must be sufficiently strong and durable. Zwamborn and Scholtz (1965) have concluded from their tests that doloses can resist high waves with minimum

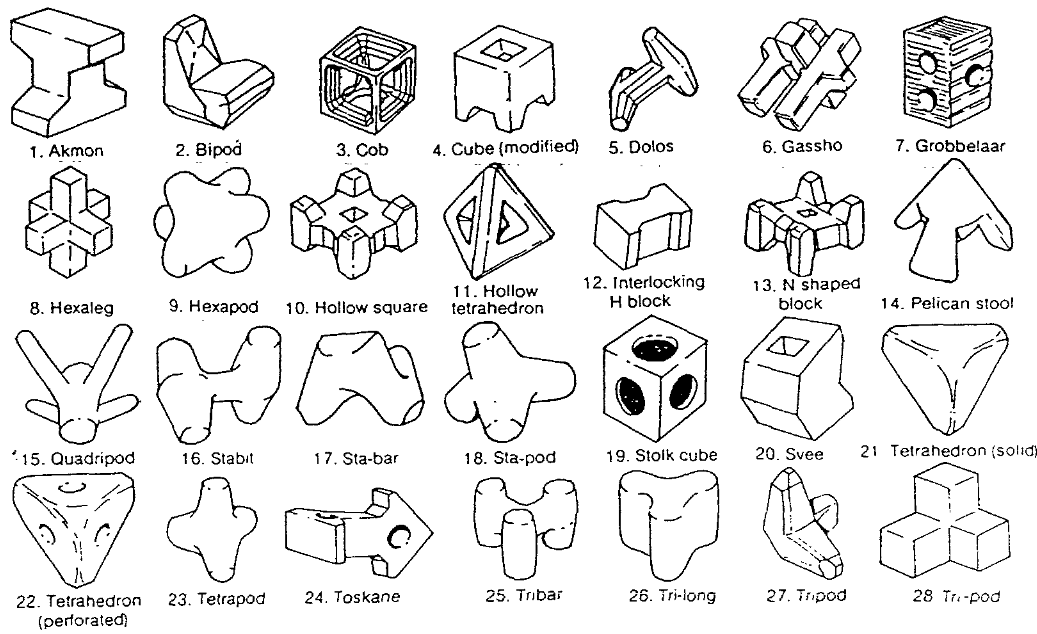


Figure 4.45. Armour Units, Thorn and Roberts (1981).

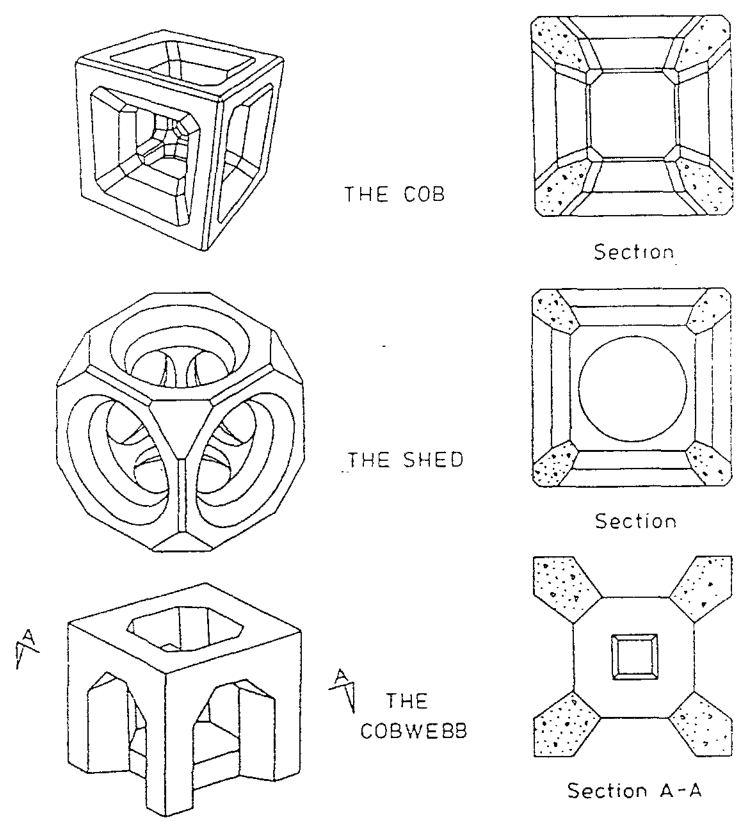


Figure 4.46. Hollow Armour Blocks.

damage (e.g. 30-ton units suffered 1...2% damage under 10-m waves while 6... 7 - m waves were resisted by 20-ton blocks throughout the year). However, adequate rules of proper design must be complied with.

Glodowski et al (1982) ascertain that the stability and durability of dolos mounds depends criticaly on wave breaking and overtopping. Their conlusions were based on the Liviere-Renard breakwater in St. Lawrence Bay (Canada). Built in 1972, the breakwater was unexpectedly damaged in 1978 and 1980. The inspection has exposed 10...50 - percent damage on the front side (and 30...90 - percent on the primary spot of disaster; the doloses weighed 12.7 tonnes). The design was based on Hudson's formula with $k_D = 15 \ldots 22$ for the front and major sections, respectively. This yielded 12.7-ton units for W = 5.3 m and 4.5 -ton ones for 5.1-m waves. The disastrous storm waves were lower but yet the damage occurred. Overtopping was found responsible, and doloses were claimed paraticularly vulnerable to this type of failure. Glodowski et al (1982) determined in their scale tests that 20-ton doloses were necessary to withstand waves and remain stable during overtopping. Hence heavy doloses are uneconomic and overdimensioned with regard to wave impact (if the stability requirement follows from overtopping). One may recommend doloses at the foot of mound, but still their bahaviour must be frequently inspected. - Incidentally, the practice of routine inspections of coastal structures to **repair minor damage prior to major disaster** is paving its way in many engineering communities, not only in Japan where it has been well established for all breakwaters since the early post-war era.

Pope and Clark (1983) infer from their two-year monitoring in Lake Erie that the dolos armor has been dynamic with the greatest movement near the waters edge and there seems to be no decrease in movement with time. A general settlement has occurred, both inward and downward, with no apparent loss of structural integrity. Individual doloses have broken especially in the vicinity of the water's edge but the breakage appears to be random except for a concentration of breakage at the head section which was caused by the April 1982 storm. The November 1982 aerial photograph of the breakwater head contrasts the loose packing of newly placed doloses with the tighter packing after 2 years of adjustment.

Construction material of doloses obviously affects their durability. Bradbury et al (1986) conclude that cracking, breakage and abrasion are concentrated in the area of maximum dissipation of wave energy while crushing and decay occur at high temperature and salinity. Durability may be improved if the conditions specified by Kohno et al (1986) and Seiji et al (1987) are observed.

The scatter of mound units is pronounced at places where local depth at breakwaters exceeds mean depth or waves approaching at angle generate strong currents (Kohno et al (1986) for Fuji coast). Seiji et al (1987) provide the following conclusions.

1. The scattering of concrete blocks was found in 13.7% of all surveys.

2. The ratio of the number of the breakwaters that suffered scattering to the total number of the breakwaters on each coast takes the maximum value, when the bottom slope, the depth at the breakwater, and the offshore distance of the breakwater are equal to 0.02-0.03, 3-5 m and 80-100 m respectively.

3. A high scattering rate appears when the actual weight of concrete blocks is smaller than 1.1 times the weight calculated from the Hudson formula. The scattering rate reduces to 3.1% when concrete blocks heavier than 1.5 times the calculated weight are used.

4. The scattering rates of concrete blocks were 19.6% and 11.6% for the cases of pellmel and uniform placings. The scattering rate does not significantly depend on the kind of block placing, if a foundation is constructed. Otherwise, the scattering rate in the case pellmell placing is more than twice as high as that in the case of the uniform placing. The extent of scattering is particularly influenced by the kind of block placing in the case of detached breakwaters without foundation.

The conclusion of Seiji et al (1987) saying that the weight of armour unit obtained by Hudson's formula must be multiplied by 1.5 in order to maintain adequate stability agrees with the Canadian findings of Glodowski et al (1983).

An example of damaged mound is provided by the Akranes breakwater (Iceland) made of natural rock, and not of dolos, cf.Chapter Three.

Partenscky (1988) emphasizes the fact that breakwater safety depends primarily on the stability of its upper layer of rubble mound, this in turn being a function of the stability and durability of armour units. Basing on large flume tests in Hannover, Partenscky compares the suitability of k_D values proposed by CERC in SPM and recommends their reduction by up to 50% in some cases (if less than 2% of the units are to undergo damage). It is interesting to compare CERC values against those proposed by Partenscky (Table 4.7).

It can be seen that Partenscky also introduces a **new concept of permissible damage index I**. The latter is defined as
$I = \sum_{j=1}^{6} U_J W_j$
where
W_j = importance factor for j-th class
U_j = percentage of damage in j-th class.

The importance factor W_j depends on the rotation (counted in degrees from vertical) and horizontal dislocation experienced by an armour unit (Table 4.8).

The index I is recommended as 150 for dolos, 240 for tetrapods and 440 for cubes. The weight of armour units must be so selected as to correspond to the stability factor K_D and not surpass the cumulative damage inherent in I (e.g. for doloses one may have $1.20 + 4 \times 10 + 9.2 + 16.1 + 25.1 + 36.0 = 119 < 150$).

Table 4.7.
K_D by CERC for Mound Breakwaters

Armour Unit	Central Breaking	Section Non-break	Head Breaking	Section Non-break
Round rock	1.2	2.4	1.1	1.9
Sharp rock	2.0	4.0	1.6	2.8
Tetrapod	1.0	8.0	4.5	5.5
Tribar	9.0	10.0	7.8	8.5
Dolos	15.8(8.0)	31.8 (16.0)	8.0	16.0
Modif. cube	6.5	7.5	-	5.0
Hexapod	8.0	9.5	5.0	7.0

1) Quadripods as tetrapods

2) Two layers

3)Orderly arrangement

4) Up to 5%of units to be damaged (2% in parantheses)

K_D by Partenscky(1987)

Armour Unit	Averaged Scatter	K_D Mean	Recomm. K_D	Perm.Dam. Index I	K_D by CERC
Tetrapod	5.9...9.1	7.5	7.5	240	7.0
Dolos	6.7...16.5	11.6	10.0	150	15.8
Cube	9.5 ...15.5	12.5	10.0	440	6.0

Table 4.8. Partenscky's factor W.

Class	Rotation	Dislocation	Imp. factor W
-	o	-	-
1	$<< 5$	$<<$	1
2	5 - 15	$<< -H/6$	4
3	15-30	H/6 - H/3	9
4	30-45	H/3 - H/2	16
5	45-90	H/2 - $< H$	25
6	> 90	$\geq H$	36

H = height of armour unit.

The stablity and durability of structures and their armour units depend critically on the underlying filters. Führböter (1988) tested geotextiles and granular filters beneath concrete rock revetments under identical wave loading. Geotextiles were found better as the stability parameter of the revetment was 4.5 (versus 3 for granular filters). The difference is attributed to uplift forces generated by space-varying wave pressures (accross revetment). These forces are found most intensive about half wave height below still water level.

Fairly good stability properties of rubblemound foundations are claimed by Watanabe and Horikawa (1983).

The probabilistic aspect of failure incorporated in Partenscky's index I is treated otherwise by Mol et al. (1983). They ascertain it is not yet feasible to establish the probability of failure by means of a full probabilistic approach. As an illustration a more limited approach is used by assuming only the wave height and peak period to be stochastic parameters. Further it was assumed that:

- groupiness, tide and wave direction do not influence stability
- the chance of occurrence of a design storm with $T_p = 16$ s(sea only) is assumed to be equal to that of a combinantion of sea and swell with $T_p = 22\ s$.
- due to the ageing effect the damage curves from the model shift to lower wave height with 10% in the first 25 years and 5% in the following periods of 25 years.

The probability of occurrence of a certain damage is calculated by:

P (damage) $= T_p = \sum P(T_p)P(H_s; damage)$ for 16 and 22 seconds.

The probability of non-occurrence of a certain damage in N years is calculated by:

P (no damage in N years) $= (1 - P(damage))^N$

In the following the probability of non-occurrence of damage with the classification "little" is presented for the Outer Part of Sines breakwater for various combinations of assumptions about ageing (ageing and non-ageing and the peak period (both $T_p = 16$ and 22 s uniformly distributed and only $T_p = 22$ s which is a maximization). Four succeeding periods of 25 years were selected,and the computation was also made for the full period of 100 years.

Possible seastate T_p	Ageing/ non-ageing	Probability of non-damage return First 25yrs	2nd 25 yrs	3d 25 yrs	4th 25 yrs	full 100 yrs	per.dam. little
16; 22 s	ageing	0.93	0.78	0.64	0.47	0.22	67
16; 22 s	non-ageing	0.93	0.93	0.93	0.93	0.75	350
22 s	ageing	0.84	0.65	0.47	0.28	0.07	38
22 s	non-ageing	0.84	0.84	0.84	0.84	0.50	145

Mol et al.(1983) concluded that both the definition of the ageing process, and the knowledge about the occurrence of wave spectra influences the calculated return period of a certain damage category considerably.

Yet another proposal is offered by Meer and Pilarczyk (1987). In their formulation the dimensionless damage level, S, can be described by:

$S = A/D_{n50}^2$

where A is the eroded cross-sectional area of the profile, see Fig.4.47 a.

A physical description of S is the number of cubic stones with a side of d_{n50}, eroded around the water level within a width of one D_{n50}. The 'no damage' criterion of Hudson and Ahrens is taken generally to be when S is between 1 and 3 stones eroded. The advantage of this definition of damage is that it is independent of the length of the slope.

A rather low probability of damage in one year is noticeable in Fig.4.47 b.

The probability of exceedance for an X year period can be obtained as follows

$$P[Z <; Xyears] = 1 - (1 - P[Z < 0, 1year])^X \quad (4.104)$$

These results are shown in Fig.4.47 c. Curves are drawn for three life times. From this figure it follows that the damage level S = 2, which means start of damage, will certainly occur in a lifetime of 50 years. Tolerable damage in the same lifetime (S= 5 - 8) will occur with a probability of 0.2 - 0.5. the probability that failure (filter layer visible) will occur in a lifetime of 50 years is less than 0.1.

Such probability curves can be used to make a cost optimization for the breakwater during its lifetime.

STABILITY OF LOW-CRESTED BREAKWATERS AND SUBMERGED BREAKWATERS

Meer and Pilarczyk (1988) propose the following categories of subaeral and submerged breakwaters:

- Dynamically stable reef breakwaters. A reef breakwater is a low-crested homogeneous pile of stones without a filter layer or core and is allowed to be reshaped by wave atteck. The equilibrium crest height, with corresponding transmission, is the main design parameter

- Statically stable sub-areal breakwaters. These structures are close to non-overtrpped structures, but are more stable due to the fact that a (large) part of the wave energy can pass over the breakwater

- Statically stable submerged breakwaters. All waves overtop these structures and the stability increases remarkably if the crest height decreases.

Stability analysis for reef breakwaters consists more frequently in determination of crest variability under waves. Van der Meer (1988) provides a few dimensionless parameters, among which the following three are most important:

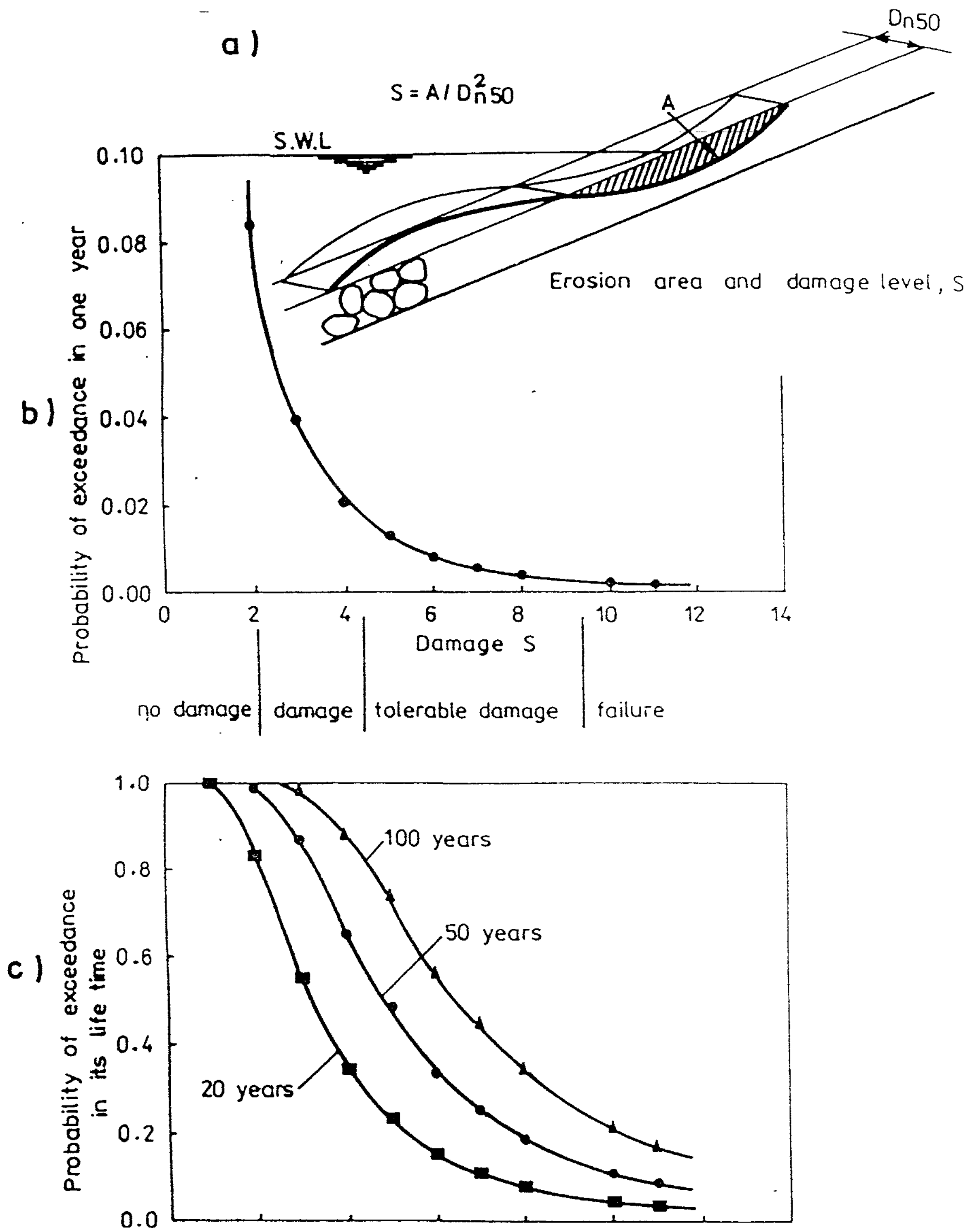


Fig.4.47. Damage Level for Rubble, Meer & Pilarczyk (1987)

- breakwater crest height reduction $\frac{h_c}{h'_c}$
- relative crest height $\frac{h_c}{d_s}$
- effective slope C' and C.

The parameters h_c, h'_c refer to the structure height, damaged and designed, respectively;
d_s = depth of water; and
$C' = \frac{A_c}{h'^2_c}$ and $C = \frac{A_t}{h^2_c}$
A_t = cross-section of structure.

Van der Meer provides the following stability equation for reef breakwaters

$$C = \frac{A_t}{h_c^2} = exp(aN_s) \tag{4.105}$$

in which
N_s = stability parameter
$N_s = \frac{H_{om}}{\Delta D_{n50}} S_p^{-1/3}$
S_p = local wave steepness = $\frac{h_{om}}{L_p}$
H_{om} = significant wave height
L_p = Airy wave length
D_{n50} = nominal diameter of armour units
$a = -0.028 + 0.045C' + 0.34h'c/ds - 6 \cdot 10^{-2}(A_t/D_{n50}^2)$.

The stability of subaeral (overtopped) breakwater is related to the stability of a non-overtopped structure. Stability formulae derived by Van der Meer (1988) can be used for example, but in fact each stability formula can be used. The required stone diameter for an overtooping breakwater can then be determined by:

$$\Delta_{n50} \quad (overtopped) \quad \Delta D_{n50} \quad (non-overtopped) = 1/(1.25 - 24R*) \tag{4.106}$$

for $0 < R* < 0.52$

where:

$$R^* = \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}; \qquad s_{op} = \frac{2\pi H_s}{gT_p^2} \tag{4.107}$$

H_s = significant wave height
T_p = wave top period
R_c = the crest height measured form the water level.

An average stability increase of 25% is obtained for a structure with the crest at the water level. The required weight in that case is a factor $(1/1.25)^3 = 0.8^3 + 0.51$ of that required for a non-overtopped structure.

The stability of submerged breakwaters depends on the relative crest height, the damage level and the spectral stability number. The stability (based on the tests by Givler and Sorensen, 1986) is described by:

$$h_c'/d_s = (2.1 + 0.15)exp(-0.14N*_s) \tag{4.108}$$

and

$$N*_s = H_{mo}^{2/3}L_p^{1/3}/\Delta D_{n50} = \frac{H_{mo}}{\Delta D_{n50}s_p^{1/3}} \cong N_s S_{op}^{-1/3} \tag{4.109}$$

where
$N_s = \frac{H_s}{\Delta D_{n50}}$
$s_{op} = \frac{H_s}{L_p} = 2\Pi H_s/gT_p^2$
H_o = significant wave height ($= 4\sqrt{mo}$), based on spectral analysis
(m_o = zero moment of wave spectrum); roughly $H_{mo} \cong H_s$

For fixed crest height, water level, damage level, and wave height and period, the required ΔD_{n50} can be calculated, giving finally the required stone weight.

Eq. 37 is shown in the lower plot of Fig.4.47 for three damage levels and can be used as a design graph. Here S=2 is start of damage, S=5-8 is moderate damage and S=12 is "failure" (lowering of crest by more than one diameter).

Monolithic submerged breakwaters consisting of precast units may be designed by the method proposed by Druet (1961), which is based on the following assumptions:
a. normal wave incidence
b.free foundation on gravel or stone
c. width at foot is greater than height of breakwater and half length (but smaller than L)
d. submergence sufficient to avoid wave breaking
e. stability provided excusively by friction

The critical weight of submerged breakwater is given as follows

$$G = \lambda \cdot G_{cr} \tag{4.110}$$

$$G_{cr} = \left(1.1 - 0.08\frac{L}{2B}\right) p_H h_B L \left[\frac{1}{\mu} + 0.5(\mathrm{ctg}\beta - \mathrm{ctg}\alpha)\right] \cdot \left(\frac{A}{B} + 1\right) + V_{st} \tag{4.111}$$

in which
λ = 1.5 = safety factor with regard to sliding
L_B = length structure
B = width at foot
A = width at crest
h_s = height of breakwater
μ = friction factor at foot

α, β = angles of breakwater walls,$\beta < \alpha$
P_H = wave induced pressure given by Druet (1961)
V_{st}- hydrostatic pressure acting on the submerged breakwater.

Laboratory tests by Aminti, Lamberti and Liberatore (1983) for various shapes of submerged breakwaters pinpoint erosion on the shoreward side of the structures. The breakwaters were founded on depth of 2 ... 3 m (prototype) and subjected to regular and irregular wave action. The extremum case corresponded to loss of stability. Counter-scour measures are recommended on the shoreward side in the form of stone-filled gabions on geotextile filters. This is recommendable for impervious gravity structures made of reinforced-concrete, but seems less advisable for revetments.

CHAPTER 5

Examples of design

5.1 GENERAL

Let us first refer to the aforementioned general shore and beach protection policies, cf. Chapter 1.

As put by Bijker and Graaff (1989), the sea needs a certain "play-ground". The construction of a seawall or alternative protection measures comes up as a certain possibility to avoid **dune erosion** problems. A well-designed shore-parallel structure may indeed overcome the erosion problems due to a severe storm surge. A sea wall can diminish the "play-ground" of the sea. It is argued, cf. Steetzel (1987), that if the dune erosion is prevented due to the construction of a revetment against the dune front, severe erosion just in front of the revetment has to be expected. If the so-called "denied" volume (= volume eroded from the dunes if the revetment was not there) is large, the depth increase in front of the construction may become quite considerable, cf. Chapter 3.

It is to remember that after the first years following the construction, a period with increasing problems starts. The beaches in front of the seawall will be lowered, waves reach more easily and more frequently the seawall, and the attack on the seawall increases. To prevent serious damage the seawall has to be reinforced or has to be "protected". In spite of these countermeasures the erosion of the beaches continues and after a certain time all the "protected" beaches might disappear. The seawall has little by little changed in a sea dyke.

The basic point to emphasize is that erosion is a gradual problem which can be overcome by two methods available in principle

1. Feed from time to time artificially the amount of material which has apparently been lost along the coastline (beach nourishment)

2. Try to control the sediment transport process which causes the erosion. This interference should be such that the erosion stops in the area one likes to save.

Method (1) is almost trivial but becomes increasingly attractive and can be applied almost always. The Dutch Manual on Artificial Beach Nourishment (1987) is recommended to consult if an actual beach nourishment scheme has to be designed.

Method (2) is more difficult to implement. First of all it should be established which sediment transport causes the erosion. If that has been established one has next to decide how to interfere actually.

As shown extensively throughout this document, the coastal management options, concepts and implementations, aimed at controlling, alleviating or bypassing the realm of far- and near-field effects, are indeed very vast, and open numerous avenues for a coastal engineer and manager. In this chapter, it is our intention to exemplify the above policies with regard to different types of coastal structures and management measures.

The **examples** provided are not universal; they merely shed light on **different coastal defence measures** used in the engineering practice. For better comprehension, the reader is advised to consult the earlier chapters, with their cross-references.

5.2 CROSS-SHORE STRUCTURES

GENERAL DESIGN FEATURES OF GROYNES

Different examples of groyne design have been shown herein with reference to the far-field effects (Part One, Chapter 5 in Part Two and Chapter 2 in this Part Three), and are commented on in the context of combined systems (Chapter 6), national policies (Chapter 7) etc. It does not seem worthwhile to dwell on them at this point.

The design can vary from completely impermeable (Fig.5.1 a), through partly permeable (and overtopped; Fig.5.1 b) to single-pile rows of high permeability (Fig.5.1 c). As noted elsewhere, the construction materials include timber, stone, concrete, reinforced concrete, steel, geotextiles and fascine (mostly under heavier units to prevent their sinking), and other natural and man-made products.

Examples of different types of groynes constructed on the Norderney Island are shown in Fig.5.2. The first groynes there were built in 1860...1862, as shallow sloping structures with ashlar and basalt pavement (Fig.5.2 bottom). They did not prevent beach erosion as expected. After 1948 the groynes on Norderney were constructed as crib or pile type. Their main purpose has been to turn back the tidal flow of the foreshore area. Some of them helped reduce the scouring but they did not alter the trend of beach change. Somewhat similar features are inherent in the design of groynes constructed on the Baltic coastline. Fig.5.3 depicts different cross-sections and design patterns. Fig. 5.4 illustrates a heavy design using concrete slabs placed on fascine. Fig.5.5 shows a typical "Baltic" groyne using timber (top and centre), along with a stone groyne (bottom).

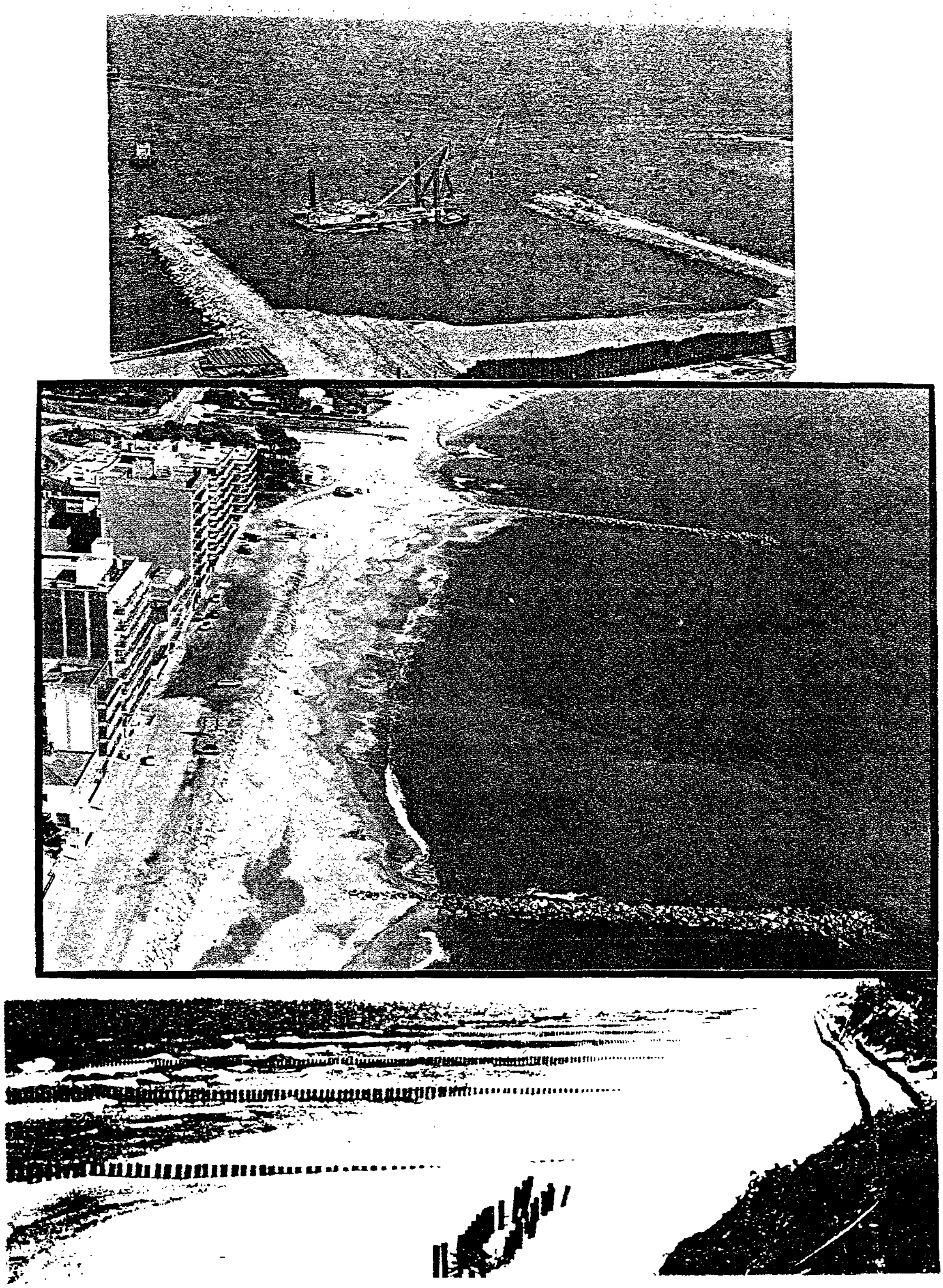


Figure 5.1. Different Degree of Groyne Permeability.

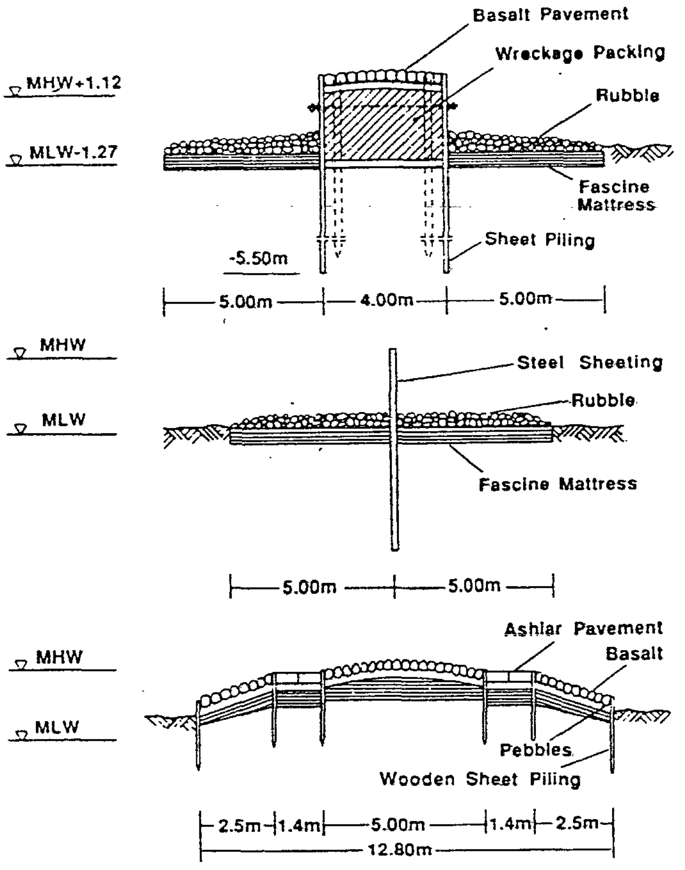


Figure 5.2. Crib Groyne (top), Pile Groyne (centre) and Shallow Sloping Stone Groyne (bottom) on Norderney Coastline.

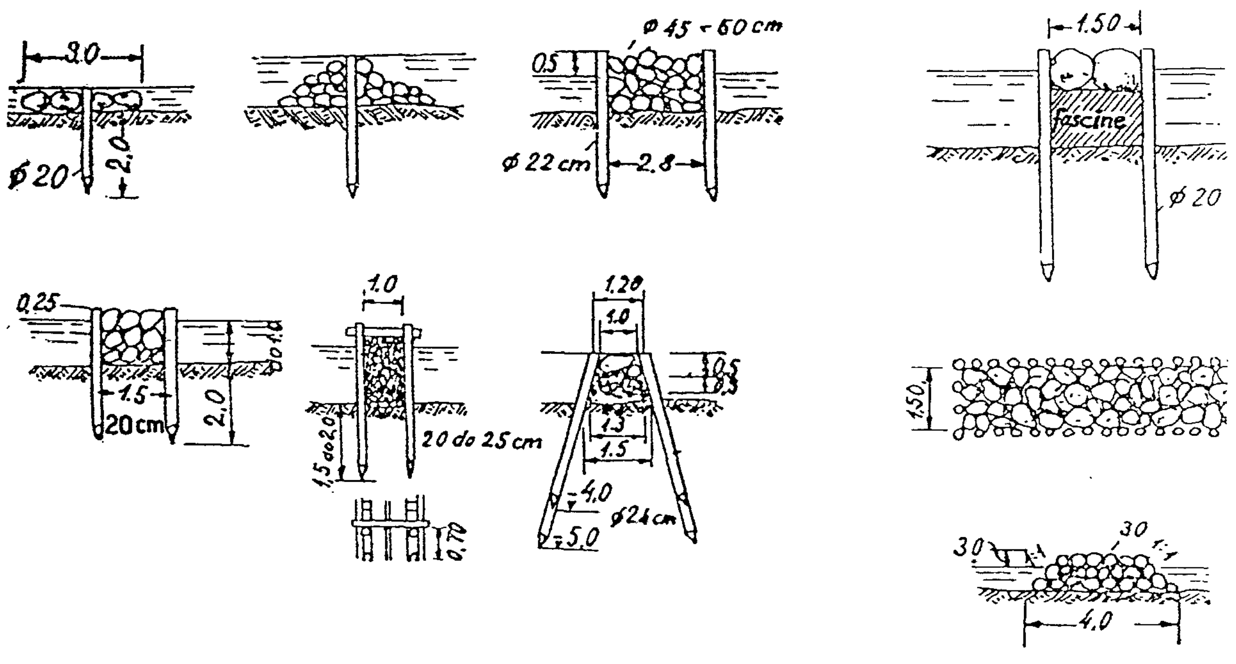


Figure 5.3. Cross-sections of Typical Groynes in Poland.

One should emphasize the importance of adequate failure prevention, which becomes crucial for proper functioning of groynes. Therefore we schematize the possible failure modes for single-row and double-row, or gravity-type design, in Fig.5.6 top and bottom, respectively.

An overview of the design problems arising in the construction of groynes is provided by the following subsection.

DUTCH RECONSTRUCTION EXPERIENCE

The Dutch coast is over a great length protected by groynes. Some of them are rather old (from the 19th century) and need to be refurbished. It was necesary to decide whether to keep the groynes in their original shape, or to change the layout. Along the southernmost stretch of the Dutch coast (near the border with Belgium) the crest level of the groynes is rather high above the beach level, because of continuous erosion of the shoreline. Because of the height of these groynes, they suffer severely from wave attack, and local authorities wish to lower the groynes in order to decrease the maintenance costs.

The groynes in this coastal section are very wide, and made of natural stone.

The question was to find the optimum height of stone groynes along a coast with strong tidal currents, cf. Verhagen (1988). The location and the length of the groynes were fixed and should not be altered. A review of the available literature showed that no ready-made answer was available to this problem. The general conclusion was that most authors state that a height of 1 m or less above the beach is optimum in order to catch sand, because higher groynes cause reflection of waves, which results in erosion of the beach near the groyne. However this conclusion is based upon visual observations and laboratory tests in nontidal situations with relatively short groynes.

The literature-review also revealed that no reliable mathematical models for the design of groynes were available. From the review it was concluded that a different approach to the problem was required. It has been decided to do an empirical parametric investigation to solve the problem.

A number of parameters have been defined to describe how effectively a groyne works, and the financial consequences of descreasing/increasing the height of a groyne were determined (both building costs, as well as capitalised maintenance costs).

With the first set of parameters the minimum and optimum height of a groyne can be determined from a view point of coastal engineering. Comparison with the costs give the overall optimum. In order to evaluate the quality of a groyne as a coastal defence work, the following steps can be made:

1. Determine how well the groyne can fulfil the seperate functions. This follows from the geometry of the groyne.

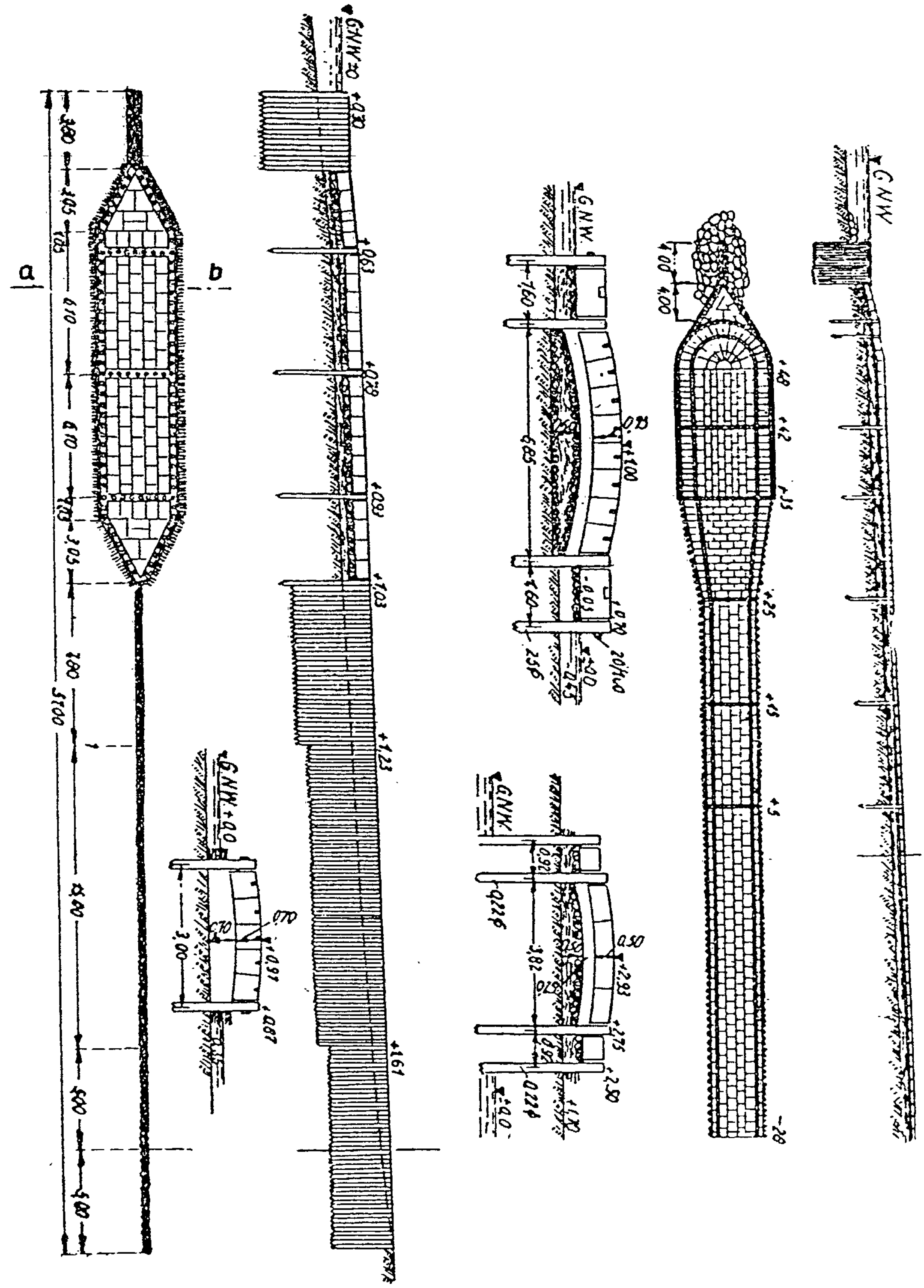


Figure 5.4. Concrete Blocks on Fascine in Poland.

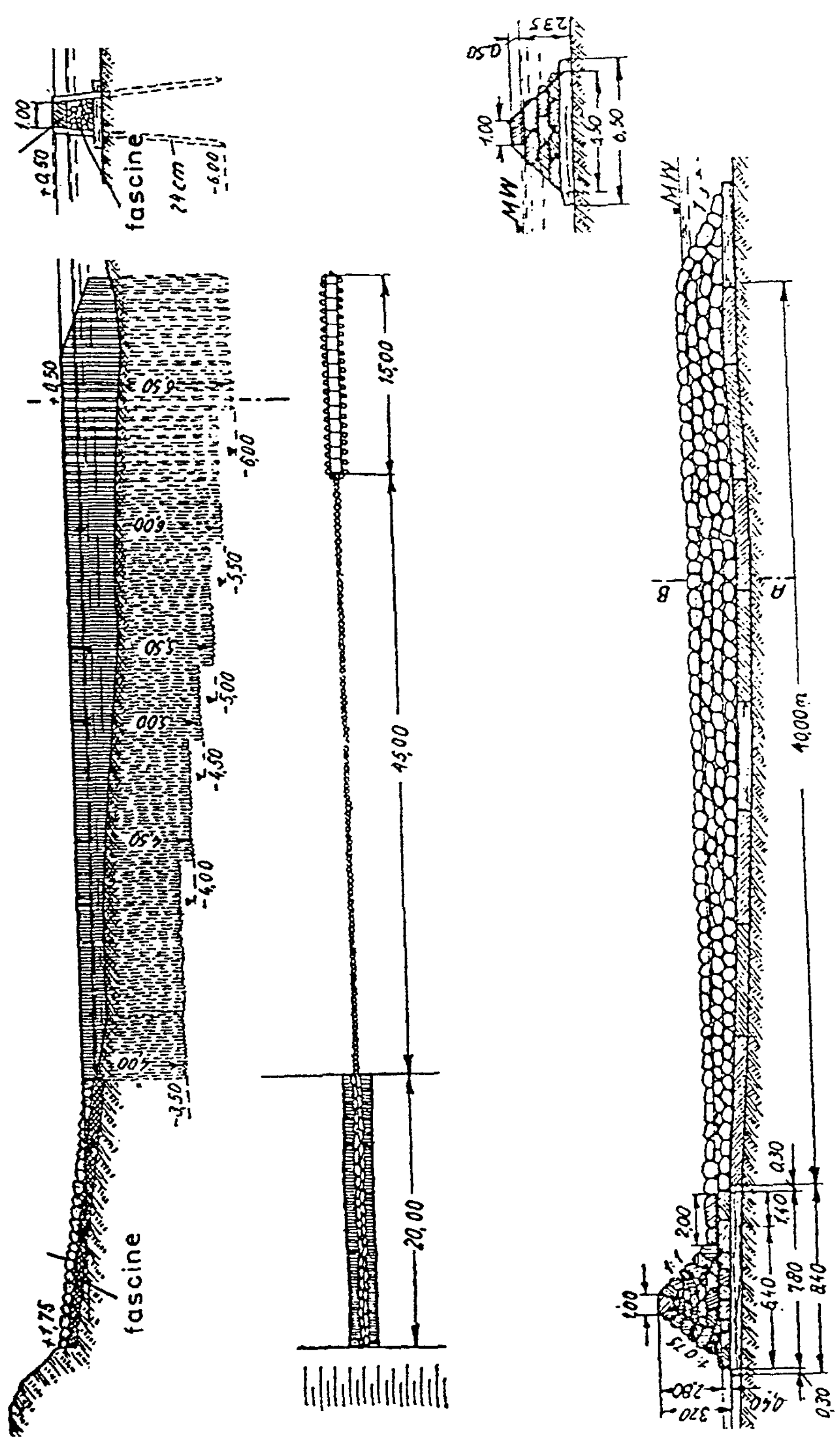


Figure 5.5. Groynes in Poland, incl. "Typical Baltic Groyne" (top and centre).

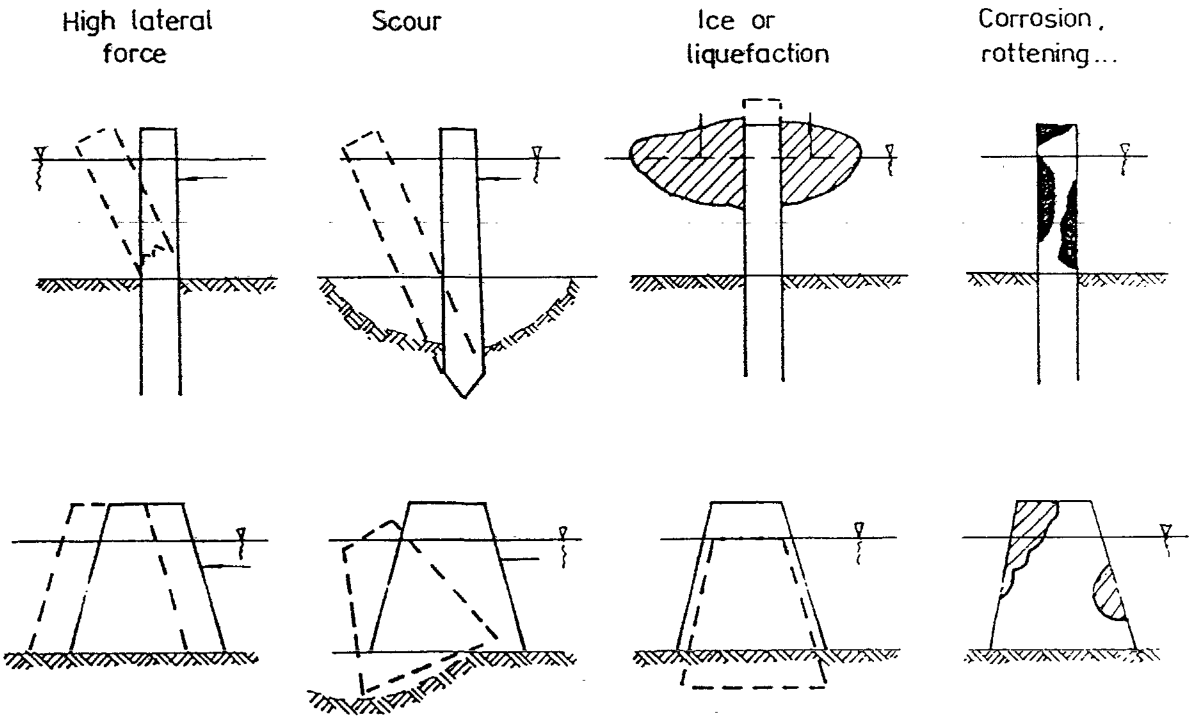


Figure 5.6. Groyne Failure Modes and Mechanisms.

2. Determine how well on the given location a groyne has to fulfil this functions. This follows from the hydro-morphological situation.
3. Determine how important it is that a groyne on this location completely fulfils the functions required.

This can be illustrated with the following example:

1. A groyne decreases the wave-driven current to a maximum, if the groyne is always higher than the water level. This is often not so. From the geometry of a given groyne it can be calculated that the wave-driven current is decreased to a certain part, for example, to 60%.
2. In order to prevent beach erosion near the groyne a strong reduction of the current is required. From a morphological evaluation follows, for example, that 80% current reduction is necessary to keep the beach in stable position.
3. From (1) and (2) it follows that the groyne does not operate as a surf-groyne, and that consequently beach erosion will occur. There may be reasons why, at this location the potential erosion is accepted, for example because it is known that within a short period there will be a supply of sand from a moving sand bank, fully compensating the expected erosion. Another reason may be that erosion is no problem because the dunes are relatively wide, and dune erosion can be allowed to a certain extent.

The parameters mentioned under (1) can be calculated from the geometry of the groynes. In order to estimate the attack of tidal currents and wave-driven currents, an analysis has to be made of the distribution of surf-energy along the coast and the depth of current erosion holes in front of the groynes. From the analysis the minimum value of the parameters follows.

Hydraulic factors

Because it is not possible with the present knowledge of hydraulics and morphology of groynes, to give unambiguous quantitative findings of the influence of lowering existing groynes on the erosion of beaches, one has to compare the groynes from a hydraulic point of view.

The idea behind it is that the morphological effect of a groyne (reduction of long-shore transport capacity and the harmful effect of sand loss in seaward direction) can be described seperately for each hydraulic factor.

Longshore transport is determined by the following factors: tidal current, wave-driven current, turbulence due to breaking waves (and in an indirect way by the lee-zone behind the groyne).

When for all the groynes along the coastline these factors are available it is possible to draw conclusions on the desirable magnitude of the factors for each groyne.

Six factors have been defined, which are supposed to describe the morphological effect of a groyne. They are identified through their major functions as "current groyne" and "beach groyne", cf. Verhagen (1988).

Current groyne

<u>Factor 1: tidal current reduction</u>

The presence of a groyne, with or without piles, has a direct restraining influence on the tidal current in this zone, and consequently a reducing effect on the sediment transport along the coast.

<u>Factor 2: head-effect</u>

Due to the current reduction over the length of the groyne there will be a current-contraction just in front of the head of the groyne. Due to this erosion an increase loss of sand in offshore direction may occur.

<u>Factor 3: current turbulence</u>

Because of the current between the piles and over the crest of the groyne, due to difference in sand transport and sand transport capacity, the extra turbulence may cause an erosion pit at the downstream side of the groyne.

Beach groyne

<u>Factor 4: wave-current reduction</u>

The presence of a groyne causes an extra resistance in the surf zone against the wave-driven current. This has a direct influence on the longshore transport.

Factor 5: wave turbulence

When a groyne is relatively high above the beach, there will be wave reflection against the groyne, causing extra turbulence. In combination with a seaward directed surf-induced current (rip current) this may cause considerable erosion on the updrift side.

Factor 6: lee-factor

If a groyne has an oblique direction with respect to the incoming waves, then there will be a shadow area behind the groyne. The wave attack in this area will decrease, which has a reducing effect on the longshore transport.

For each factor a mathematical description is given, based upon the geometry of the groyne. The parameters describing the factors have been chosen in such a way that the calculated value is always between 0 and 1, in which 0 means "bad" (no current reduction, big head effect) and 1 means a "good" groyne.

Computation of the various factors

Factor 1: tidal current reduction

The velocity on the beach v_1 is compared with the velocity on the beach at the same location, if the groyne was not present (v_o). A streamline with the length of the average distance between the groynes is regarded (see Fig.5.7). In a situation without a groyne the velocity is:

$$v_o = C\sqrt{h\frac{\Delta H}{L}} \tag{5.1}$$

The current velocity in presence of a groyne is:

$$v_1 = C\sqrt{h\frac{\Delta H_1}{L}} \tag{5.2}$$

in which:

$\Delta H_1 = \Delta H - \Delta H_2 - \Delta H_p$

The resistance due to the groyne without piles reads

$$\Delta H_2 = \mu\frac{(v_2 - v_1)^2}{2g} = \mu\frac{v_1^2}{2g}(\frac{h}{h^2} - 1)^2 = \mu\frac{v_1^2}{2g}(\frac{d}{h-d})^2 \tag{5.3}$$

and the resistance due to the piles is:

$$\Delta H_p = \eta\frac{v_2^2}{2g} = \eta\frac{v_1^2}{2g}(\frac{h}{h_2})^2 = \eta\frac{v_1^2}{2g}(\frac{h}{h-d})^2 \tag{5.4}$$

and:

$$\Delta H = \frac{V_0^2 L}{C^2 h} \tag{5.5}$$

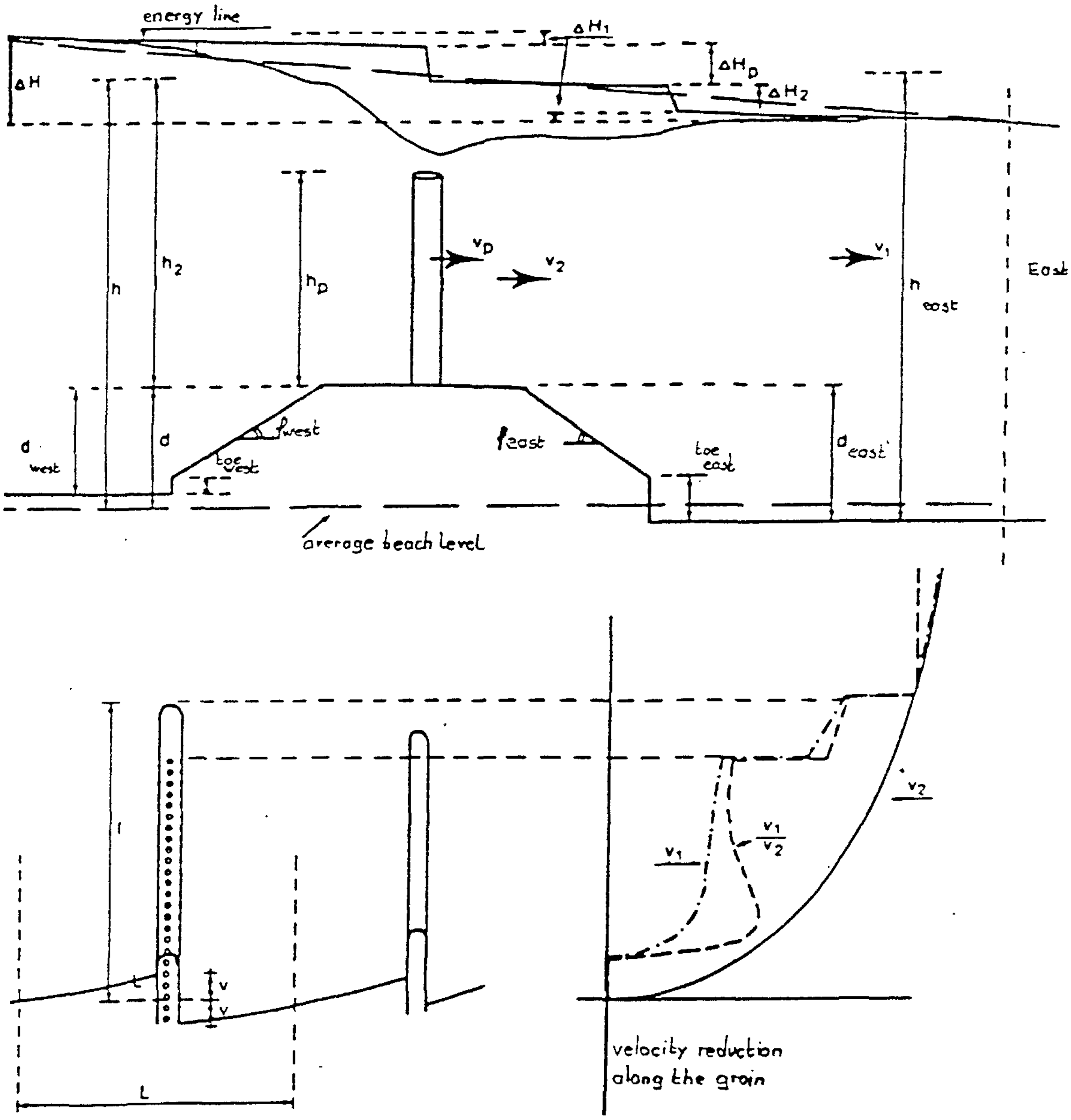


Figure 5.7. Verhagen's (1988) Schematization of Groynes and Flow Field.

The dimensionless current reduction in a streamline is:

$$\frac{V_1}{V_0} = \frac{(\Delta H_1)^{1/2}}{H} = \sqrt{1 - \frac{\Delta H_2 + \Delta H_p}{\Delta H}} \tag{5.6}$$

or:

$$\frac{V_1}{V_0} = \sqrt{1 - \frac{\mu \frac{v_1^2}{2g}(\frac{d}{h-d})^2 + \eta \frac{v_1^2}{2g}(\frac{h}{h-d})^2}{\frac{v_0^2 L}{C^2 h}}} \tag{5.7}$$

or:

$$\frac{v_1}{v_0} = \sqrt{\frac{1}{1 + \frac{C^2 h[\mu(\frac{d}{h-d})^2 + \eta(\frac{h}{h-d})^2]}{2gL}}} \tag{5.8}$$

The average current reduction over the total wet length L of the groyne is:

$$(\frac{v_1}{v_o})_{ov} = \frac{\int_0^1 \frac{v_1}{v_0} dl}{l} \tag{5.9}$$

The parameter describing the factor tidal current reduction factor f_1 is

$$f_1 = 1 - \frac{\int_0^1 \frac{v_1}{v_0} dl}{l} \tag{5.10}$$

The factor v_1/v_0 indicates the current reduction. To calculate the current velocity in a streamlane, this value has to multiplied with the undisturbed velocity v_0. This is indicated Fig.5.8 (bottom).

The coefficients μ, η and C require explanation:

- The loss-coefficient μ at sudden reduction in velocity depends on the abruptness of the increase of profile.

Φ; deg	6	10	15	20	30	60	90
μ	0.14	0.20	0.30	0.40	0.70	1.10	1.10

For the groynes in Verhagen's study Φ varied between 4° and 20° - The loss η for current through a pile-screen is:

$$\eta = R^{2/3} * \frac{h_p}{h-d} * 1.7(\frac{D}{B})^{4/3} \tag{5.11}$$

in which R is the number of pile-rows. The formula is based upon Delft Hydraulics Laboratory's research.

Factor 2: head-effect

The contraction of flow-lines around the head of a groyne can be described by the velocity-gradient over the head if the pile-row is terminated near the head; this effect will be stronger. The gradient is:

$$\Delta V = \Delta_2 = \Delta_1 exp(-l_1/20) \tag{5.12}$$

in which Δ_1 = difference in velocity reduction between the head of the groyne and the end of the piles

Δ_2 = difference in velocity reduction over the head of the groyne

l_1 = distance between the head and the end of the pilerow.

Turbulence due to contraction is supposed to be quadratic function of the velocity gradient. Thus:

$$f_2 = 1 - \Delta v^2 \tag{5.13}$$

Factor 3: current turbulence

The amount of turbulence on the downstream side of the groyne can be described by the energy loss over the crest.

$$\Delta H_2 = \mu_2 \frac{(v_2 - v_1)^2}{2g} = \mu_2 \frac{v_1^2}{2g}[\frac{d}{h-d}]^2 \tag{5.14}$$

It is supposed that the turbulence due to the current between the piles (ΔH_p) is situated above the protected part of the groyne, and is therefore discarded. Compare the energy loss ΔH_2 with the maximum loss in this flow lane $\Delta H_m ax = v_2/2g$, thus:

$$\frac{\Delta H_2}{\Delta H_{max}} = \mu(\frac{V_1}{V_2})^2(\frac{d}{h-d})^2 = \mu_2 * (\frac{d}{h})^2 \tag{5.15}$$

The coefficient μ_2 gives the extra turbulence due to a vertical wall and rubble flanking the groyne. This value has to be averaged over the length of the groyne, applying the square root of the water depth as a weighing factor:

$$f_3 = 1 - \frac{\int_0^1 \frac{\Delta H_2}{\Delta_{max}} \sqrt{h} dl}{\int_0^1 \sqrt{h} dl} \tag{5.16}$$

Factor 4: wave-current reduction

A breaking wave is the generating power for the surf-current; the bottom shear stress is the dissipating power. A groyne will increase the resistance, and thus the dissipating power. The surf-current is a linear function of the bottom shear stress, so a linear change in shear stress causes a linear change in the velocity.

In the definition of factor 4 it is assumed that the increase is resistance is equal to the quotient of groyne-height/water depth. Averaged over the length of the groyne this is:

$$f_4 = \frac{\int_0^1 \frac{d}{h} dl}{l} \tag{5.17}$$

Factor 5: wave turbulence

Waves reflected by the vertical side of a groyne are causing turbulence on the upstream side. It is assumed that the amount of turbulence is a function of the wave reflection:

- wave reflection increases linearly with the groyne-height
- wave reflection is maximal for a vertical groyne
- wave reflection on a sloping groyne is a function of the slope Φ, according to $\sin^2 \Phi$
- oblique waves have a turbulence as function of the angle of approach α according to $\sqrt{\sin \alpha}$

Factor 6: lee factor

The presence of a groyne will generate a lee area in the wave field. The size of this area depends on the groyne height, wave height and the orientation of the groyne. The size of the lee-area is estimated as: $\frac{1}{2}(l_{shadow})^2 \tan \alpha$ The length l_{shadow} gives a point on the groyne where $h - d = 0.5m$. The lee-area is compared with a standard area of 100x100 m^2 to find factor 6. For computation of the various factors a computer-program is available, Verhagen (1988).

For a coastal section in the south-west of the Netherlands, near Breskens, various factors have been calculated. In Fig.5.8 the factors 1, 4 and 6 are plotted as a function of their height above the beach. Also the factors for a normalised "test groyne" are given, with one and with two rows of piles (R = 1 and R = 2).

For the design of groyne reconstruction, with this technique a number of alternative designs can be evaluated. As an example groyne number 8 near Breskens is worked out. For this groyne, using the computer program, all six factors have been calculated, with varying crest heights, one and two rows of piles and a varying pile-length. Figure 5 in Verhagen (1988) gives the results of these calculations. One also determined the costs of reconstruction (capitalized and maintenance costs).

This has been done for refurbishment within the actual profile (Dfl 690.000) and for reconstruction with varying crest heights. Reconstruction at the actual crest height costs Dfl 770.000 (level at $1.20m$). Reconstruction at a height of only $0.50m$ above the beach costs Dfl 555.000. It is assumed that the slopes of the groyne are continued in stone until $0.20m$ below the beach. For crests in between costs are linearly interpolated.

From these two aspects:

1. costs at varying crest height
2. hydraulic factors at varying crest height

Diagrams have been composed, which have given the relation between costs and the hydraulic factors. These diagrams show a discontinuity at a crest height of $0.80m$ because at lower crest heights, the slopes have to be continued at greater

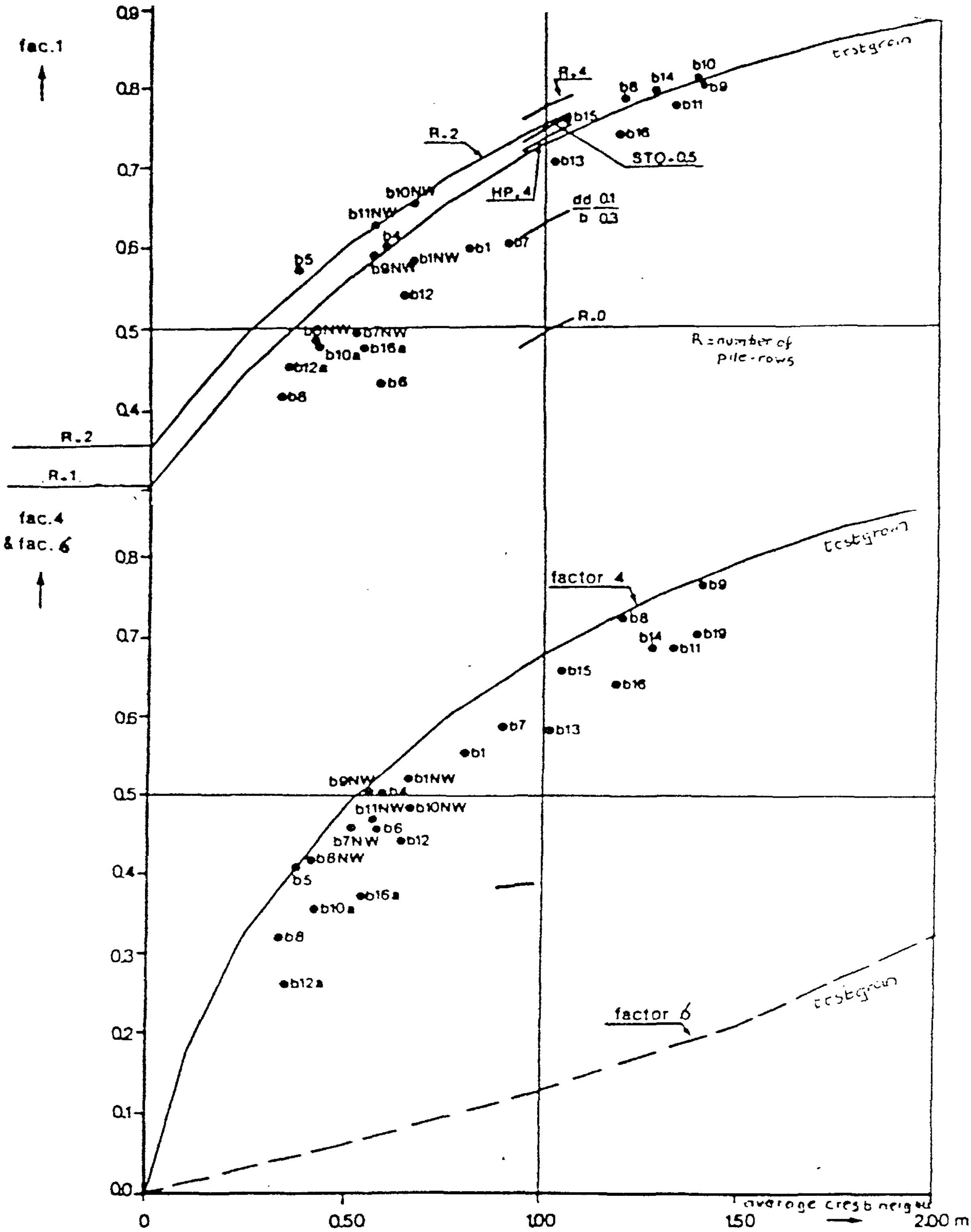


Figure 5.8. Factors 1, 4 and 6 versus Groyne Height, Verhagen (1988).

depth into the beach sand, in order to prevent undermining by scour.

From morphological investigations it followed that in this area the tidal current reduction (factor 1) is most important. In order to work properly, this factor should be at least 0.65, preferably higher.[1]

The application of a second pilerow makes it possible to lower the groyne with $0.10m$, keeping factor 1 at the same level.[2]. Refurbishment of the existing groyne with the present geometry is also possible. It gives a factor 1 of 0.78, but costs Dfl 45.000 more than to construct a new groyne.

It might be possible that the "optimum current groyne" is not so good regarding the other factors. From the diagrams of the other factors it follows the with respect to wave-current reduction the "optimum groyne" is not particularly good. However, surf-induced currents are of minor importance in this area, and consequently a factor fo 0.6 can be accepted. Making the groyne lower and applying two rows of piles not only makes the groyne more expensive, but also is worse regarding the surf-induced current. Doubling the pile-rows is only profitable for the wave-turbulence factor. The improvement to gain (factor 5 increases from 0.55 to 0.58) is too small to balance the increase in cost (with dfgl 55.000) and the lowering of factor 4 and 2. So, there remain two realistic options:

1. reconstruction at a crest level of $0.80m$ above the beach, using one pile-row
2. refurbishment of the groyne, keeping the existing geometry.

Summarised this has the following consequences:

	Present situation	refurbisment within present geometry	reconstruction at d = $0.80m$
Factor 1,current reduction	0.78	0.78	0.66
factor 4, wave-current reduction	0.72	0.72	0.61
factor 6, lee factor	0.35	0.35	0.21
factor 3, current turbulence	0.80	0.96	0.96
factor 2, head effect	0.61	0.68	0.72
factor 5, wave turbulence	0.47	0.48	0.55
costs		Dfl 690.000	Dfl 645.000

In this case no simple decision can be made; there are about as many advantages as disadvantages. A decision can be made by studying also the neighbouring

[1]From Verhagen (1988) it follows that costs increase significantly if it is tried to raise this factor above 0.67. Reconstruction of the groyne, keeping the crest height at the present height ($d = 0.80m$) costs Dfl 645.000. Factor 1 is in that case 0.66.

[2]Financially this lowering of the groyne with $0.10m$ is no improvement; the groyne becomes Dfl 55.000 more expensive

groynes. The geometry of the subsequent groynes along one coastal section should not vary too much.

In this way every individual groyne can be judged. If the whole groyne system is judged, reconstruction has to be planned in such a way that the group becomes rather homogeneous.

Morphological considerations

A few paragraphs ago it was mentioned that "from morphological considerations followed that factor 1 should be at least 0.65". The problem is always to carry out these considerations.

In case of this example a full analysis of the whole coastal stretch has been made and, among others, the following parameters have been determined for each groyne:

- difference in beach-height at left/right side of the groyne
- measured head-effect (=depth of scouring hole in front of the groyne)
- potential erosion/accretion.

The last parameter was determined by calculating sediment transport with the CERC-formula (assuming absence of groynes) and using the difference in potential sediment transport to calculate the potential erosion.

Also, the actual coastal regression was determined, as well as the six-factors for all the 70 groynes in the area.

From these data it followed that, in cases with heavy current attack (deep scouring holes) groynes with a factor 1 value lower than 0.65 were not effective.

In those cases with relatively high potential erosion, groynes were effective with the factor 4 of 0.75 and higher. These figures are only valid for the particular coastal stretch studied. Universal application of the given vaues 0.65 and 0.75 is not possible. They have to be determined for every coastal strech.

Conclusions of the Groyne Reconstruction Study

In many parts of the world groynes are applied. For groynes which do not simply work as sand transport traps, there are no mathematical methods to calculate the optimum geometry.

In many cases the groynes built in the past were based upon experience only. When erosion is not completely stopped, groynes are rising above the beach level, and maintenance becomes expensive. Because of high maintenance costs the coastal manager wants to lower the crest levels (sometimes even below the beach level, because there are no maintenance costs any more), From a viewpoint of coastal morphology high groynes are attractive. Using the technique described by Verhagen (1988) it is possible to determine the optimum groyne-height for existing groyne-fields in tidal areas, thus not to design new groyne fields.

The following **notation** has been used throughout this subsection:

ΔH = energy loss in current without groynes
ΔH_1 = energy loss over the beach while a groyne is present
ΔH_2 = energy loss over the crest of groyne
ΔH_p = energy loss over pile
h = average water depth on beach
h_2 = water depth above the crest of groyne
d = height of the groyne above beach
h_p = length of piles
v_o = undisturbed current without groynes
v_1 = reduced current
l = length of groyne
L = distance between two groynes
$g = 9.81 m/s^2$ =acceleration due to gravity
C = Chezy-coefficient
D = diameter of piles
B = distance between piles
η = loss-coefficient by current through piles
μ = loss-coefficient by current over the crest
ϕ = slope of groyne
x = angle of wave incidence
n = number of pilerows

5.3 OFFSHORE BREAKWATERS

Design of offshore breakwaters usually incorporates **rubble-mound** of natural rock or concrete units. Precast elements of diversified shapes are employed, as mentioned and illustrated in Section 4.2.

Many structural types are now being examined, ranging from traditional breakwaters, sandbags, prefabricated shell-shaped structures, to suitably shaped concrete blocks, etc. Investigations are under way and preliminary results are encouraging, as noted earlier, cf. Cortemiglia et al. (1986).

Wilkinson and Allsop (1983) discuss various artificial armour elements. Known as "hollow" (see Chapter 4), many of them are cheaper and more effective in wave energy dissipation than the earlier versions of armour units, such as the tetrapodes, doloses etc. As already mentioned, the hollow block design has three main cost advantages over the interlocking block design. These are:

1. Steeper slopes on the seaward face of a breakwater can be used, which might greatly reduce quantities of stone.

2. The hollow blocks needed for any given wave size or condition are very much smaller and lighter than those needed for interlocking blocks.

3. Only one layer of hollow blocks is required here as in many cases two layers of interlocking blocks are considered mecessary.

Hollow blocks of COBS type (Fig. 4.45) were used in the construction of the Jersey breakwaters in the Channel Islands and on the Das Island in the Persian Gulf. The following highligthts may be extracted from the documented experience on operation of existing breakwaters using hollow blocks:

1. A single thickness of hollow 2000-kg blocks placed on a rubble slope of 3 on 4 can withstand waves of up to at least 5 metres in height.

2. Larger blocks will withstand larger waves.

3. Blocks should be secured firmly at the toe. If the toe is below heavy wave action it is more easily secured.

4. The top row of blocks must be prevented from moving upwards, if subjected to overtopping waves, by a parapet wall, concrete slab or large stones.

5. Most of the stone under the blocks can be smaller than the holes in the hollow blocks but there should be sufficient large stone to prevent downward movement of small stone.

6. Two or three adjacent blocks can be removed from a surface without a failure or loss of stone from below.

7. No empirical formulae have been derived to aid in the design of holow block protection. Hydraulic model tests should be carried out if blocks are required for conditions not yet tried or tested.

8. The steeper the slope the more stable the block surfaces. The limit is determined by the maximum steepness it is possible to build the rubble bank below.

9. Careful control during the casting of blocks and the use of pulverised fuel ash and chopped fibre in the concrete mix does increase the strength of the blocks and helps to prevent shrinkage or impact cracks.

10. It has not yet been conclusively proved which of the three hollow blocks is hydraulically the most efficient or stable. The reasons for the differences lie in copyright and preferences in munufacturing techniques.

Another type of blocks, HARO, is advertised by De Rouck et al.(1987). From laboratory investigations and qualitative as well as quantitative analysis of doloses, tetrapodes, accropodes and grooved cubes one can state that HARO is a safe

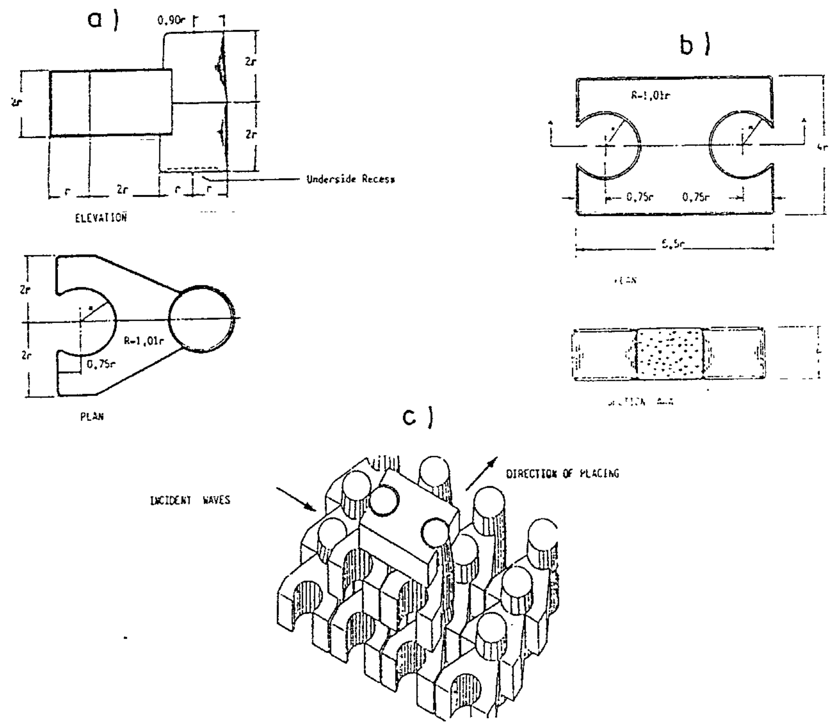


Figure 5.9. MONOBAR Blocks: Precast Unit (a); Collar Unit (b); Isometric View (c).

hydraulic mechanical and economic solution for protection of maritime structures. The primary properties of the elements include:

- The hydraulic characteristic stability coefficient K_o (run-up and reflection) of the HARO blocks are comparable with those of the so-called slender units.
- The structural characteristics (structural strength) and operational characteristics (casting and striking) of the HARO are comparable to those of the massive blocks.
- Preparation of the toe and the underlayer of the breakwater and placing the units on the slope are comparable for cubes, doloses, tetrapods, accropodes and HARO.
- Cost price of the armouring using HAROs is comparable to that found for slender units.

Another solution devised in Italy are hollow elements MANOBAR described by Benassi et al. (1986). The elements may be used for the construction of vertical walls and utilized for submerged breakwaters (Fig. 5.9).
Emerging breakwaters having typical cross-section of rubble -mound breakwaters, rectilinear and shore-parallel in plan view, are schematized in Fig.5.10 and Fig.5.11, taken from SPM where idealized cross-sections are recommended for nonbreaking and breaking wave conditions.

Precast elements were used in Italy, to the north of Riconte for construction of segmented breakwaters incorporating concrete piles of stellar cross-section. Individual V-shaped segments are situated on a depth of about 3.5 m. The design is described by Moretti and Pedone (1984) and Franco (1985).

The offshore breakwaters constructed in Leasowe Bay (Great Britain) are shown in Fig.5.12 ($H_s \approx 3.0$ m; T=5–7 s).

Submerged breakwaters may assume different shapes. Hueckel (1975) distinguishes four types of submerged breakwaters constructed in the Soviet Union (Fig. 5.14):

Type I is a rubble-mound breakwater, the crest and side of which are covered with heavy rock reaching 1000 kg and more. The internal core consists of finer boulders and gravel, which perform the role of filter during down-rush and prevent fine particles from removal. Type I is used under weak waves.

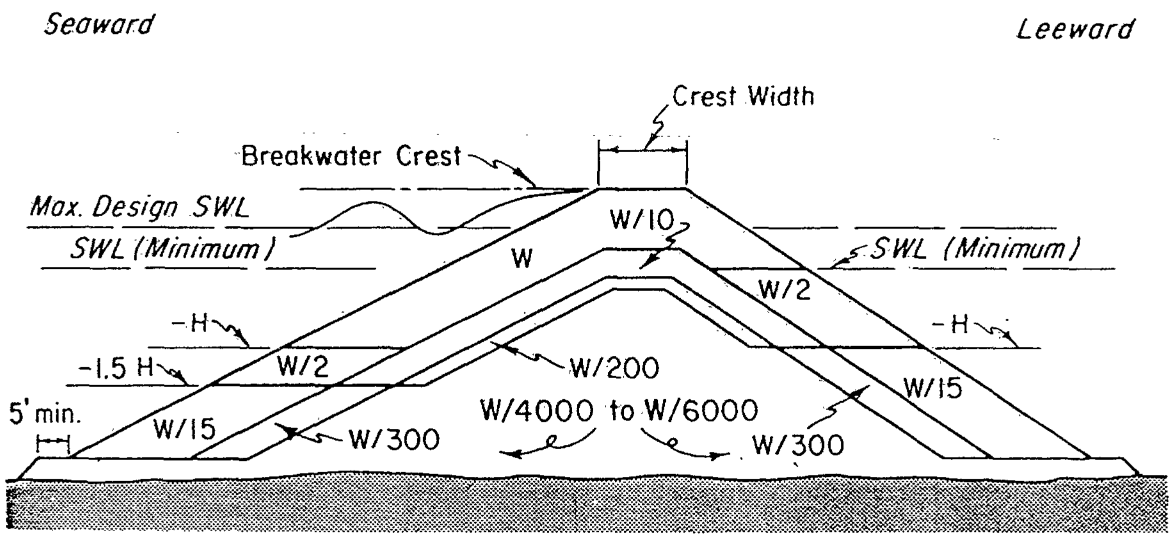
Type II is a submerged breakwater covered with concrete slabs. The toe of the structure may be implemented from blocks, steel bars or rails protruding by 60 cm above sea bed.

Type III is a rock dyke between two vertical sheet piles (mostly steel) and supported on the seaward side with a rubble revetment covered with concrete slabs. The slope is supported as in type II. The upper layer between the sheet piling may also be covered with concrete slabs or bags.

Type IV differs from other versions by having a gravity reinforced concrete caisson filled with stones and covered with heavy rock or concrete in bags.

Mixed design is also in use in the Soviet Union along the Black Sea coastline (Fig. 5.15). One of the solutions incorporates a submerged breakwater consisting of cuboidal concrete blocks or reinforced concrete caisons, the top of which is 0.5 to 0.8 m below SWL (Fig. 5.15 a). The cuboidal concrete blocks, 27 tonnes in weight, and $2 \times 2.3 \times 2.7$ m in size, were arranged on a stone subgrade 0.7 m thick. Some other segments of the submerged breakwater were made of caissons 5 m wide and 6.8 m long. The caissons are reinforced and stiffened with a central longitudinal wall 3 m high and 0.4 m thick. The angular walls are 0.8 m high and 0.2 m thick. The inside of the elements is filled with coarse stone. The seaward walls are inclined at 1 : 2.5.

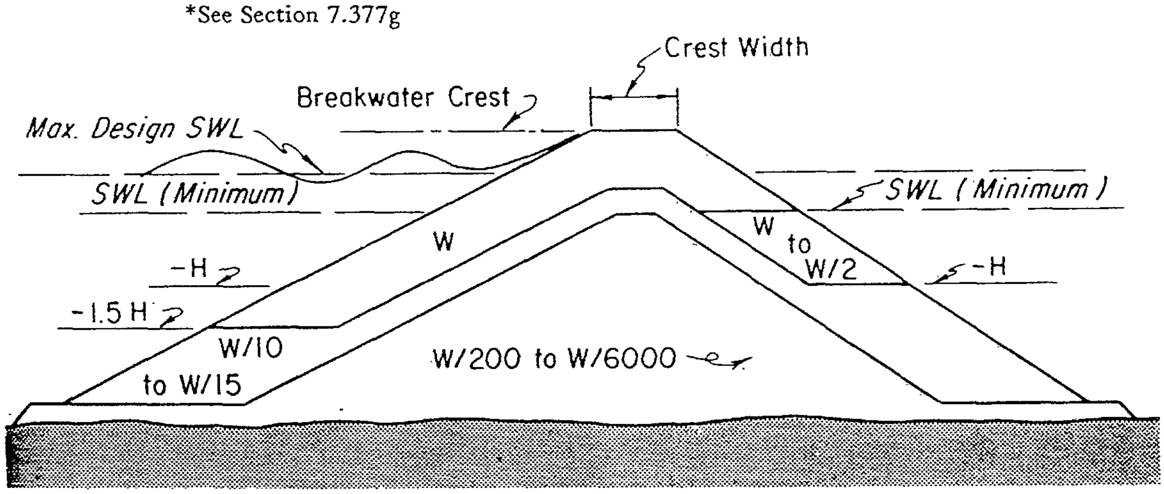
Another section consists of inclined concrete blocks weighing up to 100 tonnes (Fig.5.15.b). Two types of blocks are arranged on a subgrade 1.2 m thick. The upper edge of the submerged breakwater is 0.6 m below water level. The seaward angle of the blocks is 1:1.5; the dimensions are shown in Fig. 5.15.b. For economy, further sections are placed on thinner subgrades, 0.4 m, covered additionally with concrete slabs weighing up to 5 tonnes. All types of submerged breakwaters are shore connected by groynes to prevent longshore sediment transport. Mean depth of submerged breakwaters varies from 3 to 6 m.



Idealized Multilayer Section

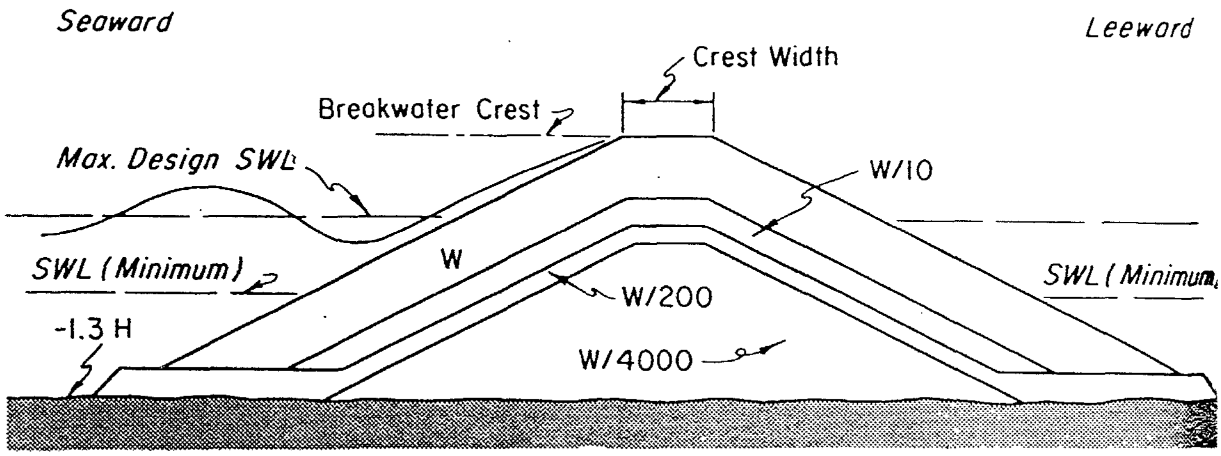
Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer	125 to 75
W/2 and W/15	Secondary Cover Layer	125 to 75
W/10 and W/300	First Underlayer*	130 to 70
W/200	Second Underlayer	150 to 50
W/4000–W/6000	Core and Bedding Layer	170 to 30

*See Section 7.377g



Recommended Three-layer Section

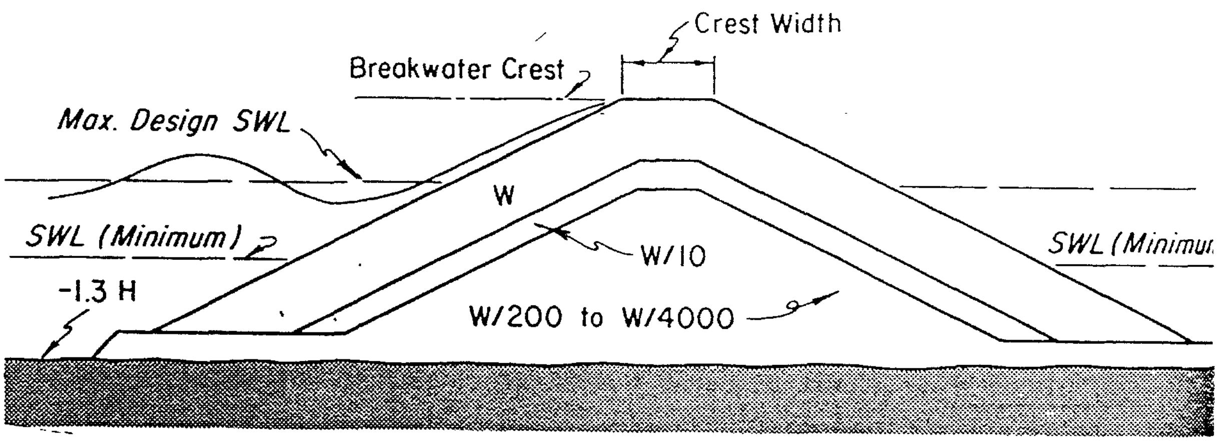
Figure 5.10. Rubble-Mound Section for Nonbreaking Wave Condition (zero to moderate overtopping), after Shore Protection Manual (U.S.Army 1977).



Idealized Multilayer Section

Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer	125 to 75
W/10	First Underlayer*	130 to 70
W/200	Second Underlayer	150 to 50
W/4000	Core and Bedding Layer	170 to 30

*See Section 7.377g



Recommended Three-layer Section

Figure 5.11. Rubble-Mound Section for Breaking Wave Condition (moderate overtopping), after SPM(1977).

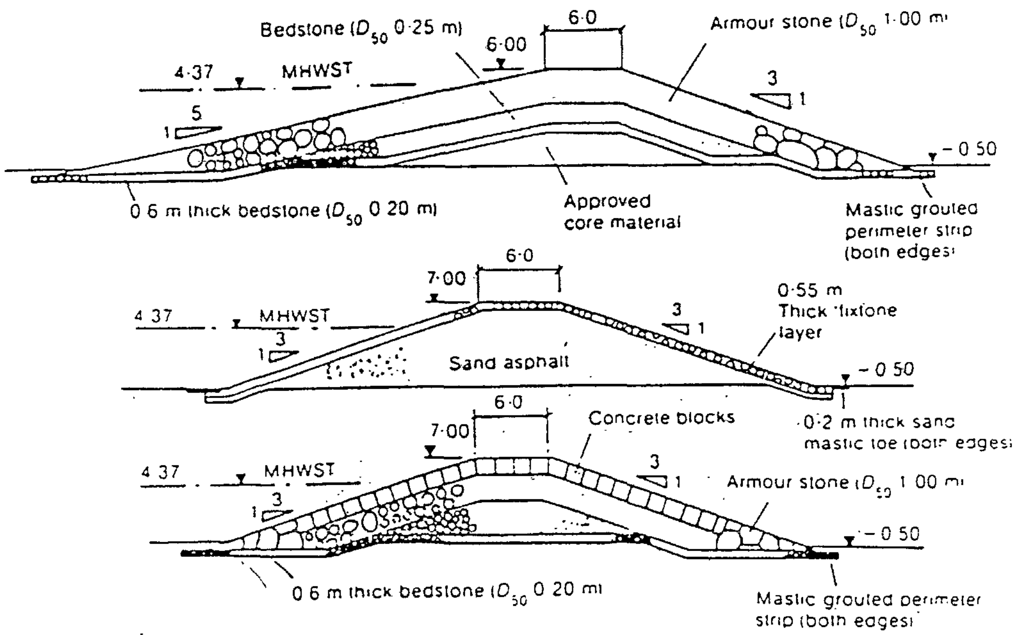


Figure 5.12. Offshore Breakwater Sections Designed for Leasowe Bay (GB); Barber & Davies (1985).

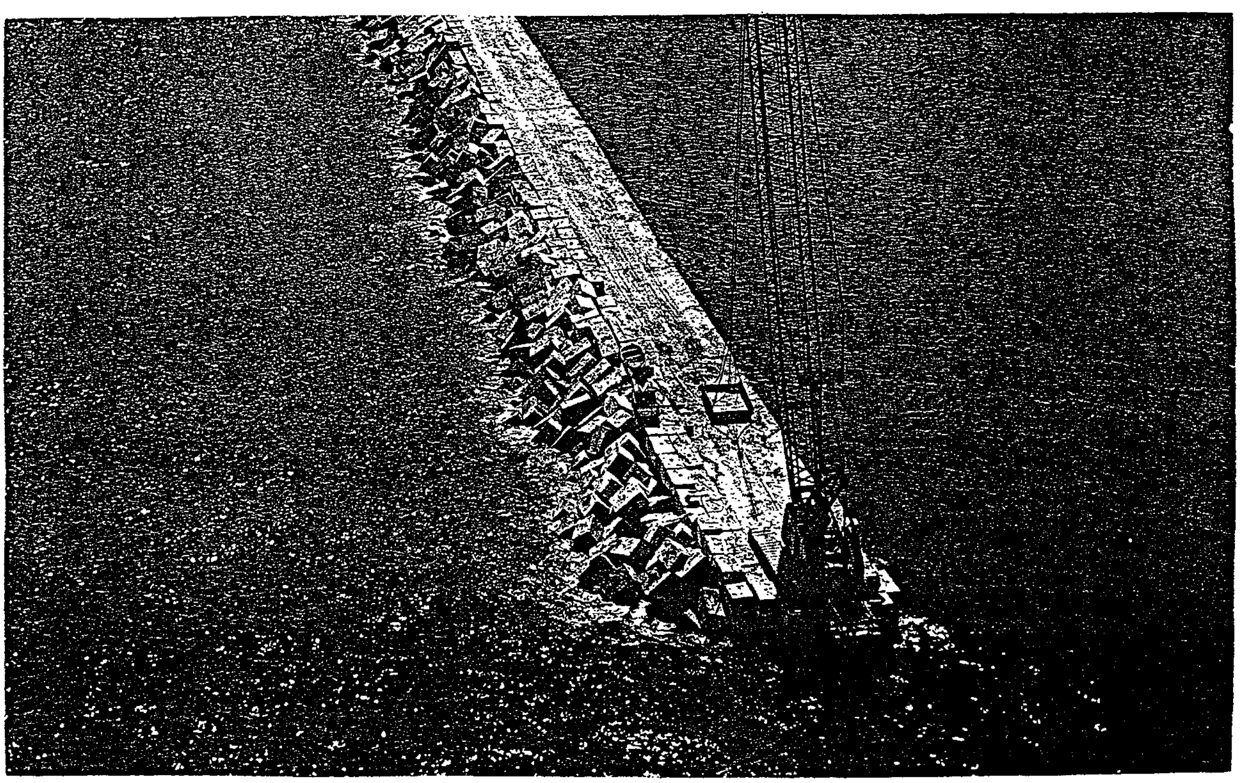


Figure 5.13. Breakwater Under Construction.

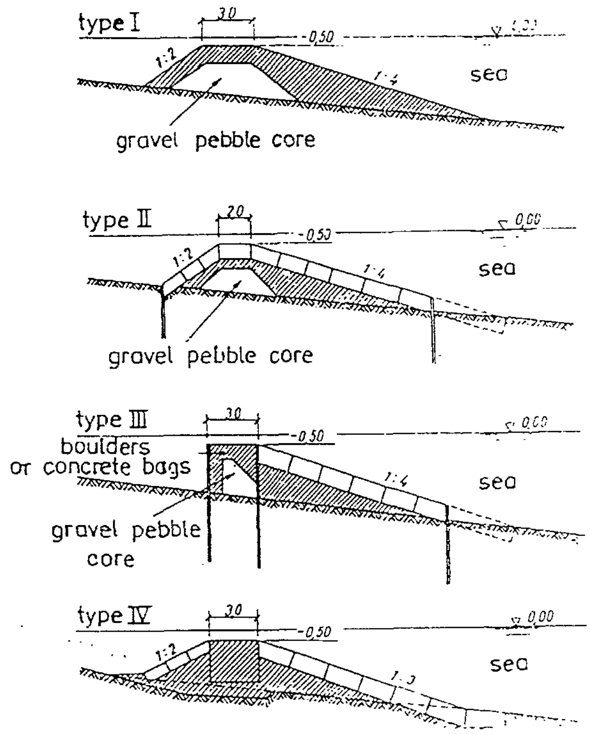


Figure 5.14. Submerged Breakwaters Constructed in the USSR.

Field investigations show that not all of submerged breakwaters are fully satisfactory. Better results in Soviet Union were obtained for precast reinforced concrete blocks 4 m long, weighing from 80 to 100 tonnes (Fig. 5.15 c). The stability is improved with anchors.
Spreading devices (1) are arranged to dissipate wave energy. Their upper edge should lie 0.8...1.0 m below SWL.

In Poland, Wilski patented a lightweight triangular submerged breakwater (sill), some 4.0...4.5 m high, submerged 2.5...3.0 m below SWL (Fig.5.16).

Prevention of overtopping

As noted earlier, if a structure is overtopped, even by minor splash, the stability can be affected. Overtopping can:
(a) erode the area above or behind the revetment, negating the structure's purpose;
(b) remove soil supporting the top of the revetment, leading to the unraveling of the structure from the top down; and
(c) increase the volume of water in the soil beneath the structure, contributing to drainage problems.
The effects of overtopping can be limited by choosing a higher crest-level or by amouring the bank above or behind the revetment with a splash-apron.

For a small amount of overtopping a grass-mat on clay can be adequate. The

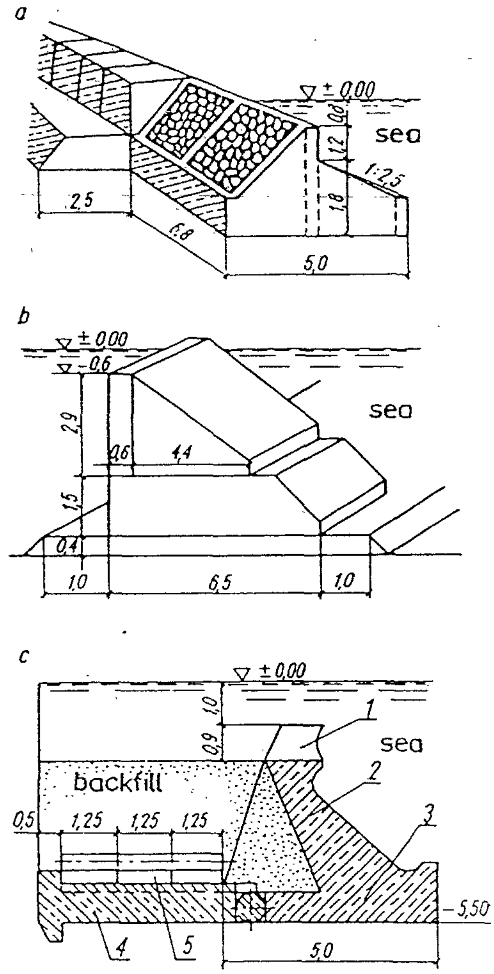


Figure 5.15. Submerged Breakwaters on the Soviet Black Sea Coastline, Hückel (1975);
hollow blocks (a)
slanted concrete blocks (b)
blocks with backfill (c).

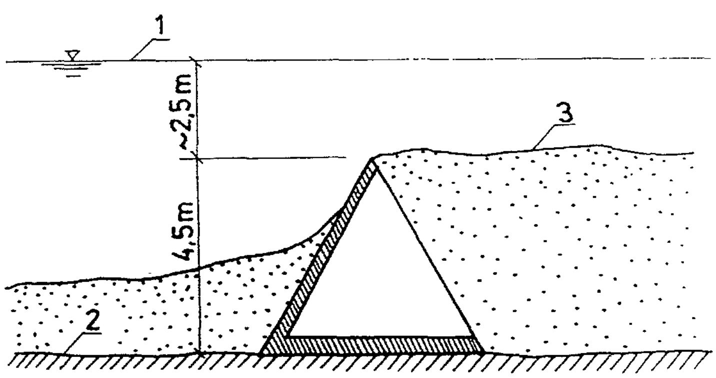


Figure 5.16. Polish Design of Submerged Breakwater Patented by Wilski.

splash apron can be a filter blanket covered by a bedding layer and, if necessary to prevent scour by splash, riprap or pavement of concrete units or asphalt.

No 100-% safe method for designing against overtopping is known; it is due to the lack of the proper method on estimating the hydraulic loading. For revetments, as a rule of thumb one may suggest to protect the crest over the width equal to the length of slope face from the crest to the point of run-up on the undisturbed slope and applying protection measures as for undisturbed slopes. For specific projects scale model studies are recommended.

Prevention of scouring of seabed in front of a **breakwater** is a common concern in Japan (Goda, 1985), from where a few hints can be borrowed.

A vertical breakwater reflects most of the wave energy incident on it, thus creating greater agitation at its front than that caused by a mound breakwater. This agitation is thought to enhance scouring of the bed in front of the breakwater. The disaster of the Mustapha breakwater in Algers Port in 1934 is a well-known example of seabed scouring, which is considered to have accelerated the collapse of the upright section [Ref.4.22]. It is interesting to note that in Japan there has been no case of the collapse of a breakwater due to seabed scouring, even though several hundred kilometers of vertical breakwaters have been built along the coast. Scouring takes places at the breakwaters in a number of ports and the tips of the rubble mounds are dislocated, but **repairs are always made before the stability of the upright sections is threatened.** The absence of breakwater collapse may also owe to the practice of providing a **quite broad berm and gentle slope** for the rubble mound in front of the upright section.

Nonetheless, dislocation of the rubble mound foundation due to scouring of the seabed is a phenomenon against which precautions must be taken. For this purpose, various materials such as plastic filters and asphalt mats are spread beneath the areas of the tip of the rubble mound and extended beyond it. However, no effective method has yet been found to stop scouring of the bed and dislocation of the tip of a rubble mound. Present practice is toward the revival of gravel matting, i.e. a thin layer of quarry run extended beyond the tip of the armour layer. Gravel may be dispersed by strong wave agitation, but a mixture of gravel and original bed material will withstand wave action for a longer duration than an unprotected seabed.

Berm Breakwater

The berm breakwater concept has been discussed in Chapter Four, where general guidelines have been provided for the design, primarily with regard to geometrical dimensioning. These guidelines are extended herein to a case study carried out by van der Meer (1988).

The following boundary conditions were chosen for the design of a berm breakwater designed for a site in the Spanish Mediterranean:

- water depth up to 18 m.
- no tidal range
- wave climate: 1/1 year :$H_s = 4.7m T_z = 8.2s$
1/5 years: $H_s = 5.9m T_z = 9.0s$
1/50 years:$H_s = 7.6m T_z = 10.0s$
- storm duration of 6 hours
- available stone classes: $0.5 - 9t, D_n50 = 1.01m$
$1 - 9t, D_n50 = 1.11m$
$3 - 9t, D_n50 = 1.19$ m
- relative mass density: $\Delta = 1.55$
- berm 0.5 m above the still water level (SWL).

Design of berm profile

The optimum values of m, n, b cf. Ch.4) and crest height will be estabilished for a water depth of 18 m and the 1/50 years wave conditions, $H_s = 7.6m$ and $T_z = 10.0s$. This means that the structure is designed for $H_s/\Delta D_n50 = 4.9$. The optimum value of b can be established for various combinations of m and n and for the stone class with $D_n50 = 1.01m$. The criterion for the optimum value of b was the minimum value for which the upper point of the beach crest was not a part of the erosion profile. In fact the upper point of the beach crest should lay on the initial slope, in order to prevent erosion of the crest of the initial profile.

For each combination of m and n the minimum calue of b was obtained interatively, using the computational model. Fig.5.17 shows the minimum length of the berm as a function of the upper and lower slope angles. The berm length decreases almost linearly with increasing lower slope, m. The same conclusion can be drawn for the upper slope, n. Fig.5.17 gives no information on optimum values for m and n. In fact, it provides various structures with more or less the same stability (no erosion on the upper slope). Therefore another criterion is introduced. The amount of stones required for construction can be minimized, giving the cheapest structure. The height of the upper point of the beach crest amounted from 7 to 9 m. The crest height of the initial profile was chosen at a fixed level of 9.5 m above SWL which is about 1.25 times the significant wave height. The area of the cross-section from the crest to the toe of the structure gives a measure of the amount of stones required. This amount, B, was plotted versus the lower slope and for various upper slopes in Fig.5.18. The berm lengths are the same as in Fig.5.17.

From Fig.5.18 it can be concluded that a steeper upper slope reduces the amount of stones required. The difference is small for the steepest slopes of n = 4/3 and 1.5. The lower slope has less influence on the amount of stones required. Based on Fig.5.18, the lower and upper slopes were chosen for m = n = 1.5. The berm length becomes b = 19 m (Fig.5.17). This choice of berm breakwater dimensions and the profile after design conditions is shown in Fig.4.3.

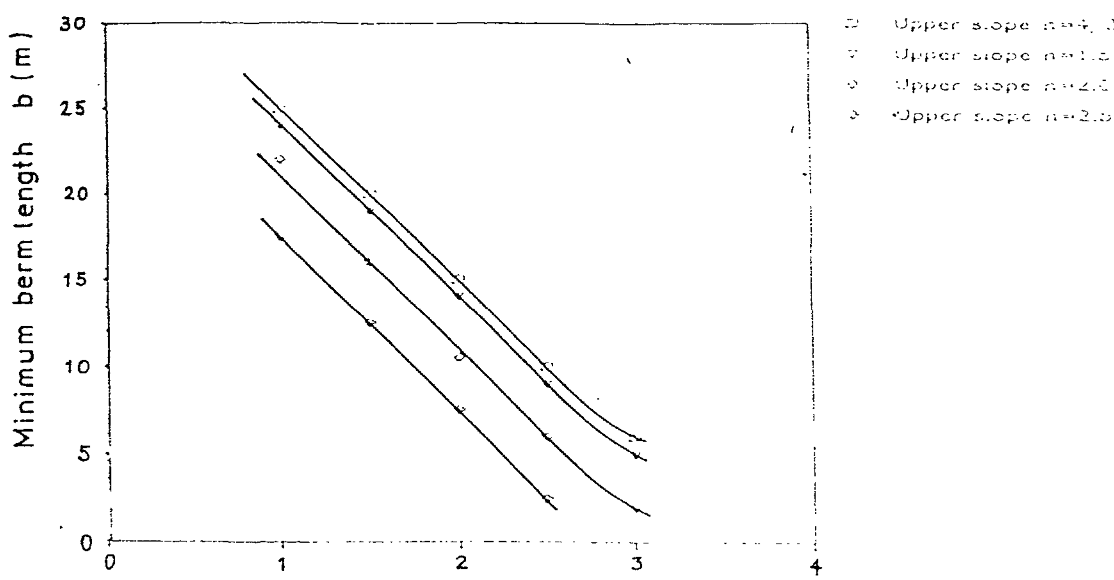


Figure 5.17. Minimum Berm Length as a Function of Lower Slope and Upper Slope, van der Meer (1987).

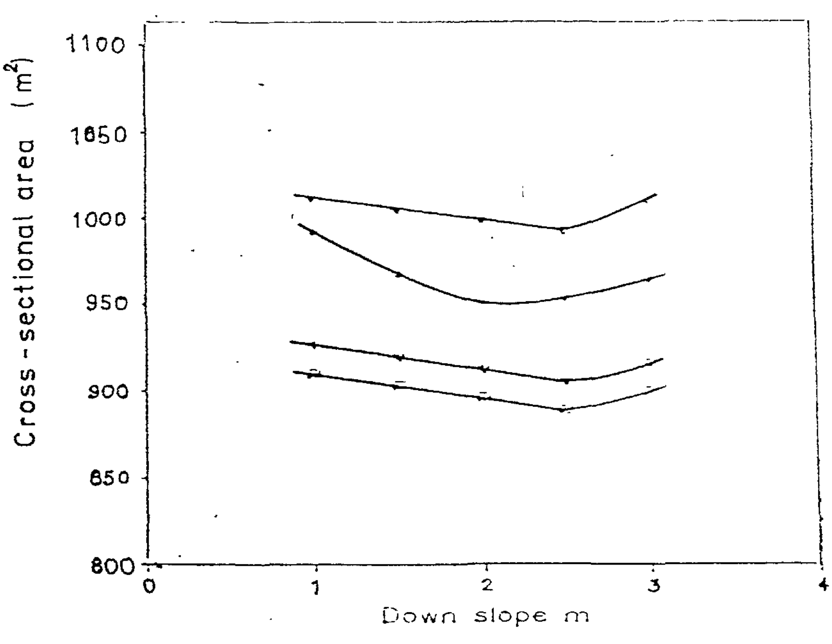


Figure 5.18. Cross-Sectional Area as a Function of Lower Slope and Upper Slope.

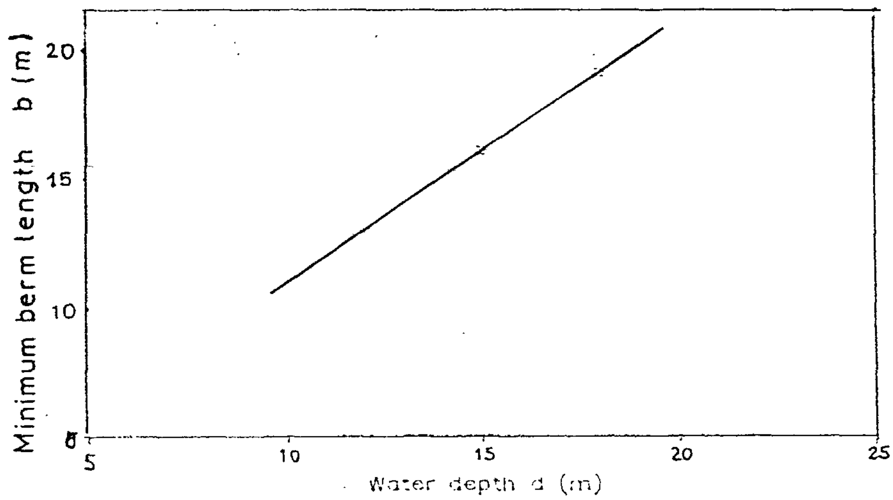


Figure 5.19. Influence of Water Depth on Minimum Berm Length.

Influence of water depth

With the lower and upper slopes fixed at 1:1.5 the berm length becomes 19 m for a water depth of 18 m. The berm length can be reduced in shallower water using the same design conditions. Fig.5.19 shows this reduction of b for shallower water.

Influence of stone class

Until now the wide gradation of 0.5...9 Mg with $D_{n50} = 1.01\ m$ was used. Heavier stone will show less displacement of material. Therefore the profiles under design conditions were computed for the stone classes 1/9 t and 3/9 t, according to output curves of a view quarries. Fig.5.20 shows all three profiles. As the differences in D_{n50} are small, the differences in profile are small too. It can be concluded that the wide (and cheaper) class of 0.5-9 t is satisfactory.

Length and slope of the gentle part of the S-shaped breakwater can be estimated together with the steeper upper and lower slopes. Another application is the prediction of the behaviour of core and filter layers under construction when a storm hits the incomplete part of a breakwater.

5.4 SEA WALLS

Since a variety of construction materials and technologies are being used in a number of design solutions, it seems worthwhile to begin with the examination of the commonest practices recommended for the moderate conditions of Great Lakes (Ministry...,1987).

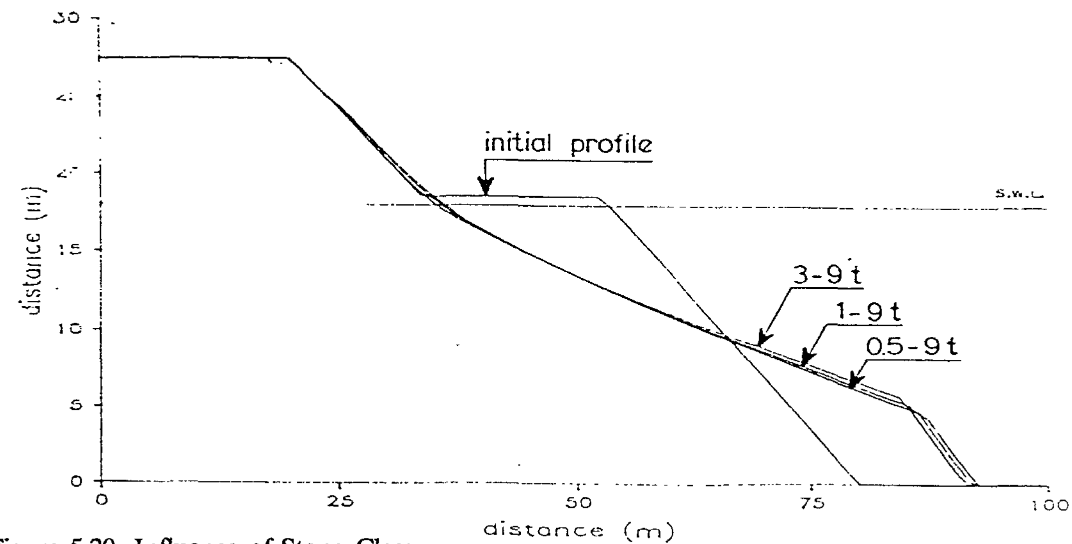


Figure 5.20. Influence of Stone Class.

CONCRETE RUBBLE

This material should only be used for temporary protection in relatively low wave environments. It should not be dumped along with dirt or other loose fill, and should be stacked at a low angle (3:1 or less), to reduce settling. Elongated or flattened pieces should be removed if practical, but the material will still be much less stable than rip rap, due to its tubular form, relatively low density and low friction between units. It is recommended that rubble not be used for core material in rip rap walls, especially if concrete pieces are likely to be placed at the toe of the wall or exposed by minor failure of the armor stone. It also presents a particularly serious aesthetic problem.

Large concrete blocks are not likely to provide any significant protection on sandy beaches during storms, since they not only sink into the sand, but may disperse rapidly. Even very massive blocks (5 to 15 tons) are not recommended for simple placement on top of sand surfaces. They must be rigidly tied together, so that if they settle, they will still present a continuous barrier to the waves. Even so, they will be difficult to stack high enough to provide significant protection from overtopping, and are likely to be outflanked.

RIP RAP

In placing rip rap over bedrock, it is most important to determine the depth to bedrock off and along shore. By identifying low areas in the rock, such as surge channels, and areas where rock platforms slant steeply seaward, the degree of seaward spreading of the revetment can be anticipated. Excavating a keyway in a bedrock surface should be helpful, unless the rock surface is narrow, steep seaward, or eroded by wave action (either when severe storms remove sand, or constantly exposed to wave action and abrasion).

For engineered rip rap on sand, at least 2 to 5 feet to vertical settlement must be planned for in design; for non-engineered rip rap placed on top of sand, as much as 10 to 15 feet of settlement is possible. In general, rip rap should never be dumped directly on top of the beach surface, but this is particularly true in late summer, when sand levels are usually highest. Because it's maintenance costs are so high, non-engineered rip rap may cost are so high, non-engineered rip rap may cost more than engineered rip rap in the long run. In sand, a deeply buried toe (-3 feet MLLW) or large rock (3 to 5 tons) and filter cloth may reduce downward movement of the rock, but may be less successful in preventing toe stones from rotating and/or moving (horizontally) seaward. Finally, the strategy of engineering a toe does not provide a guarantee of overall stability, because even minor rotation and settlement of toe stones may destablize the upper portions of the structure.

For all rip rap walls, both vertical and horizontal settlement should be planned for, either by building the wall higher and deeper than necessary for a design storm, or by preparing for the rapid addition of new rock under storm conditions. The second strategy may allow significant damage to occur behind the wall it settles rapidly and unexpectedly. Because virtually all rip rap walls will settle, they should not be used as foundations for any permanent, rigid structures. Finally. rip rap is much more stable at a 2:1 slope than at the standard 1.5:1, but varying the slope may be secondary in importance to controlling settlement of the stone. Where property boundaries are not a problem, rip rap stacked in mounded, self-supporting walls should require less long-term maintenance than revetments.

Some erosion and structural damage caused by splash overtopping of rip rap revetments against bluffs up to 30 feet above MLLW can be prevented by piling the rip rap up to the top of the bluffs (to protect terrace deposits). In 1983, lots on 30 foot cliffs along northern Monterey Bay which had rip rap piled to the top of the cliff experienced much less erosion of terrace deposits than did similar lots with rock only part way up the bluff. For bluffs less than 15 to 20 feet above sea level, even pilling rip rap above the level of the bluffs may not eliminate erosion, flooding, or damage by overtopping.

Filter cloth does not significantly reduce gullying due to wave due to wave splash and water running o,ver the bluffs from streets and other impermeable surfaces. Where rip rap has settled, or is low to start with, erosion behind the revetment is particularly difficult to prevent. Careful attention to drainage paths, and the use of filter cloth, filter rock, or gunnite may help, but all will be damaged by direct wave attack. In low bluff areas, such as those in San Mateo Country, replacement and repair during and after storms may be the only viable option, short of building walls so high or so wide that overtopping splash will not reach the erodible material behind the walls. Such walls would not only be prohibitively expensive, but would probably cover significant summer beach areas and/or block any view of the ocean from these areas.

CONCRETE SEAWALLS

Concrete walls founded on rock must be deeply embedded in the rock (5 to 10 feet) to provide a safety margin, if the bedrock is erodible or subject to frequent wave action. At every study site, engineered concrete structures proved more resistant to erosion than the bedrock around them, and thus were likely to be undermined or outflanked before significant degradation of the concrete occurred. Rip rap placed at the toe and flanks of such structures may help reduce these problems, but the rip rap itself often settles during storms, exposing the base of the wall.

Concrete walls on sand should incorporate deep (to 5-8 to -15 feet MLLW depending on the height of the wall), interlocking sheet pilings, to eliminate the possibility of sand being scoured out from underneath the toe of the wall. The interlocking should be carefully grouted with some material (flexible, not brittle) to as great a depth as possible (in theory, walls should be grouted to the expected depth of beach scour and/or liquefaction) to prevent piping of sediment from behind the wall. In all cases, the walls should be at least 10 to 12 inches thick, and designed to retain their stability and structural integrity even after major loss of fill. They should also be inspected after every major storm, if possible, to identify any loss of fill before it becomes a major problem.

The potentially large volumes of seawater overtopping concrete walls must be drained carefully. The best approach is to do as O'Shaughnessy did in San Francisco (cf. Fig.3.33): prevent water from seeping in behind the wall by installing a thick, continuous reinforced concrete apron (not clay or asphalt) extending ten to twenty feet landward of the wall, grouted, and underlain by a layer of impervious clay. Also, oversized drainage holes should be incorporated at the seaward edge of this apron to allaow rapid drainage during severe overtopping conditions.

WOODEN SEAWALLS

Wooden seawalls, even when new, are generally less able to resist direct wave forces on an open coast than concrete or rip rap walls. The repeated destruction of piling and timber walls indicates that they should not be used by themselves, especially where large logs and debris are likely to be washed ashore during storms. Also, they should not be expected to maintain their structural rigidity after more than about twenty yrars if that long.

Because they are limited by the strength of their materials, wooden walls at most exposed sites cannot be made high enough to eliminate evertopping. Also, they can seldom be constructed deep enough to resist underminig. Using a concrete apron to prevent overtopping water from seeping into fill behind a wooden wall will be successful only if the apron is very wsell sealed and drained. Alternately, a region 10 to 20 feet inland of the wall may be filled with large filter stone. However, large volumes of water will then be forced to drain out through, around, or beneath the wall, and accommodating this water may be difficult. Another option that can be used in some cases is to direct surface water inland into oversized storm drains, which will not be clogged by sand and debris.

Wooden walls, like other seawalls, should not rely on the fill behind them for stability, because this fill is often washed out by waves, exhuming buried deadman anchors. Wooden walls incorporating horizontal planks on both the landward and seaward sides of their pilings may resist wave forces better than those with only one layer of planks. However,if the supporting pilings themselves do not extend eeply enough, then failure can occur through scour. Timber lagging (four to six inches thick) placed in the sglots of steel H beams which are driven into the sand, fared somewhat better than piling-supported structures, particularly where drainage has been incorporated into the structure. Incorporating a reinforced concrete cap into the wooden wall may also help prevent the removal of planking. Wakefield piling walls appear to resist undermining and loss of fill better that the designs generally used today, but they are even more limited in maximum height, due to earth pressures.

Undermining of the lowest boards on wooden walls can best be reduced by placing rip rap in front of the wall to a level at least even with the top of the wall. This measure will also reduce damage caused by floating debris, if the rip rap does not settle too much. Four to six inch thick lagging should provide greater resistance to debris batterring than do two or three inch thick planks.

AVOIDING DAMAGE TO BUILDINGS AND PAVED AREAS

Buildings: Assuming that for economic and aesthetic reasons, seawalls will continue to be built to allow some splash overtopping, some precautions can be taken to minimize damage caused by this overtopping. Wherever possible, decks, buildings, and all other permanent structures should be set back as far as possible from the crests of seawalls, preferably twenty feet or more (depending, of course, on such factors as the seawall height, wave exposure, and local geology). To protect such structures, a concrete splash wall three to five feet high should be erected behind the main seawall, especially where the top of the main wall is less than 15 to 20 feet above Mean Lower Low Water. Alternatelwy, all or part of the seaward wall of a building may be reinforced and designed as a splash wall. However, this will usually leave weak points at doors and windows, and provides little margin for error if large waves, rocks, or debris may be expected to hit.

Concrete slab foundations on short pilings or poured directly over sand or other loose deposits are most susceptible to serious damage by scour or erosion behind seawalls. Even a small amount of scour or liquefaction under the seaward edge of these foundations can cause them to crack, settle or tilt seaward, destroying and entire house. This took place at a number of locations along the California Coast in 1983. Buildings on deep (well below any maximum scour depth), carefully poured concrete pilings, and those with high floor levels, generally fare better than buildings with low floors and/or wooden pilings. A strong bond between the piling and the floorings is also important to resist vertical wave forces. In general, the further inland on a beach a building is, the less wave damage it will incur, if the factors mentioned above are all equal. However, flooding and damage caused by debris are still possible far indland on a wide, flat beach particularly near stream and lagoon mouths.

Bluff-top houses are seldom damaged directly by wave forces, but are often threatened indirectly by flooding and/or erosion, depending on the height of the bluff. As indicated else-where, a well-maintained seawall (along with periodic bluff fill in some severe cases) may significantly reduce long-term erosion, particularly at one toe of a bluff. Small but strong splash walls can reduce flooding damage in areas with low, easily overtopped bluffs, but should be located as far inland as possible to make them less vulnerable to undermining if cliff retreat occurs. Along high bluffs, unprotected terrace deposits often continue to fail despite the presence of seawalls below, but careful control of surface drainage and (if possible) subsurface drainage can reduce these types of failures. Placing buildings as far inland from the bluffs as possible is always a good idea.

Paved Areas: Asphalt has a relatively low density, and in this slabs, a very high surface area to mass ratio. For this reason, it is easily moved (and removed) by wave action. Because of its flexibility and softness, waves can erode rapidly through cracks in asphalt pavement, exposing the fill beneath. Once fill is exposed to wave attack, asphalt tends to be lifted up in large slabs, and "floated" inland or broken uyp and destroyed. Concrete pavement is also easily undermined, but if anchored to deep pilings, may resist mild wave attack and overwashing.

Rip rap, because of its tendency to settle and its high permeability, generally provides poor protection for fill beneath asphalt or concrete paved surfaces. Where coarse rock is incorporated into roadbed and slope fill, and where the roadbed is relatively high, rip rap may be more successful. However, runoff must be directed away from the seawall, or gullying will erode behind the rip rap. Concrete walls with concrete splash aprons behind them probably provide the most satisfactory protection for paved areas, but surface runoff and subsurface piping may still present persistent problems.

As concluded elsewhere in this document, of the three major of protection, concrete walls generally have been most successful in reducing erosion and property damage, and have been most durable, over the long term. However, concrete walls supported on discrete pillings have required moderate to high maintenance, in the form of rip rap toe protection, to survive. Rip rap walls fared less well than concrete walls, but better than wooden walls. However, their maintenance costs have often been much higher than anticipated, particularly in sandy environments. Wooden walls have proven to be least successful in preventing erosion and damage, and most are easily damaged during severe storms. Wooden walls fronted entirely by rip rap have been more successful, as long as the rip rap does not settle.

On the whole, few protective structures in the Canadian area studied (Ministry..., 1987) have stood the long-terms tests of time, surviving unassisted and preventing damage and erosion, for more than twenty years or longer than their design life. Many structures have become structurally unsound, required considerable maintenance or repair, and/or failed to adequately reduce property damage for more than one severe storm period. Thus, the effective lifetime of a structure often depends on how many mild winters pass before the next severe storm. However, most of the structures have reduced erosion rates, at least over the short term.

Following the severe coastal storms of 1978 to 1983, a large number of protective structuress were designed and emplaced, and many more are being planned and considered. We have presented our observations along the central California coast with a view that these will be of assistance to those who must make decisions about protective works in the futurure. There are a number of options for those with threatened property - -some structural, some non-structural. Before any protective structure is to be built, its initial costs and maintenance costs and its probable lifespan must be carefully considered, as well as its technical merits and limitations.

Some of the above Canadian observations can be regarded as confirmed in good a many oter coastal environments. In general, heavy design of the gravity type sea wall has proved costly but durable. Hence it can be recommended if the precious site is to be defended at any price and despite the adverse impact on the adjacent strips of shore (armouring). Aside from the repeatedly cited case of O'Shaughnessy Sea Wall (Fig.3.33) one can provide the examples on the Polish and German coastlines, Figures 5.21 and 5.22. The history of the sea wall on Sylt island has been described by Dette and Gärtner (1987).

Sheet piling alone, with auxiliary revetment and wave dissipation devices proves an effective design provided adequate corrosion prevention is secured (Figures 5.23 and 5.24).

Sea walls without sheet piling are susceptible to removal of fill material from the lee side; this is usually cused by a combination of different factors, including overtopping, undercutting, percolation, seepage, suffosion, failing drainage, etc. as already discussed at a few places in this document. Hence the type depicted in Fig.5.26 appears much less durable than the former examples. The palisade type of sea walls constructed in Poland in the years 1957...1961 (Fig.5.27) can be regarded temporary measures at the best. Their cost is low but but it does not seem to be returned effectively in terms of dune erosion prevention or alleviation. The design is vulnerable to overflowing and overtopping by waves, The following **seawall-oriented** conclusions are drawn by Bijker and Graaff (1989):

- Demolished seawalls harm the prestige of "coastal engineering" as a respectable profession

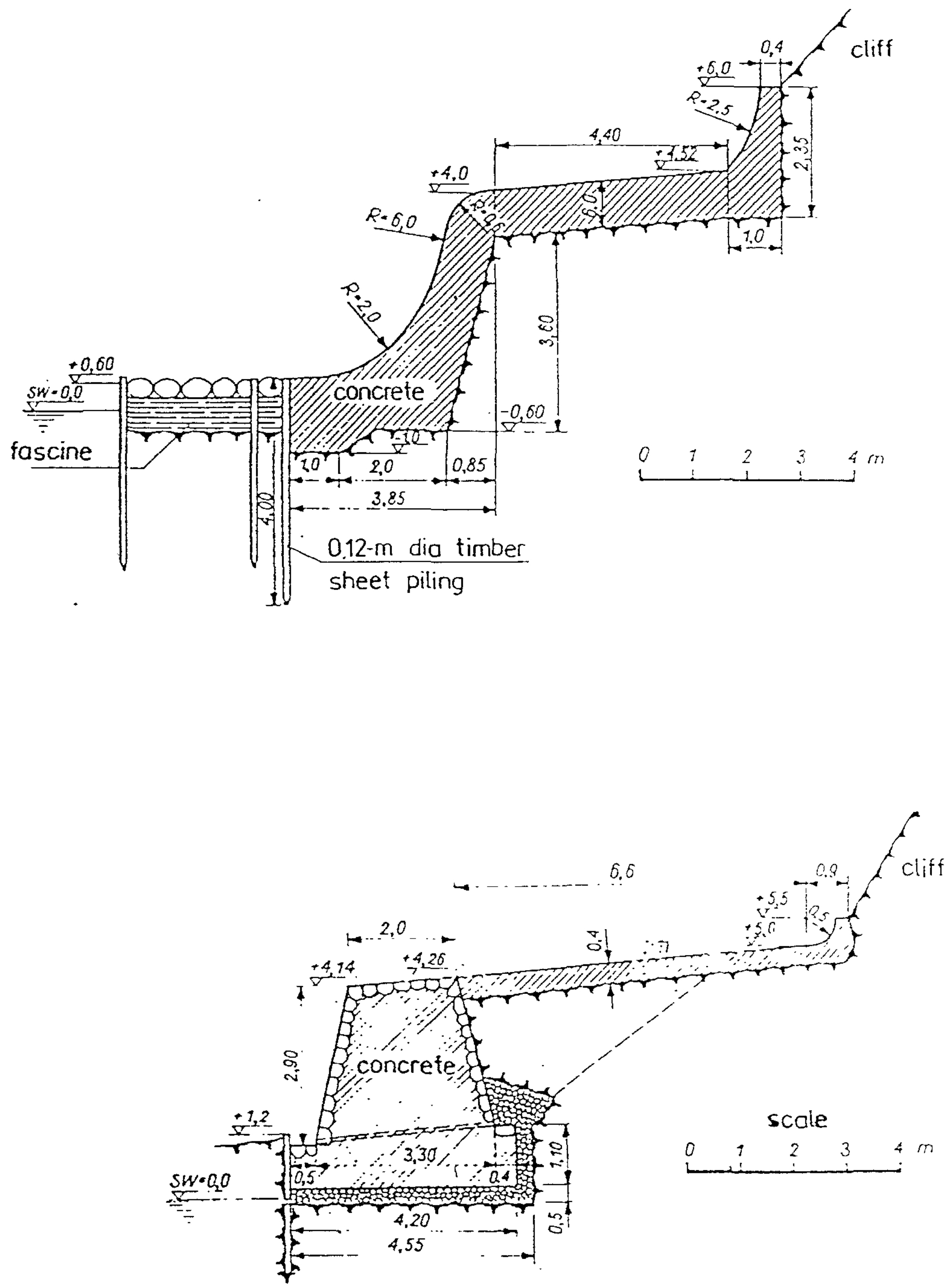
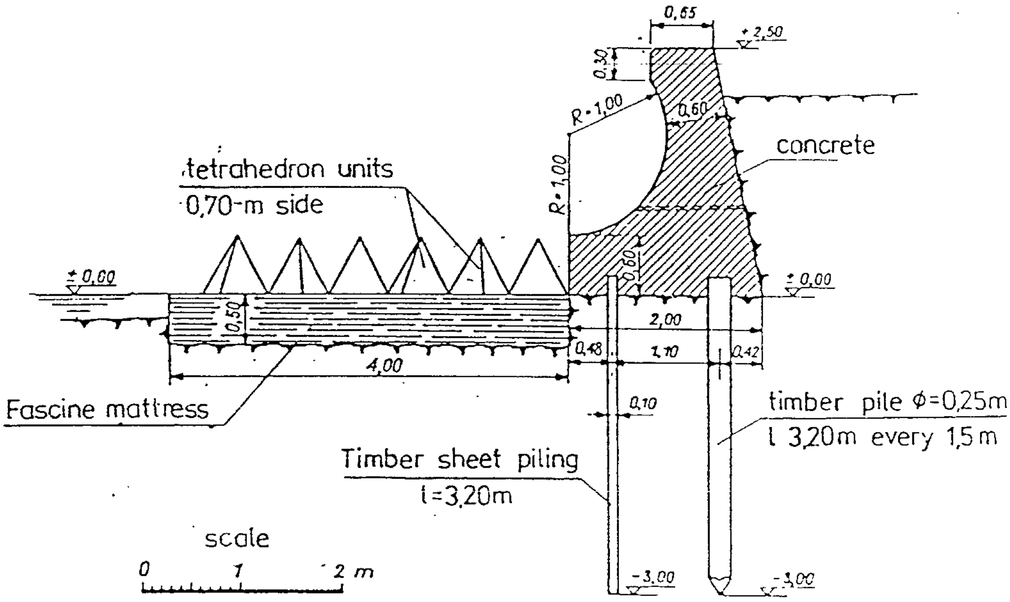


Figure 5.21. Heavy Concrete Sea Wall at Niechorze (Poland):
Type I, constructed in the years 1914...1915 (top)
Type II, equally durable (persisted over decades)(bottom).



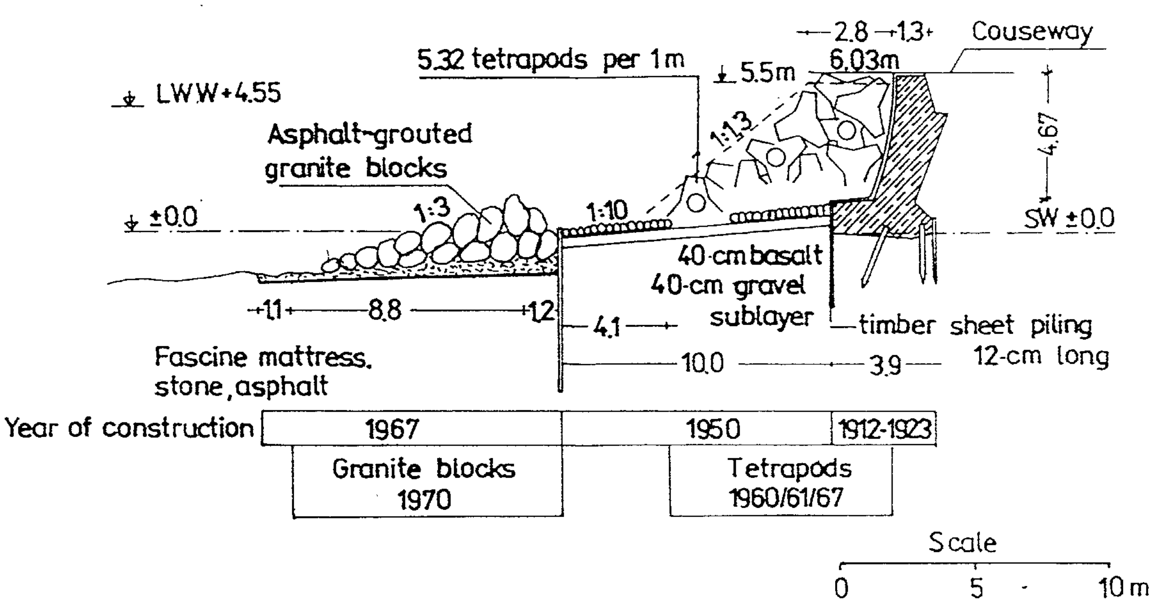


Figure 5.22. Gravity Type Sea Wall on Piling at Wladyslawowo, Poland, constructed in 1937 and 1952 (top) and Mixed Design Sea Wall on Sylt Island (bottom).

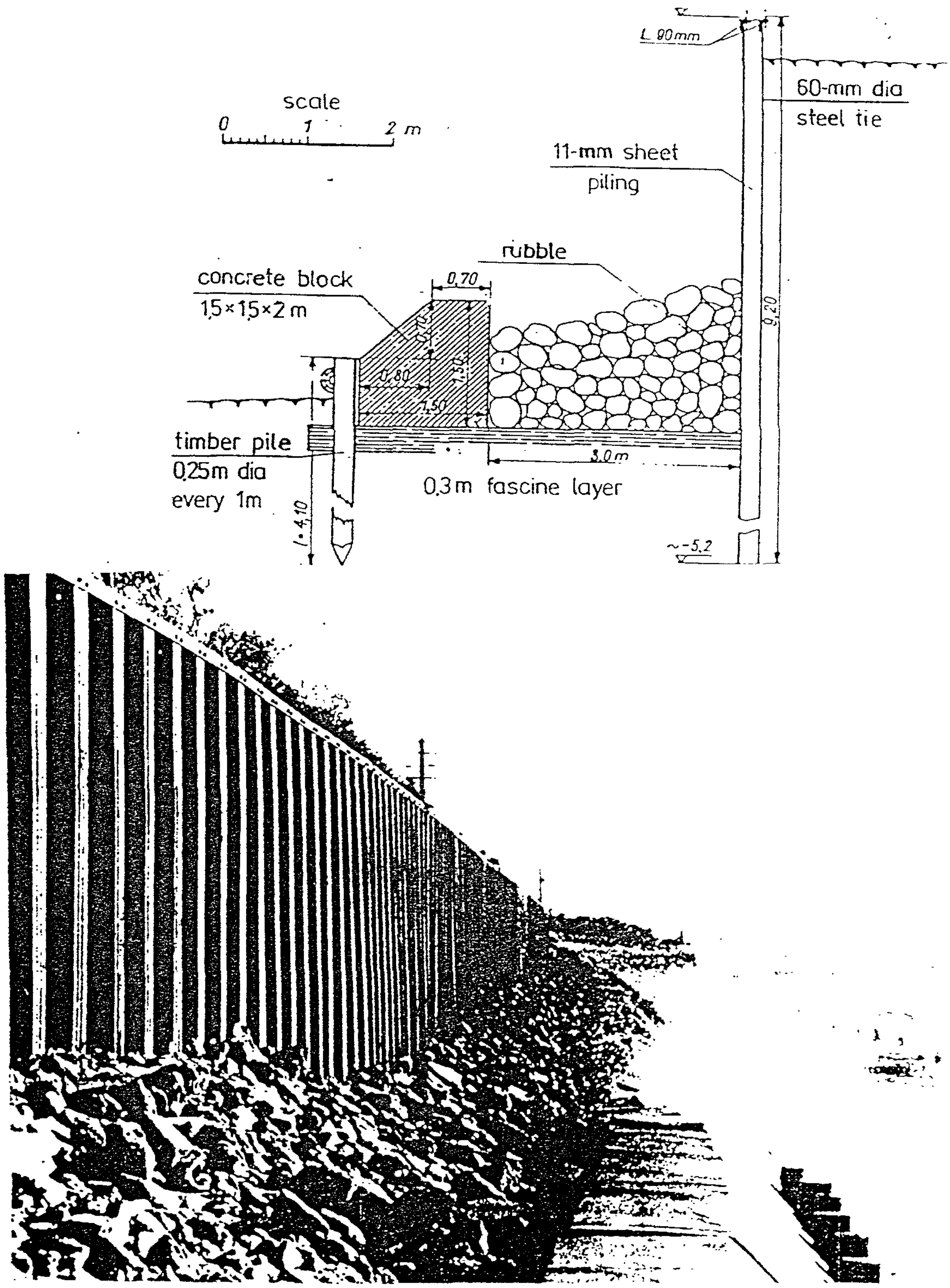


Figure 5.23. Steel Sheet Piling Wall at Dziwnów, constructed in 1925 (wall) and 1954 (blocks).

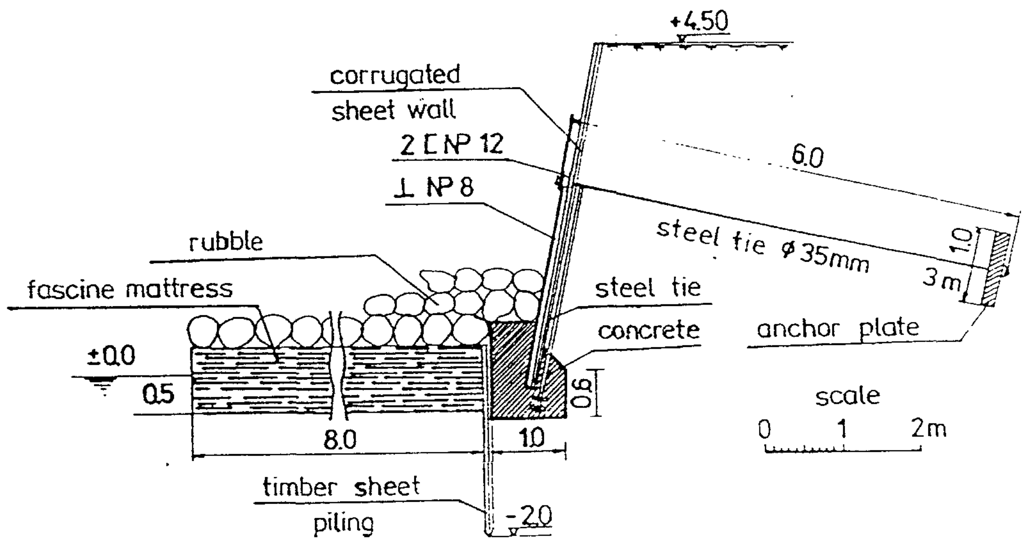


Figure 5.24. Corrugated Sheet Piling at Westerplatte (Gdańsk), constructed in 1914...1915.

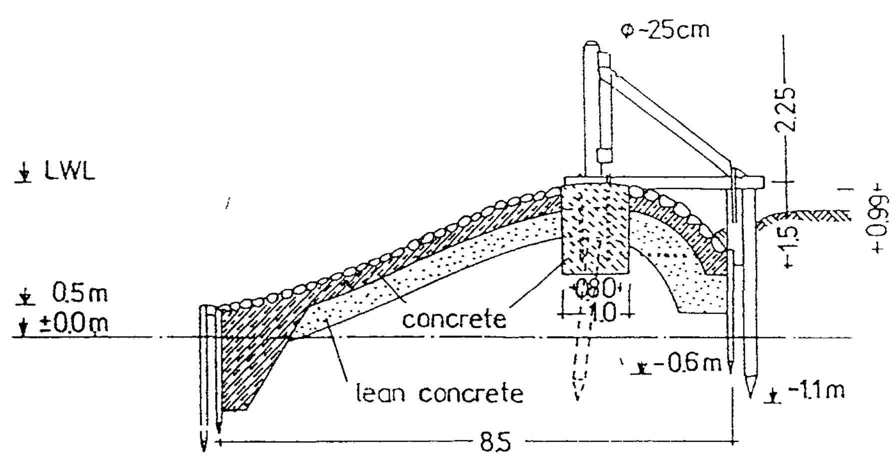


Figure 5.25. Palisade Sea Wall and Revetment on Baltrum Island, constructed 1883 and reconstructed in 1931.

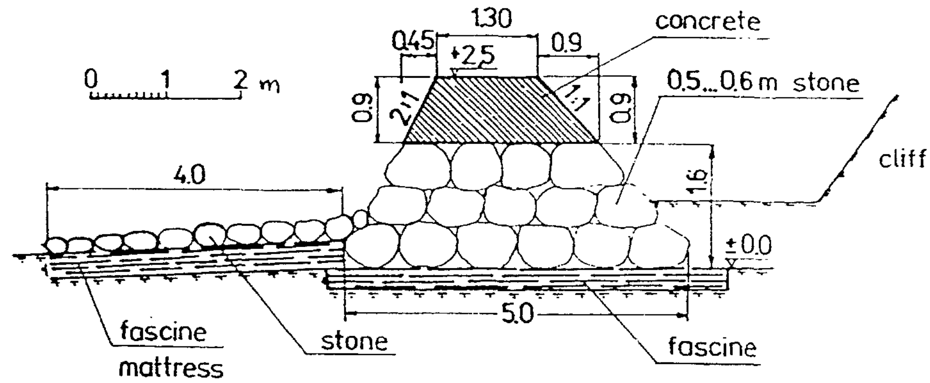


Figure 5.26. Stone-Concrete Sea Wall at Rozewie, Poland, constructed in 1910.

- The application of seawalls as a means to combat erosion of coasts should be restricted to a very limited number of cases
- Only in essentially stable cases where some reduction of the so-called "playground" of the sea is required, a seawall would be an appropriate solution to achieve that goal
- Along gradually eroding coasts, seawalls should not be used to prevent further erosion. A seawall does not work in these case since a seawall does not interfere in the sediment transport processes.

CASE STUDY

Will et al. (1985) describe the developments of design criteria and design studies for the sea wall and breakwater provided at the power station at Torness in East Lothian, under construction for the South of Scotland Electricity Board. The preliminary design studies are outlined, and the design finally adopted is described in detail, together with the methods for controlling and supervising site construction.

Torness power station, situated on the east coast of Scotland approximately 7 km south-east of Dunbar in East Lothian District, is currently under construction for the South of Scotland Electricity Board (SSEB).The layout is depicted in Fig.5.28. The station, which is due to be commissioned in 1987-88, is a 2x660 MW(e) nuclear station of the advanced gas-cooled reactor (AGR) type. Main construction work was preceded by a preliminary civil engineering works (PCEW) contract which included a dolos-armoured sea wall and breakwater, cofferdams for later construction of cooling water works, excavation for main foundations, roads and services for temporary and some permanent works, and landforms and screen embankment.

Some 22 ha have been reclaimed by the construction of a 1500 m long sea wall and infilling with soil from excavations. The sea wall also forms part of the station's flood defences.

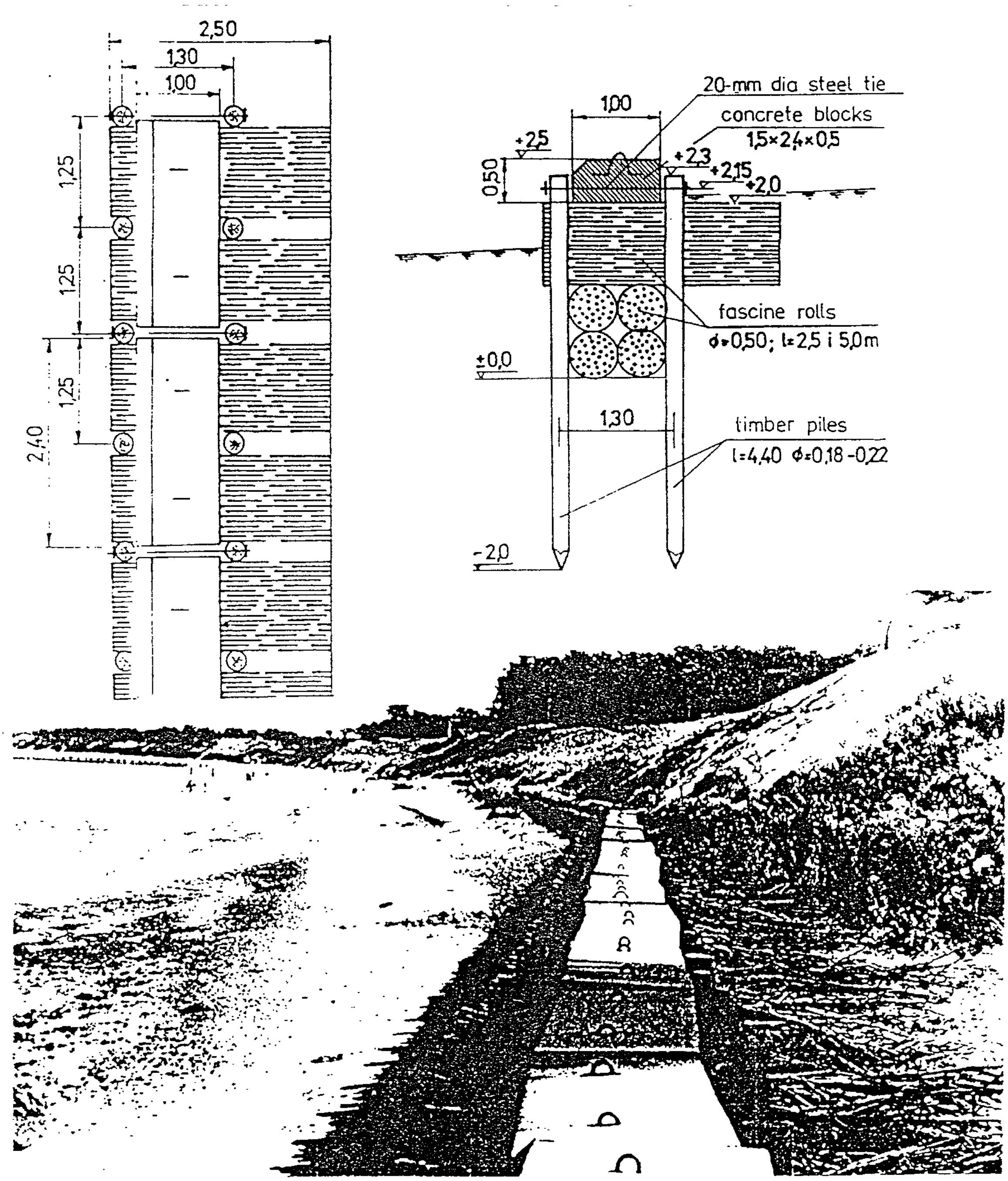


Figure 5.27. Double-Row Palisade-Type Sea Wall at Ustka, Poland.

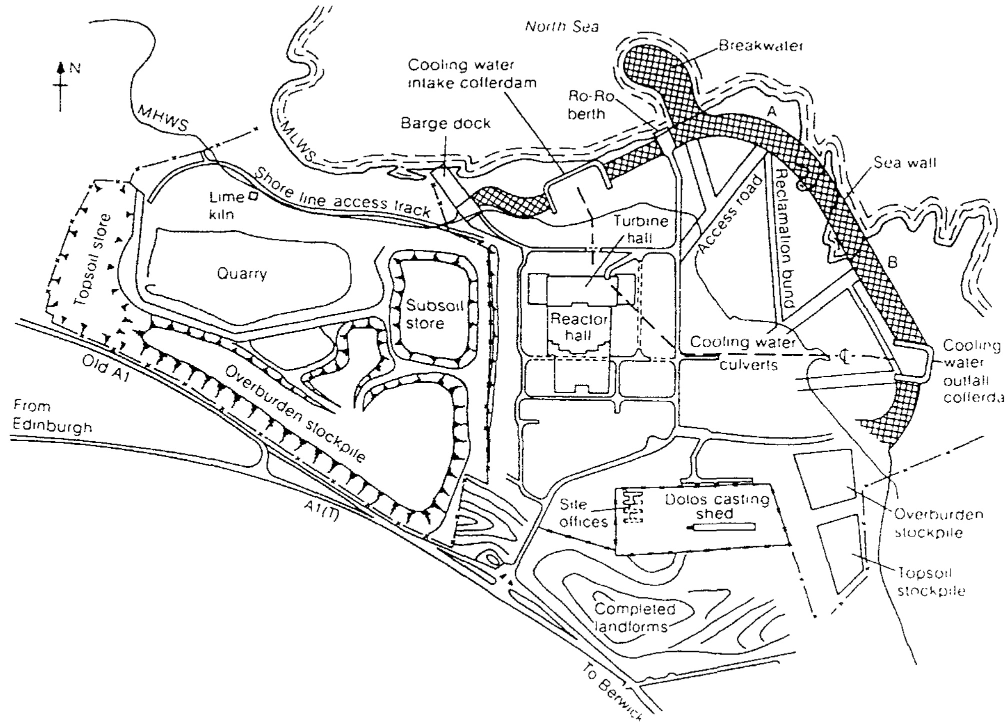


Figure 5.28. Site Layout of Torness Project, Will et al.(1985).

A study of the concidence of abnormally high tides in combination with high on-shore waves was undertaken for probabilities of occurrence in the range $2x10^{-2}$ to 10^{-4}/year, the latter being the minimum design provision for naturally occuring hazards in the current United Kingdom reactor programme.

It is not practicable to design sea defences for zero overtopping for probabilities as low as $10^{-4}/year$. In deciding the level of the crest of the sea wall, a balance had to be achieved between visual intrusion, cost, programme, operational and safety considerations. Consequently the sea wall has been designed to accommodate some overtopping along its length, and the level eventually adopted was designed to limit overtopping to about 1-2 % of waves at a frequency of $2x10^{-2}/year$. Provision therefore had to be made to drain overtopping water in order to limit maximum flood levels resulting from a very low-frequency storm. The derivation of crest levels, sea wall design and the drainage provisions are discussed by Will et al.(1985).

In the absence of any long-term wave records for the Torness area, wave heights were predicted from wind records.

Before selecting the sea wall type and profile, a comparative study was carried out of various alternative designs. Amongst those considered were double-skin steel sheet pile walls, concrete pile walls, caissons and a variery of shapes of rubble mound, armoured either with natural rock or precast concrete units.

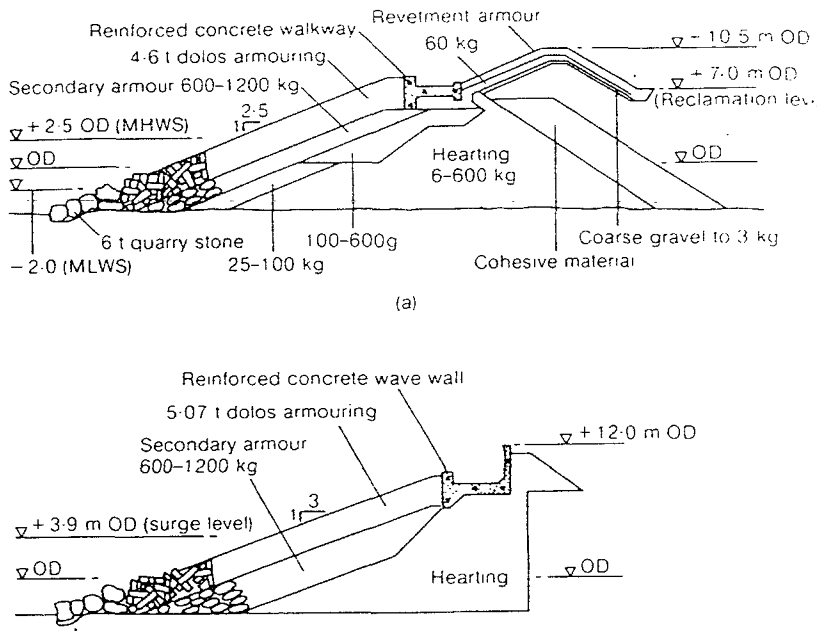


Figure 5.29. Torness Sea Wall Profile Tested at HRS: original (top) and final (bottom); Will et al.(1985).

After a comparative study of costs, ease of construction and programme aspects, the sea wall type illustrated in Fig.5.29 was selected for more-detailed design studies. The basic sea-wall designs were subjected to two-dimensional model tests to examine the stability of the wall and its armour, and the overtopping performance of the wall under various wave climates.

The tests, which included long-term test designed to demonstrate the stability of the armour after repeated exposure to storms, confirmed that dolos armour units weighting 5.4 t would be stable for all of the wave conditions tested.

Overtopping rates at several points along the sea wall were calculated by combining the results of the physical and numerical models, and by a simple integration process the total volumetric overtopping was estimated for a range of crest levels.

The crest levels and the extent of drainage provisions were optimized on the basis of costs, programme constraints and safety requirements, having regard to the views expressed by local authority and amenity bodies during the process of consultation. For the section of sea wall between the cooling water outfall and the breakwater, a general crest level of $+10.0mOD$ was selected and elsewhere a general level of $+9.0mOD$ was adopted.

Overtopping and drainage provisions

When storms cause sufficient overtopping of the sea wall, a 10 m wide drainage ditch at the toe of the landward slope will fill and discharge at the roll-on/roll-off berth and at the cooling water outfall. For very infrequent and extreme overtopping conditions the drainage ditch will overflow on to a reserve flood area to an estimated maximum depth of 1.7 m for the $10^{-4}/year$ design condition. In this event the maximum water level will be approximately $+7.7mOD$ and the duration of flooding about 6 h. As the storm subsides the flood water will drain by way of the ditch and the outfalls back to the sea.

Dolos armouring

The doloses were designed for a seaward slope of 1 in 3, which is well within the range of slopes usually regarded as acceptable for these units. The comparative shallowness of the slope plays an important part in controlling the amount of wave overtopping. The toe of the armour slope was bedded into a toe trench cut into the sea-bed rock, increasing the stability of the slope.

The dolos units were designed to weigh 5.4 t, have an overall length of 2.32 m and a stem-to-length ratio of 0.35 . In practice the density of site concrete was higher than assumed, an the units have an average weight of around 5.5 t. The dolos units are unreinforced.

To maintain their interlocking properties, it is important that the units are placed on the wall at a uniform density but with part random orientation. In keeping with practices developed elsewhere the units were laid in two layers, there being aproximately 55 % of the required number in the lower layer and 45 % in the upper. This represents 25.5 and 20.8 units per 100 m^2 in the respective layers.

Previous experience also suggests that about 60 % of the units in the lower layer should be placed with one fluke facing seawards and approximately vertical. In practice generous tolerances were allowed on these requirements. Orientation of upper layer units was entirely random.

Rock armouring

At the ends of the sea wall where the water depth is shallow and the wall is more sheltered, the primary armour consists of 6 t limestone blocks. At the south end, where the sea wall joints the existing shore, the primary armour is progressively reduced in size from 6 t to 600 kg and the slope is flattened and broadened to form a transition to the natural sand dune formations. This transition is blending satisfactorily with the original beach as a result of natural littoral processes.

Secondary armour

Secondary armour is provided under both dolosse and rock primary armouring, and consists of a double layer of limestone blocks in the size range $600 - 1200kg$.

Hearting

The core of the sea wall is formed from graded stone nominally in the range 5-600 kg. Initially this was produced by screening quarried rock to remove fines, after primary and secondary armour sizes had been extracted. However, experi-

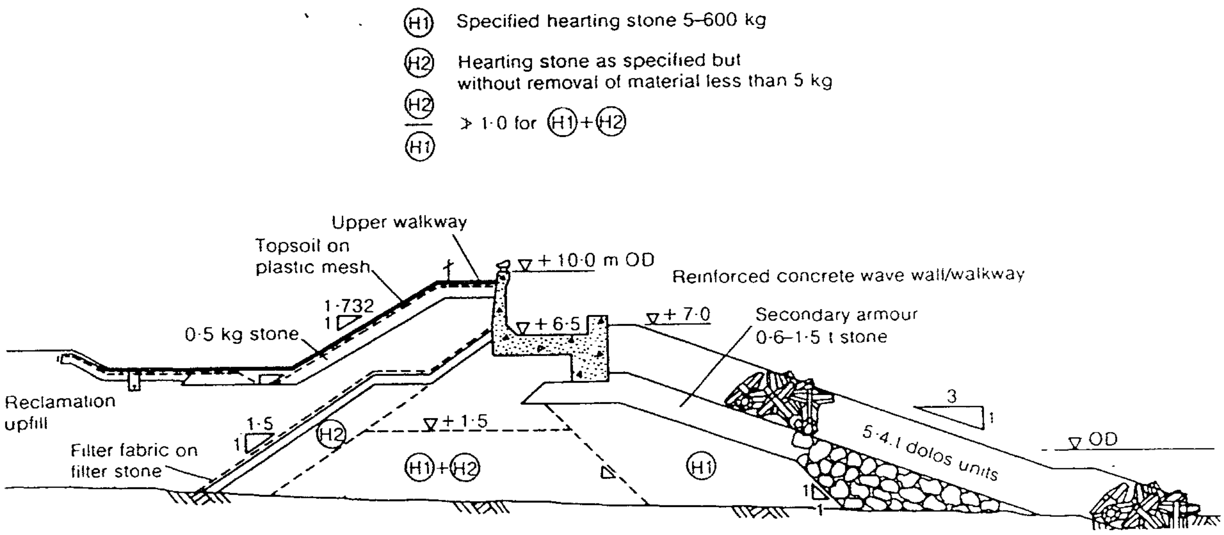


Figure 5.30. Torness Sea Wall Profile as Built, Will et al.(1985).

ence showed that fine material amounted to a significant proportion of the quarry production, and so core construction was altered to enable the landward part to be constructed from a mixture of specified hearting and as-dug material, i.e. heating which had not been screened. This amendment gave rise to a more complicated cross-section (see Fig.5.30) and construction sequence but usage of rock was much improved. The effect on performance of the sea wall will be minimal.

To prevent loss of material from the reclaimed areas by migration through the sea wall, a Lotrak 16/15 filter membrane was laid on a regulating layer of gravel at the back of the heating core.

As well as the combined wave wall/lower walkway, which gives access for light maintenance vehicles along the full length of the wall, an upper pedestrian-only walkway was incorporated.

Relatively small rates of overtopping have been shown to scour the landward face of sea defences. To resist such erosion, the landward embankment is faced with a 750 mm thick stone layer anchored with mesh, and landscaped with grass on a layer of topsoil, the protection extending into the drainage ditch.

The weights of some prefabricated reactor and turbo alternator components precluded their transport by road or rail, and these abnormally heavy loads have been brought to site by sea. The facilities for their handling comprise a roll-on/roll-off berth and a barge landing facility, protected by a breakwater 170 m long. The breakwater does not form part of the primary sea defences and, in consequence, has been designed for less severe conditions of wave exposure than the sea wall itself. Design conditions are approximating to a 100-year storm, and a typical cross-section is shown in Fig.5.31.

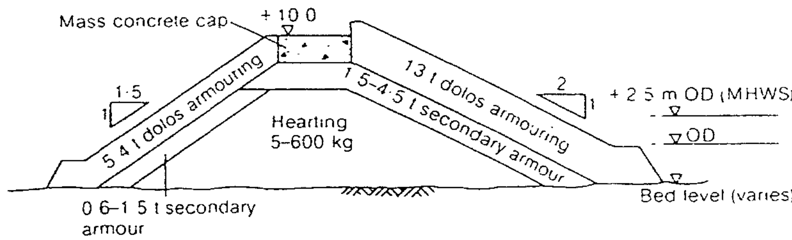


Figure 5.31. Torness Breakwater of Typical Cross-Section, Will et al.(1985).

The intake and outfall structures for the cooling water system are located in the sea wall.

5.5 FLEXIBLE REVETMENTS

General Requirements

Revetment is a slope protection measure designed to protect and stabilize a slope that may be subject to action by water movement in various types of currents and waves. To fulfill this function, the following aspects have to be considered in the design procedure:

(a) overall and partial stability (of structure and its constituents, i.e. toplayer, sublayer, subsoil)
(b) flexibility (i.e. gradual damage without sudden loss of major properties)
(c) durability (of all materials and constituents)
(d) overall safety (primary and secondary defence, geometry of foreshore, etc.)
(e) possibility of inspection of all constituents (monitoring of damage)
(f) easy placement, repair and maintenance
(g) low cost (construction vs. maintenance)
(h) additional functional requirements, i.e. transitions to and interaction with adjacent structures, special measures for reduction of run-up, access roads, berm requirements, etc.

The best revetment is one which combines all these functions.

As noted throughout this document, one distinguish the primary constituents of toplayer, intermediate layer(s) (sublayer), and subsoil. Units of the toplayer are dealt with extensively in Section 4.2, and partly in Section 4.3. The structural design requirements imposed on the sublayer are identified in Section 4.2. The subsoil is discussed most often in connection with sublayer problems.Some important guidelines concerning the subsoil are added at the end of this section.

Hence adequate rules of structural design are given in the sections cited. Therefore we are not providing too many examples in this section. The reader is referred to the pertinent in-depth document on flexible revetments to appear in 1992 under PIANC auspices (PTC II, Working Group 21).

Dimensioning (Pilarczyk 1987)

Given below are a few general guidelines which add to those provided in the earlier sections.

For revetments it is essential to distinguish the nature of the attack and the duration of time; short loading times and loading with a long cycle time. To the first category belongs the wave attack caused by waves; to the second category belongs the variation of the water level caused by tidal and seasonal effects, which can induce groundwater flows. The wave load is of importance for all types of revetments, while the second type of load (slow variation in groundwater flow and in phreatic-line) is of primary importance for impermeable revetments only.[3].

The following factors can be controlled

- A high degree of compaction can, among others, avert the liquefaction of a saturated or almost saturated soil by impact loads, for example wave attack. A relative proctor density degree of 95 – 100%, down to a depth of about 2 m, can in sand reduce the possibility of liquefaction, in general, to an acceptable minimum

- The permeability of the sand bed is important in connection with groundwater flow in the dike body and, therefore, the occurring of uplift pressures under a relatively watertight revetments and the softening of the sand body

- The compaction by vibration in a loosely packed saturated sand body can cause liquefaction. The dry placing of an open asphalt mix on a saturated sand bed through the influx of water will result in the early development of stripping. Under impermeable mixes, as asphalt concrete, uplift pressures can develop while the asphalt is still soft when placed on a hydraulically filled sand bed. To obtain a good compaction the sand body can be built up in thin layers using bulldozers for compacting and for profilling of the dike face. It is also possible to dump an excess of material, and then, after this has been compacted (for example by a vibration roller) to make the required profile

- After construction the dike body will tend to settle. It is has not been well compacted or if there are clay of peat layers in the subsoil, the settlement can be large and irregular. If the bed is at the same time badly permeable

[3]The changes in the pressure due to the variation of phreatic- -line can be determined using an electric analogue or finite element computations. The uplift pressure due to wave attack has to be determined mostly by model (or prototype) tests for each revetment type under consideration. For permeable revetments on permeable sublayer the mathematical model "Steenzet", as developed in the Netherlands, can be used

then it is possible that the grain stress only recovers slowly and that the bearing capacity of the bed temporarily appears to be insufficient. This effect must certainly be taken into account with clayish subsoils; good drainage in this case is essential. With very permeable materials the situation does not develop

- For placement of block revetments on clay subsoil (or sublayer), besides the requirement of right composition and homogeneity, the proper compaction and smooth surface (blocks placed as close as possible to the clay surface) are of primary importance In the case of "poor clay" (concerning composition or surface preparation) it should be recommended to protect the clay surface by a geotextile
- The use of relatively open top layers directly on sand body (with geotextile in between) is restricted to wave height of $H_s = 1.2m$. The good compaction of sand is essential to avoid sliding or even liquefaction. For loads higher than $H_s = 1.2m$ a well graded layer of stone on a geotextile is recommendable (e.g. layer $0.2...0.3$ m for $1.2m < H_s < 2.5m$).

As noted in Sections 3.2 and 4.2., the **sublayer**, which can consist of
- granular materials;
- sheet-type materials (like all kinds of geotextiles) and
- cohesive sublayers

should be designed with proper consideration of the following possible major types of instability and failure:
-Erosion of granular filter material through the holes in the toplayer
- Internal instability of a granular layer
- Erosion of a granular base layer through a geotextile.

No universal rules are available. Hence sound engineering judgement is of utmost importance. The observations provided in pertaining parts of this document should prove useful in working out an appropriate design scheme.

Examples

The slope revetment types suggested in Germany are depicted in Fig.5.32. The design makes provisions not only for the revetment proper but also for adequate **toe protection.** Two German revetment structures constructed are illustrated in Fig.5.33.

A few examples of revetments constructed on the Danish North Sea coastline are shown in Fig.5.34. The design waves have been taken about $H_b \approx 3\ m$ for about 40-yr return period, and about $T \approx 9...11\ s$. No serious damage has been encountered despite heavy loading caused by storms in November 1981 and November 1985.

Toe Scour is prevented by the measures depicted in the above drawings. To generalize it a bit, given in Fig.5.35 are different scour prevention measures

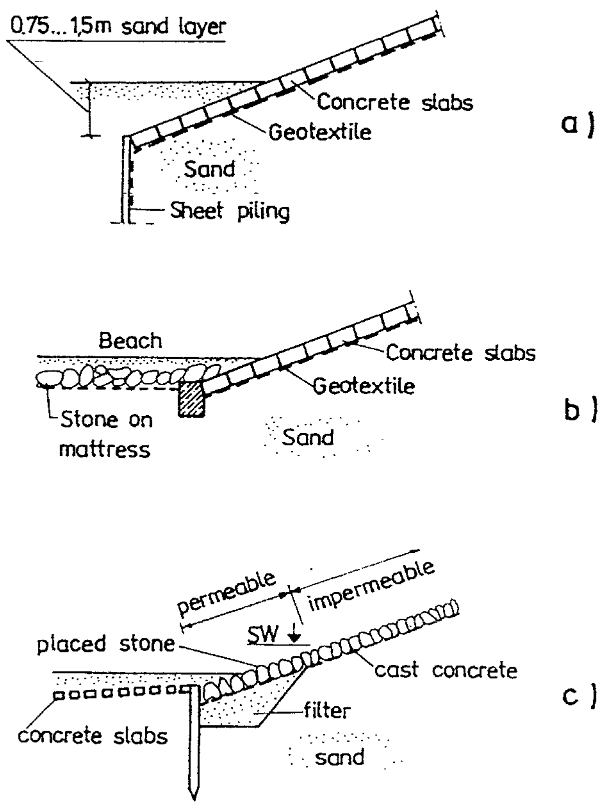


Figure 5.32. Slope Revetment Design Recommended in Germany (Empfehlungen...1981).

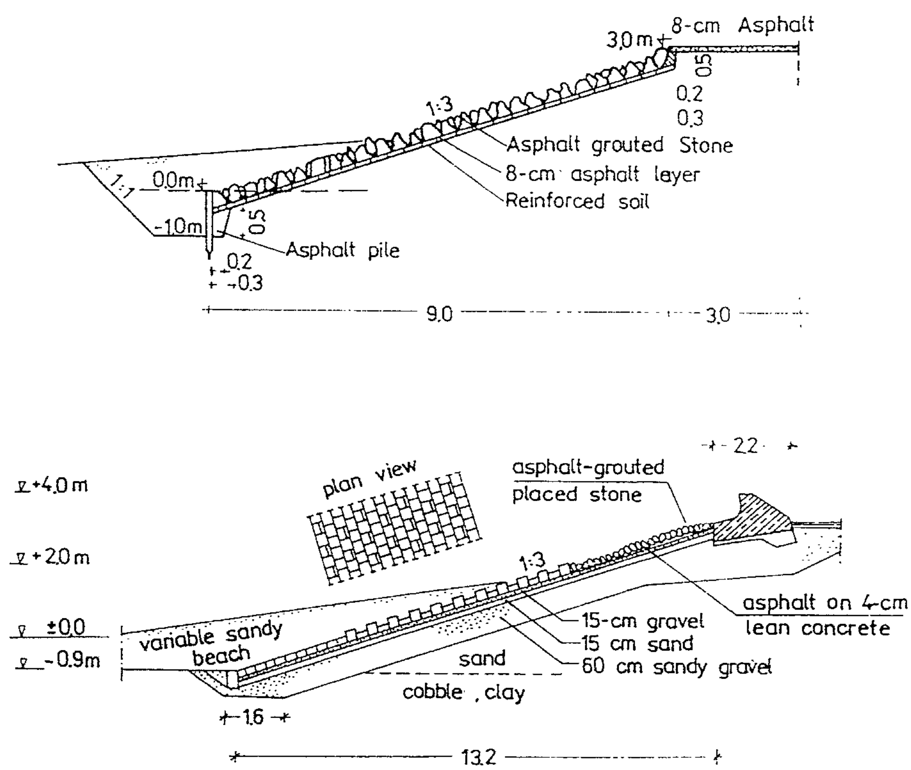


Figure 5.33. Stone Revetment at Heiligenhafen, constructed in 1971 (top) and Placed-Slab and Rubble Revetment at Travemünde, constructed in 1962...1963 (bottom).

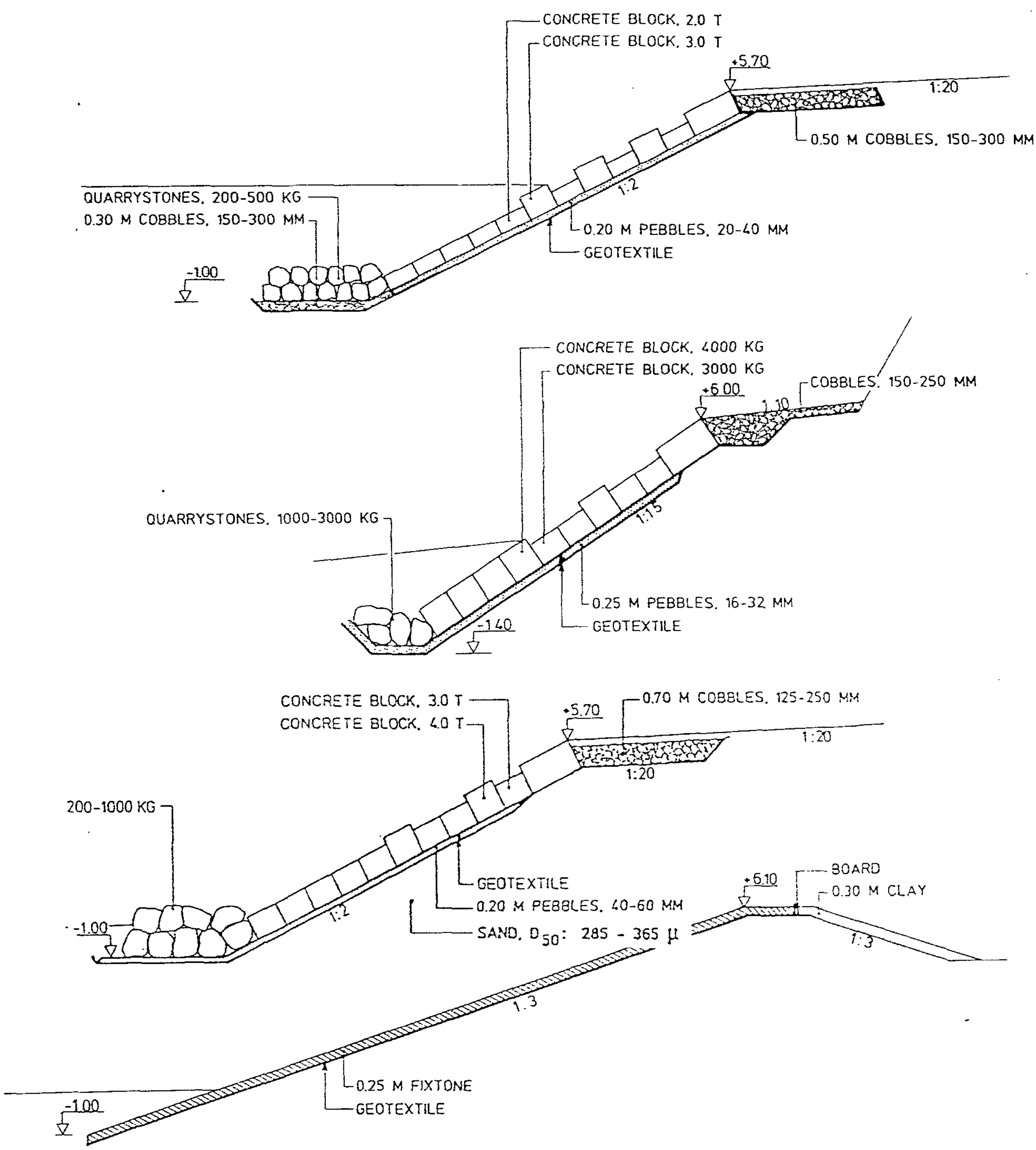


Figure 5.34. Revetment Design in Denmark (courtesy K.Laustrup, Danish Coast Authority)
Placed Concrete Blocks at several locations, constructed 1987...1988 (top)
Placed Concrete Blocks at Fjaltring, constructed in 1982 (upper centre)
Placed Concrete Blocks at several locations, constructed 1982...1986 (lower centre)
Fixtone at Fjaltring, constructed in 1979 (bottom).

designed in Italy at the toe of various coastal structures, mostly breakwaters. *Subsoil* and *geotextiles*, important constituents of coastal structures in general, and revetments in particular, deserve at least a few additional comments[4].

SUBSOIL

Instability (erosion) of sublayers and/or subsoil can lead to failure of a toplayer. The stability of toplayers and sublayers must therefore be designed proportionally (with an equal opportunity of failure).

Good adjustment of the permeabilities of the toplayer and sublayers (including geotextiles) is an essential condition of adequate design.

Subsoils in dykes and/or bank body play an important role in the stability of revetments and in the total stability of protective structures. Thus, the type and state of subsoil or dike body can be decisive for the choice of the revetment type. In this respect, in addition to the factors listed under '*Dimensioning*' a few pages earlier, the following aspects are important:

- The bearing capacity of the dike/bank body determines *inter alia* the performance of a revetment under wave and current attack. If the bearing capacity is large then the thickness of the revetment can be reduced (especially for asphalt revetments). Of importance are the properties of soil such as the modulus of elasticity, the bedding constant and Poisson's ratio. They themselves are influenced by the degree of compaction.

Some comments on GEOTEXTILES

The most striking development in the design and construction of coastal structures over recent years, has been the application of geotextile (or as it is locally called "filter cloth") in a variety of ways. Filter cloth is now available in a wide range of patterns, textures and forms - the choice is already becoming bewildering. Its applications are equally diverse. Its most common use is as a replacement for tertiary armour in rubble walls, but it is also widely used beneath rubble structures and toe mattresses, in an effort to inhibit settlement of the rubble. Sand filled groynes have been built with the material and it is apparently commonly regarded as an ideal structural barrier, between any fine sediment and any larger sized armour. The coastal literature abounds with descriptions of all the different types of these textiles, together with many suggestions as to where they could be applied. Much is made about what the facbrics should be able to do, but little is available about what they actually do do, and how they finally behave in service on the prototype.

There are already probably over half a dozen or so boulder walls constructed on the Gold Coast (Australia), where filter cloth has been applied as a direct replacement for the traditional clay-shale soil filter layer. Unfortunately none of these walls have

[4]Geotextiles are also presented at the end of Chapter Six, as they qualify as an intermediary between the traditional and unconventional measures

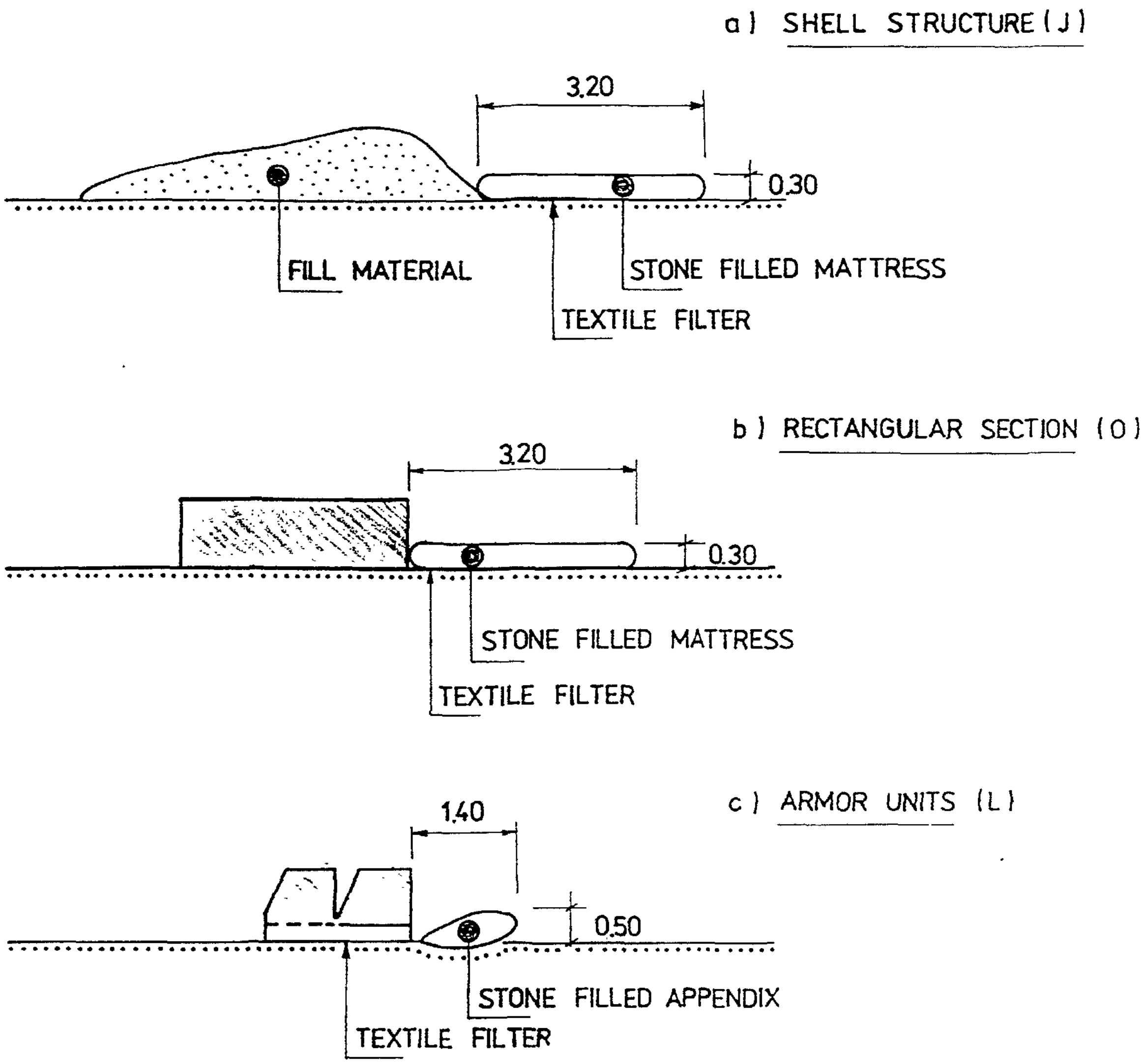


Figure 5.35. Scour Prevention Measures Using PCV Fill, Adopted by Aminti et al.(1983).

yet been exposed to any real wave energy input, so they all remain quite untested to date (Smith, 1989). If we want "hard" site data on the behaviour of rubble walls, backed with filter cloth in service, then we have to turn to California for our testing ground, where Fulton-Bennett & Griggs (1986), under storm conditions, observed that filter cloth underlays beneath rubble walls did not prevent significant wall settlement nor did they remain stable as porous backing membranes, behind secondary armour. These are disquietening observations; perhaps we should reconsider what we expect from these geotextile materials in coastal works.

The classical approach is to consider filter cloth as a porous layer behind the rubble wall, that will allow ground water from behind the material to flow through it, but at the same time, prevent the background fine matgerials (e.g. sands and silts) from doing the same. The intent in this, seems to be twofold. Firstly, high hydraulic pressures behind the structure, will be avoided, and secondly, loss of background fines by piping or flow into the structure, will be prevented. Following the same reasoning, filter cloth is often laid **under** the base and toe of the primary and secondary armour, to prevent wall settlement - settlement in this sense, being attributed to sediments piping "up" through the larger armour units. In both locations, filter-cloth should be (in theory) highly effective and much more efficient than traditional natural granular filters. Perhaps however, we are seeing Hardin's Law in action, i.e. "You cannot only do one thing". We might thus suspect that geotextile filter cloth, as it is usually applied in rubble walls is actually generating unexpected 'side-effects'.

To comprehend the Californian experience, we first look at their filter cloth backed rubble wall failure scenario, as shown by Fulton-Bennett & Griggs. The most striking feature of the diagram, is the manner in which nearly half the main armour - and perhaps a quarter of the secondary armour - has been transported well seawards. The second significant feature, is how the armour that remained, has adopted a clear concave-up surface profile. This is very different from normal Gold Coast boulder wall failure mode, where the armour tends to settle vertically around the toe, and very few units are rolled far seawards. Now a great deal of power is required to roll armour seawards along a very flat slope, **against** the incoming wave energy. Smith (1989) deduces therefore that there can be only one agency capable of doing this, and that is wave reflection.

Smith (1989) compares the type of wall shown in his Fig. 86.1 with the standard Gold Coast wall, as shown in Fig. 86.2. The comparative feature of the Gold Coast wall, is the thick filter layer, constructed from natural clay-shale material. This layer in service, is actually almost completely impervious, it relies on its own weight to resist the groundwater loadings from behind it. When a new Gold Coast wall is initially exposed to wave action, the first thing that is seen is that the water in front of it, turns yellow. The waves that penetrate the armour, wash the clay from around the shale gravel in the "filter" layer. This provides a naturally and progressively graded layer of tertiary armour, along the leading face of the clay-shale layer and the larger the incident wave, the thicker this layer becomes. Up to some limit, the wall becomes "tempered"by wave exposure. By comparison, filter cloth has fixed hydraulic properties, and although it may be fully porous to groundwater seepage, it is completely opaque to waves.

On the Gold Coast, all the boulder walls are constructed against a background of pure sand. In California however, it appears that many are built against a background of a cliff face, and such a face would very efficiently reflect wave power, whether the filter cloth was there or not. We may now begin to appreciate why rubble walls in California tend to be much less effective than their counterparts on the Gold Coast. We would deduce that the rubble walls in California are dominated by wave reflection behaviour and they are not in fact, built nearly "thick enough" to eliminate this problem. Smith (1989) concludes that filter cloth is not a direct substitute for tertiary armour. If filter cloth is used, then the primary and secondary armour layers, must be increased in thickness or some tertiary armour must be provided as well.

For more comments on geotextiles the Reader is referred to the closing part of Chapter Six.

Transitions and Joints (Pilarczyk 1987)

In general, a coastal structure consists of a number of structural parts such as: toe protection, major structure in the area of heavy wave and current attack, upper slope protection (very often grass-mat), berm for run-up reduction or as maintenance road, etc. Different materials and different execution principles are mostly applied for these specific parts. Very often a new slope protection has to be connected to an already existing protective construction which involves another protective system. To obtain a homogeneous strong protection, all parts of protective structure have to be taken under consideration. Erosion or damage often starts at **joints and transitions**. Therefore, an important aspect of coastal construction, which requires special attention, are the joints and the transitions; joints within the same material and with other materials, and transitions between different constituents and units of structures.

A general design guideline is that transitions should be avoided as much as possible. If they are inevitable, the discountinuities introduced should be minimized. This holds for differences in elastic and plastic behaviour and in the permeability or the sand tightness. Proper execution is essential in order to obtain satisfactory joints and transitions.

When these guidelines are not followed, the joints or transitions may influence loads in terms of forces due to differences in stiffness or settlement, thermal effects, migration of subsoil from one part to another (erosion), strong pressure gradients due to concentrated groundwater flow, etc.

5.6 ARTIFICIAL BEACH NOURISHMENT

Inventory of Equipment Available

To cope with the particular conditions at a project site, including the borrow area and the reclamation area, an inventory is made of available dredging equipment with its specific capabilities and restrictions:

Work scenarios (see also CUR, 1987)

When confronted with an actual beach nourishment project, one has to combine all data and site conditions in order to evaluate which combinations of equipment will lead to an optimal solution with respect to cost and risk. One can combine the availabity of sand sources with the particular properties of dredging equipment. This leads to a number of work scenarios

(a) Hopper (direct method)
(b) Hopper (rehandle method)
(c) Stationary dredger pumping directly ashore
(d) Stationary dredger loading barges
(e) Portable dredgers

Measurement

One of the most evident risks inherent in beach nourishment is the volume of work involved. If the volume of work is uncertain, any contractor will allow a provision for this risk in his offer, however depressed the market. But determining the volume of work may be rather complicated.

In principle, there are several possibilities. A single "best" choice cannot be given as the optimum will largely depend on local conditions, but the main options are:

- through survey of the borrow area
- through survey of the beach
- measurement in barge
- measurement in pipeline.

In most cases, the use of a trailing suction hopper dredger leads to the most economic solution for beach replenishment.[1] Only if the borrow area is at a very short distance from the fill area, or if the borrow area is inaccessible to trailing hopper dredgers, may stationary equipment prove competitive.

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[1]The unit rate of a beach restoration operation varies only slightly with the total quantity involved. Only if volumes fall below 4...5 million m^3 does the cost of mobilisation play a considerable role. In both cases (coarse sand and/or large pumping distance) the use of a booster station is very cost effective.

When using a trailing suction hopper dredger the unit costs are lowest for medium-sized sand (approximately 250 μm), but if the effects of erosion losses and required initial volume, both as functions of grain size, are also considered, the optimum shifts towards larger grain sizes (in the example described, from 250 μm to 800 μm).

Artificial beach replenishment is a technically and economically sound solution for beach protection. The initial capital requirements are lower than for appropriate fixed structures, which makes it a particularly attractive alternative for developing countries.

An example of a beach fill project in the USSR is depicted in Fig.5.36. The circulation of water in the Bay of Gelendzhik (Black Sea) is counterclockwise in the urban area (bottom of the drawing) and clockwise in the north (top) which has caused deposition of sand in the central bay. Dredgers were used to nourish the dotted beach area in early eigthies to provide a perfect beach staying stable until now, without perceptible erosion and need for renourishment.

Kunz (1987) reviews the projects for two of the East Frisian Islands that experienced a different evolution of the inlet and the shore line. On Norderney, for more than one hundred years people attempted to stop the erosion of its west end by seawalls, groin and beach nourishment. On Langeoog however, most of the time there is a sufficient sand supply by approaching sand bars. Nevertheless for the last fifteen years, artificial beach fills have been carried out to reinforce barrier dunes and the beach in front of them.

The islands are composed of sand and silt. The higher parts are covered by dunes. A chain of barrier dunes stretches along the sandy beach on the seaside. The southern border is a salt marsh, naturally developed by siltation.

The islands have supposedly existed for several thousand years (Fig.5.37). The inlet between Norderney and the western island Juist (J) changed from a double to a single shaped profile. It was shifted towards the west end of Norderney by accretion of Juist and by extension of the inlet-catchment area to the east. Therefore man protected the west end of Norderney by alongshore structures and groins. On the contrary, the inlet between Langeoog and the island of Baltrum to its west remained comparatively stable. Langeoog was sufficiently supported by sand bars that migrated from Baltrum in a crescent shapped bow to its west end. Therefore this part of the island naturally enlarged and no constructions had to be built. The east end of Juist and Norderney propagate to the east as a result of accretion. This phenomenon is known as the "eastward migration" of the East Frisian Islands.

If the west end of the island is not sufficiently supplied with sand, the bow of sandbars approaches the island a distance from its west end. On Norderney this stage had been developed about 200 years ago causing man to prevent the increasing erosion of beach and dunes. On Langeoog this occurs periodically, since the beach approach pattern of the offshore bars scatters.

Beach nourishment on Norderney

The history of shore protection on Norderney is outlined in Chapter Seven. In this section we dwell only on the beach nourishment schemes.

The "classical" approach to beach protection by means of embankment and groins

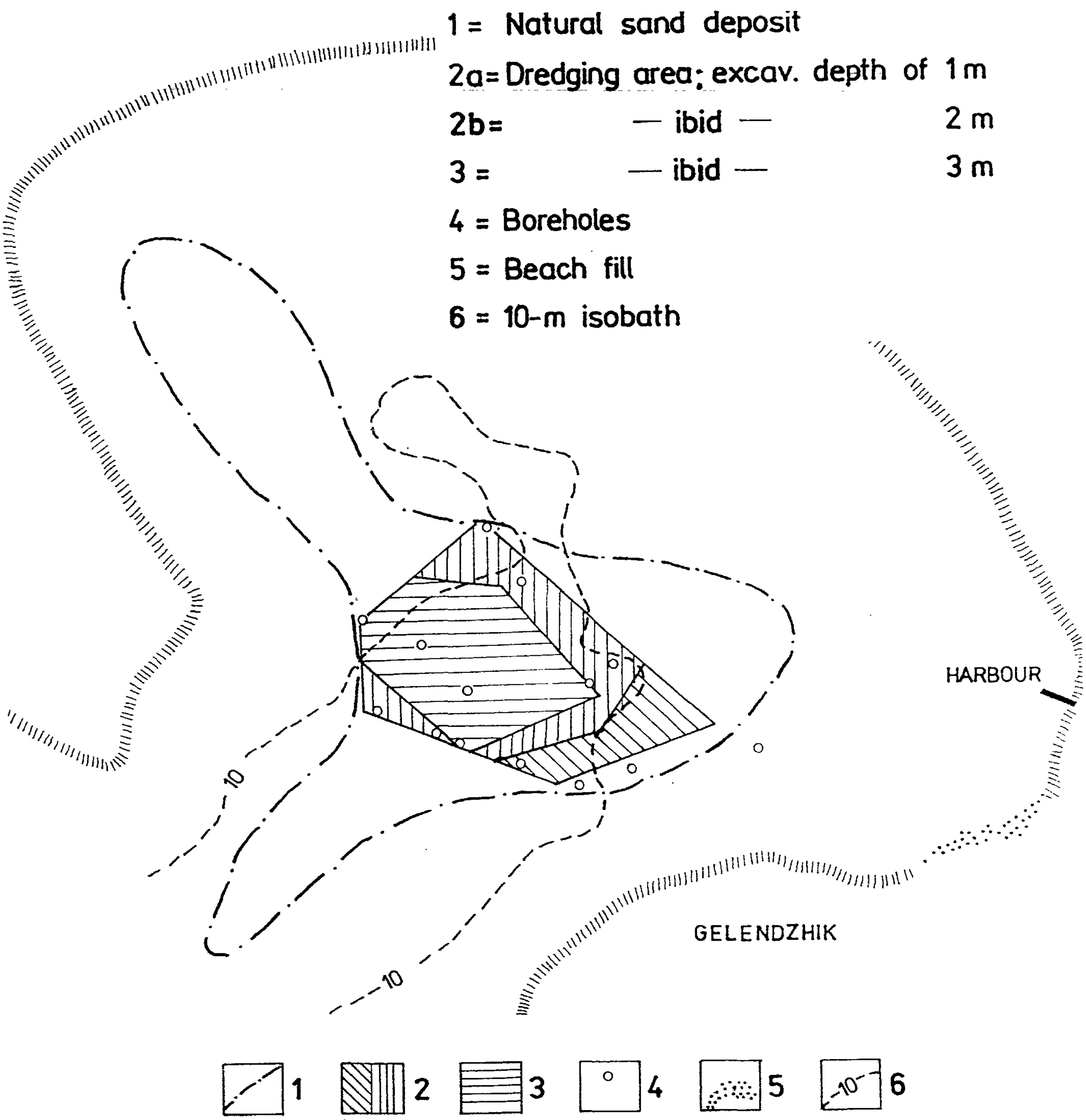


Figure 5.36. Beach Fill in Bay of Gelendzhik (Soviet Caucasian Black Sea).

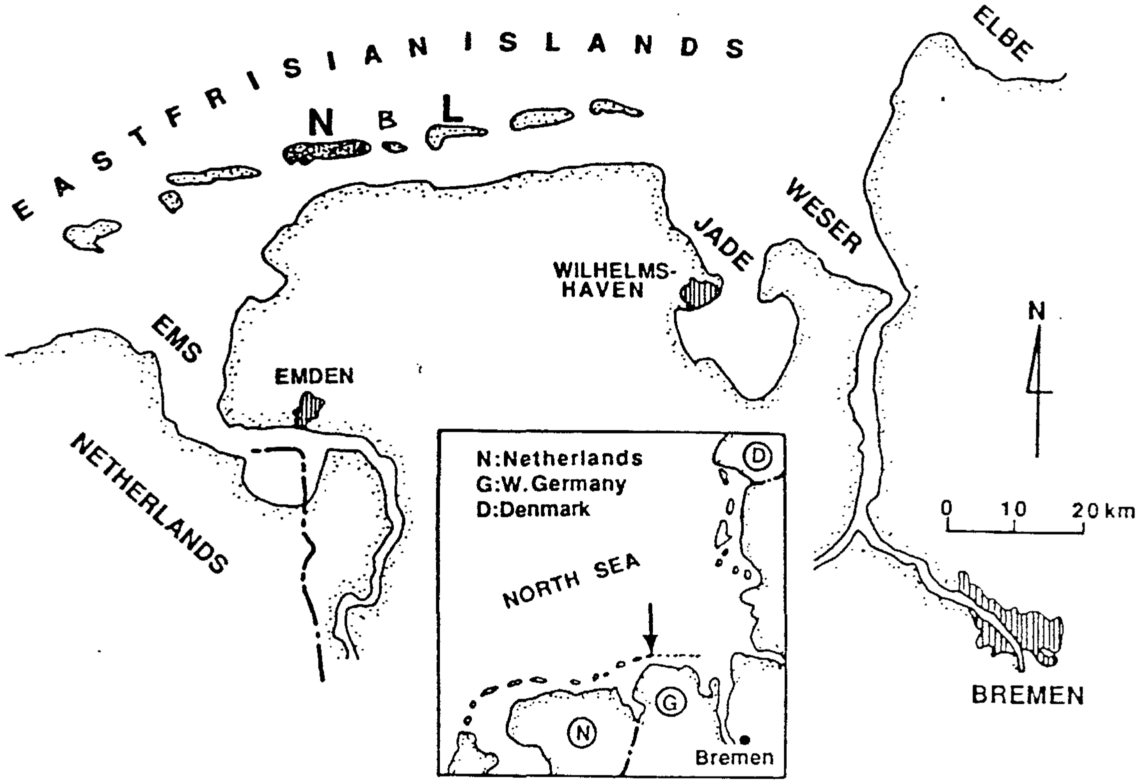


Figure 5.37. Location map of the East Frisian Islands, Kunz (1987).

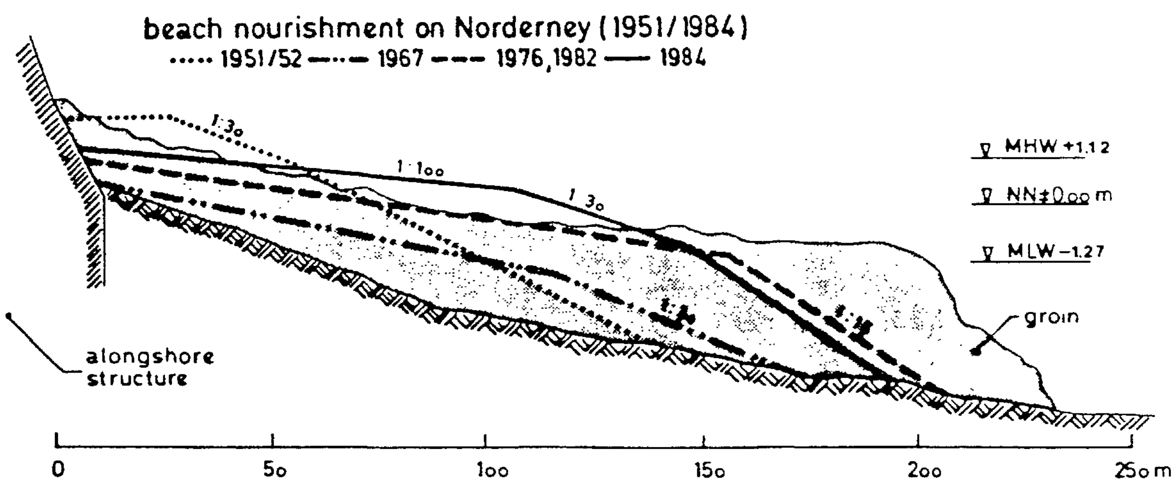


Figure 5.38. Cross-Sections between Groynes G_1 and H_1 on Norderney.

ended on Norderney in 1951-52. Upon recommendation from the "Working Group Norderney" (1952), the first artificial beach nourishment in Germany was implemented. Since the beach along the protected coastline was eroded, the restoration programme covered almost the whole stretch. The sand was dredged from the tidal flat and was finer $d_{50} = 0.1mm$, than that on the natural beach, $d_{50} = 0.02mm$ (Kramer, 1960). The second beach nourishment was conducted in 1967. In order to import coarser sand and to maintain short distances for the discharge pipe, the dredging was carried out from a foreshore area near the centre of the renourished beach.

Subsequent programs in 1976, 1982 and 1984, encompassed approximately the same stretch. The sand could be dredged and pumped from an ebb shoal across the tidal inlet. The quantity of dredged sand deposited on the beach varied between 1,250,000 m^3 (1951-52) and 400,000 m^3 (1976); the total amount to date is 2,850,000 m^3 or an average of about 80,000 m^3 per year.
Examples of beach profiles are shown in Fig.5.38. The cross-sections have been measured just after each of the five nourishment projects.

Erosion of the artificial filled beach may be assessed from Fig.5.39. According to theory, the change in the quantity of sand on the eroded beach should be logarithmic with time; that is the reason for the scale used in this figure. Storm waves and surge have significant impact on the beach. Therefore the amount of storm floods in presented. Up to now there is no sufficient data to conclusively correlate the erosion of sand with natural conditions such as water level, waves or currents.

The experience gained from the nourishment projects and the subsequent monitoring shows the following:
(a) the grain size of the dredged sand should be equal to or coarser than that of the natural beach,
(b) the slope of the artificial beach should correspond to the natural one,
(c) the beach must not be filled higher than the MHW level,
(d) a fill concentrated on a limited area of the western beach is sufficient as it acts as a feeder beach for the adjacent areas, and
(e) a minimum fill amount should be ensured.

Beach Nourishment on Langeoog

On Langeoog, the village and the low-lying land are protected against flooding by barrier dunes. Normally there is an overall sand balance because of sand bars approaching the west end. The volume of the sandbars and the point where they approach the beach changes with time. As a result, there have been periods of accretion and erosion. The development of the beach since 1920 is described in Fig.5.40 for different contours between low water level (MLWL) and the dune foot (NN + 3.00 m). Periods of extraordinary natural sand support are sticking out as peaks.

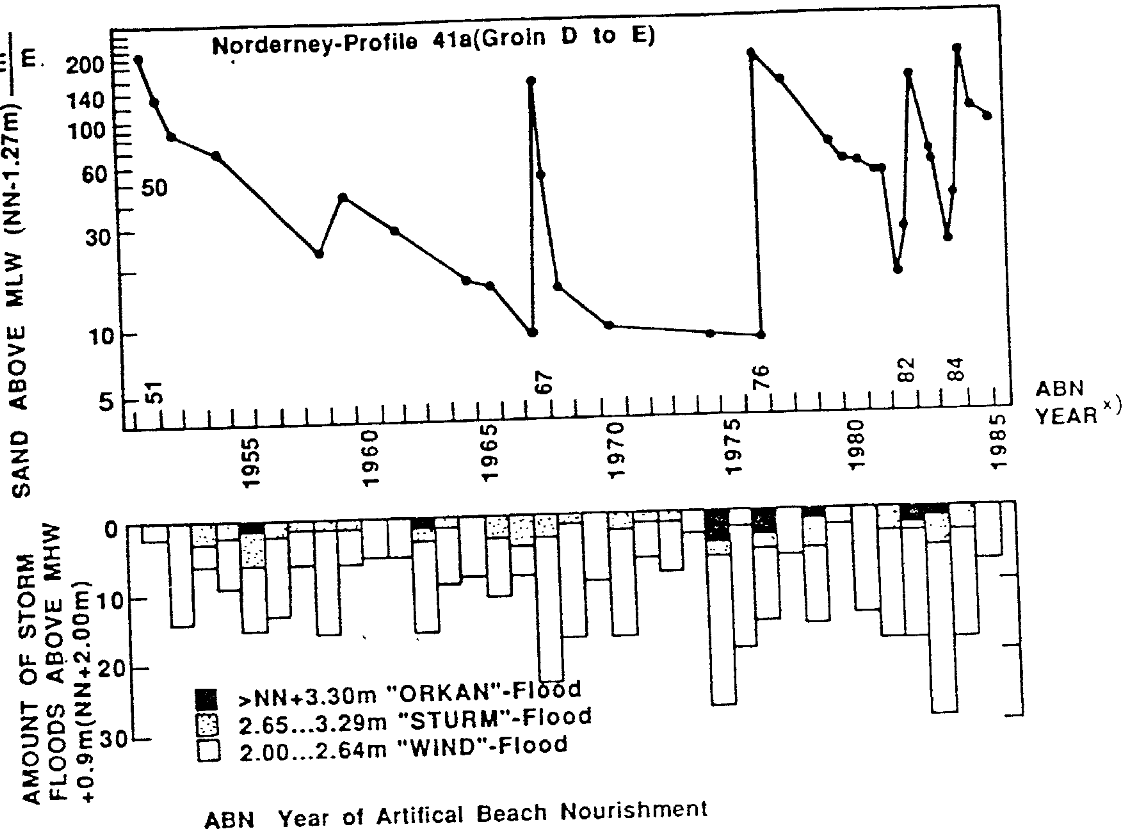


Figure 5.39. Beach Volume between Groynes D and E.

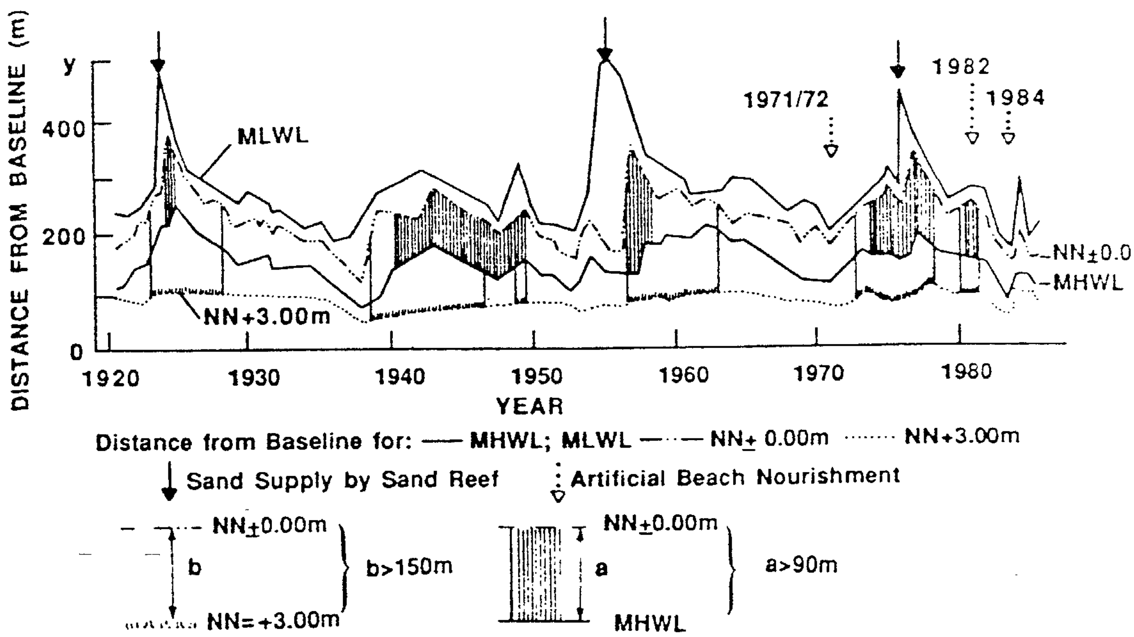


Figure 5.40. Development of Beach Profile No.16 (Fig.5.41).

The width of the beach decreases during periods with insufficient sand supply. The barrier dunes are only expected to be strong enough to withstand the impact of severe storm floods when the distance from the NN + 0.00 m line to the MHW line is more than a = 90 m, or the distance from the NN + 0.00 m-line to the NN + 3.00 m-line is more than b = 150 m . According to these criteria, in recent decades the beach often has been too small (see Fig.5.40). Such a period began about 1965 when fairly large beach erosion occured on the northern side of the island.

Five years latter the dune foot was so low that at slightly raised water levels the cliff was exposed to breaking waves. In 1971 and 1972, a 2.5 km-long stretch of the beach in front of the dune foot was widened and raised to such an extend that, even with storm surge, a natural beach surf could take place (Fig.5.41). The purpose of that fill was to raise the dune foot and not beach restoration. Therefore the replenished sand was trapped by a network of sand-filled synthetic tubes (Fig.5.42). Thus the amount of sand could be reduced from 1.5 milion m^3 for beach renourishment to 0.55 milion m^3.

Approaching sandbars covered the tube work for the next ten years. The following period of erosion brought the tubes to light again. On several stretches the tubes subsided (up to one meter), obviously by migrating channels. The tube work could not be used when a part of this stretch again had to be reinforced in the summer of 1982. At that time a sandbar passed that eroded stretch at a distance of 350 m. The migrating bar was turned onto the beach by a man-made dam through the swash channel (Fig.5.43).

The 300 m-long dam was constructed with 120,000 m^3 of hydraulically dredged sand (medium grain size 0.2 mm). It interrupted the current in the swash channel stopping the sand transport that was found to be from 0.1 m^3/h (current velocity of 0.04 m/s) to 2.2 m^3/s (current of 1.0 m/s). Furthermore, the dam influenced the wave climate as a "refraction groin". To prevent a breach in the barrier dune by storm surges the backbeach was filled with 100,000 m^3 sand. Seven months later, after a season with three times the normal amount of storm events, the bar and the dam had been deposited on the beach (pointed line in Fig.5.43).

In 1984, a part of the northeast beach of Langeoog had to be widened and strengthened. This again could be managed by turning the course of a sandbar and by backbeach fill (total 200.000 m^3). Since the dredged sand was fine and no further sand bars approached, this stretch will soon need artificial nourishment once again.[2]

Since the evolutionary process at the west end of Norderney was stopped by man-made shore protection structurs in view of insufficient supply of natural sand, this

[2]The cost of the artificial nourishment on Langeoog was : about 7.30 DM per m^3 (sand on the beach) for 1971-72 (tube work included) and about 3.65 DM per m^3 for 1982 and 1984. Even if 200,000 m^3 of sand were nourished yearly, the artificial nourishment would still be economical

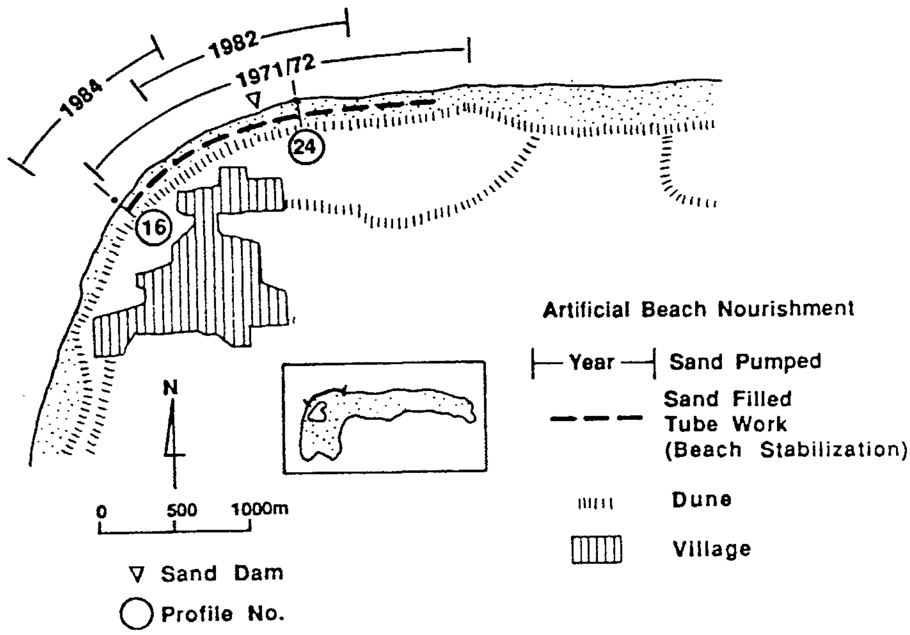


Figure 5.41. Beach Nourishment on Langeoog since 1971, Kunz (1987).

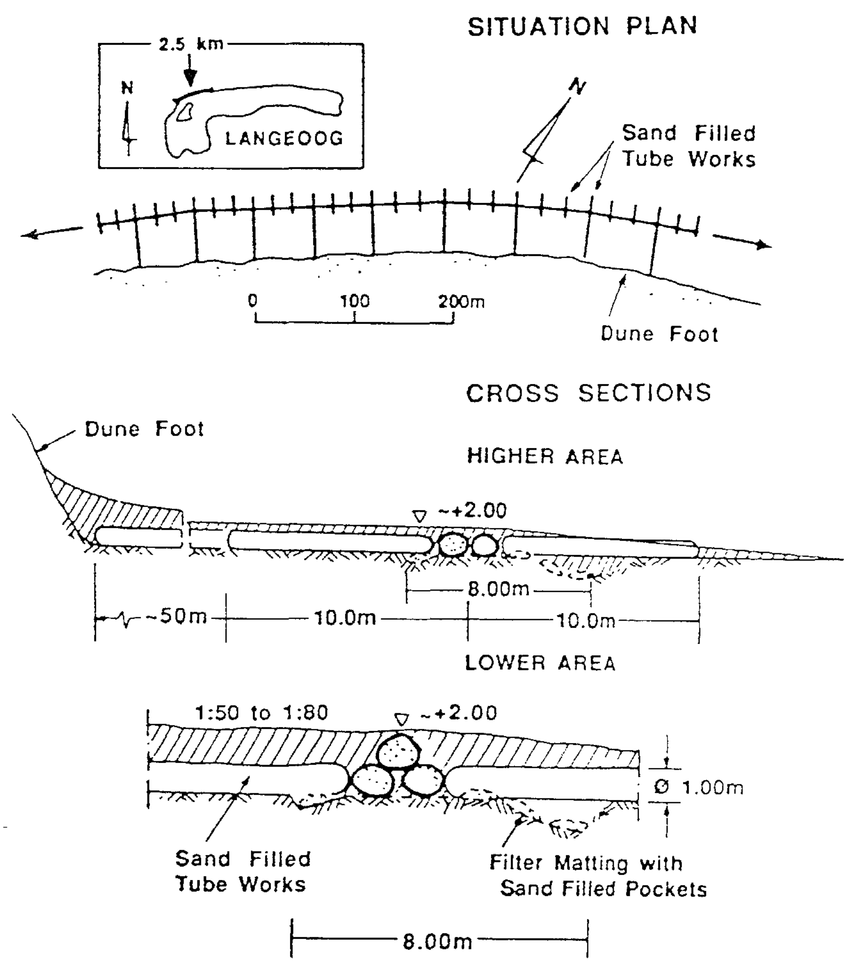


Figure 5.42. Beach Nourishment with Stabilization by Sand-Filled Tubes, on Langeoog, 1971...72, Kunz (1987).

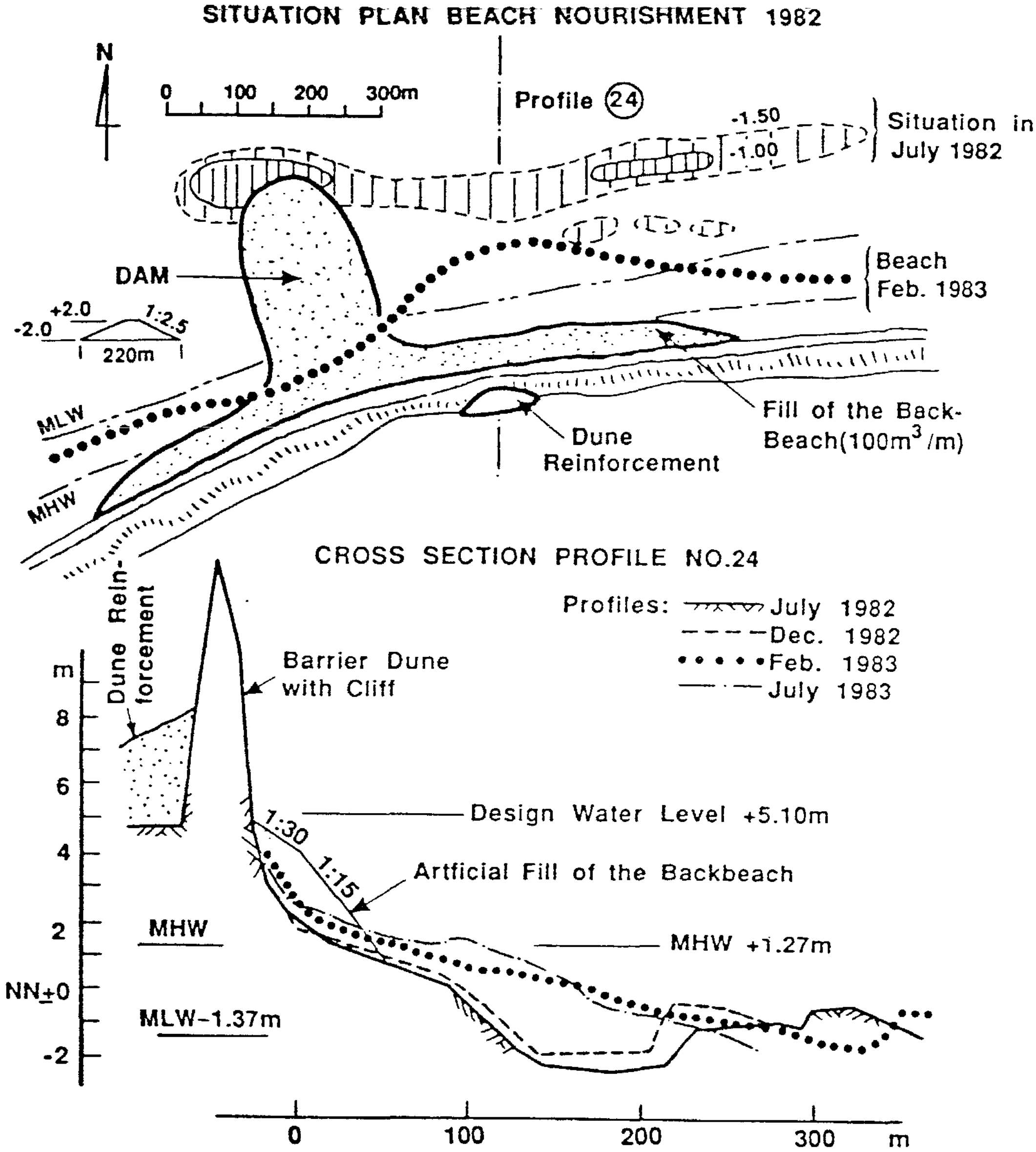


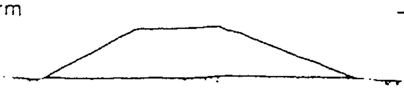
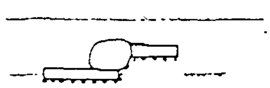
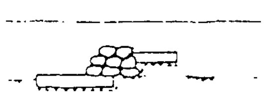
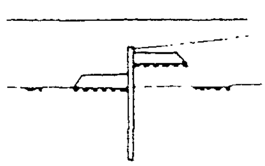
Figure 5.43. Fill of the Backbeach and Sand Dam through Swash Channel, Turning the Course of Sandbars on Langeoog, 1982, Kunz (1987).

area will further need artificial beach nourisment. Therefore a permanent pumping station has been in discussion for almost thirty years. An optimum solution is to be expected as an outcome of a recently started research programme.

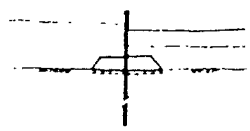
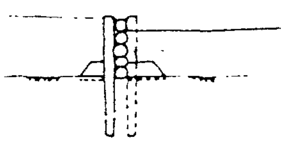
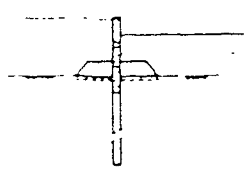
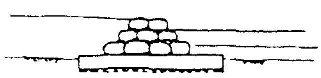
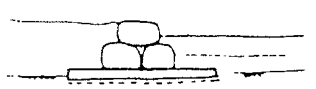
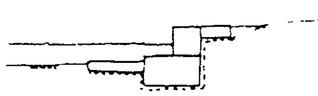
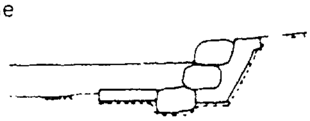
5.7 Orientation SUMMARY of Protection Methods

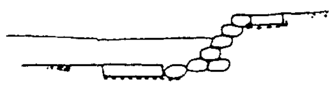
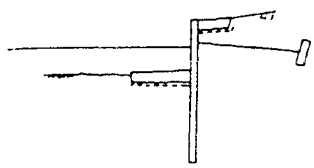
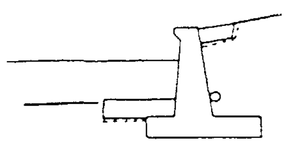
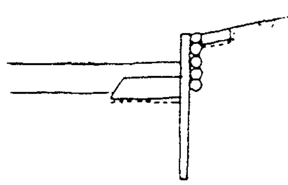
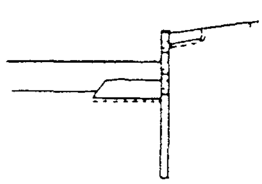
A concise outline of shore protection methods recommended for the Great Lakes coastlines has been provided by the Ministry of Natural Resources, Ontario (1987). We are copying three pages of that document to illustrate the advantages and disadvantages of different concepts. .

Summary of Protection Methods

Type of Structure	Typical Section	Advantages	Disadvantages	Life Expectancy (Years)
Beach Nourishment				
Beach Berm		— serves to provide littoral material during severe weather and/or high water periods (acts as beach dune)		NOTE (1)
— Beach Filling (bluff borrow area)		— native material involves less environmental problems	— may not have significant quantities of beach material (i.e. large loss of placed material)	NOTE (1)
(imported from inland location)		— allows choice of material to reduce losses of overly fine material — good life expectancy	— composition could affect water quality	NOTE (1)
Perched Beaches (Sill Structure)				
— Armourstone		— displaced material can be reused in structure — helps dissipate wave energy — resistant to ice damage	— could provide hazard to breaker users due to crevises between stones	
— Grout Filled Fabric Bags		— lowest cost		5 to 10
— Steel Sheet Piles			— reflective material — subject to scour	10
Revetments				
— Precast Concrete Blocks		— Construction not significantly hindered by access problems	— difficult to replace individual units when damaged — does not lend itself to staged construction — untested	10 to 50
— Armourstone		— proven construction method — maintenance and repairs easily completed	— requires heavy construction equipment and good access	10 to 20

(1) Due to the specifics of each site and the differing exposure of the fill, it is not possible to estimate the losses that could occur.

Type of Structure	Typical Section	Advantages	Disadvantages	Life Expectancy (Years)
Groynes				
- Gabion		— energy dissipating structure promotes disposition — low capital cost — can easily be installed by landowner	— wire basket very susceptible to damage by ice and abrasion — modern maintenance cost — short life expectancy — wire can pose a hazard to beach users	2 to 4
- Steel Sheet Piles		— resistant to ice damage — long life expectancy — clean appearance	— reflective surfaces does not promote sediment deposition as well as rough surface — cannot be constructed in the presence of bedrock — very high capital cost maintenance costs — requires heavy construction equipment	20
- Wooden Piles and Lagging		— can be repaired by landowner	— reflective surface — susceptible to ice damage — cannot be constructed in the presence of bedrock — high capital and very high maintenance cost	5 to 10
- 'H' Piles and Lagging		— neat appearance when properly aligned	— reflective surface — susceptible to ice damage — cannot be constructed in the presence of bedrock — very high capital and maintenance cost — short life expectancy — requires heavy equipment construction — piles tend to be displaced in service which results in loss of lagging	2 to 5
- Grout Filled Fabric Bags		— energy dissipating structure — not very susceptible to ice damage — relatively low capital and maintenance cost — good life expectancy	— lacks aesthetic appeal	10 to 20
- Armourstone		— energy dissipating structure — not very susceptible to ice damage — material can be reused if dislocated — relatively low capital and maintenance cost	— somewhat porous in nature — requires heavy construction equipment — high initial and maintenance costs	10 to 20
Shore Walls				
— Gabion		— easy to construct in inaccessible areas — can be constructed by landowner	— susceptible to significant damage when exposed to ice action	5 to 20
— Armourstone		— good wave dissipation characteristics — very stable even when subject to ice and wave action	— requires large construction equipment — scour protection required on shores without bedrock — does not stabilize against large slope failures	20 to 50

Type of Structure	Typical Section	Advantages	Disadvantages	Life Expectancy (Years)
- Grout Filled Fabric Bags		- good wave dissipation characteristics - very stable structure - least cost - relatively maintenance free	- may be aesthetically unappealing - scour protection required where rock not present - will not stabilize against large slope failures	20
- Steel Sheet Piles		- straight attractive structure - can stabilize slope against deep failures	- subject to reflection and scour - failures will generally be sudden without warning - very high cost - requires heavy construction equipment	30 to 50
- Poured Concrete		- attractive structure constructed to suit local conditions and landowner's desires	- very expensive - any failure will be sudden without warning and requires extensive repairs to rectify - subject to reflection and scour	10 to 20
- Wooden Piles and Lagging		- attractive appearance when maintained - can be constructed and repaired by landowner	- vertical wall subject to some reflection and scour	
- 'H' Piles and Lagging		- attractive appearance when piles are properly plumbed and aligned	- requires heavy construction equipment - subject to movement which will cause lagging to be lost - subject to reflection and scour	20

CHAPTER 6

Coastal defence systems and unconventional design

6.1 COASTAL DEFENCE & MANAGEMENT SYSTEMS

6.1.1 *General Layout*

Single shore protection measures have shortcomings that can be alleviated in combinations thereof. A number of coastal defence and management systems have been devised, quite often for generation and stabilization of *artificial beaches*. One can distinguish cases with limited natural sediment supply and/or substantial sediment deficit. A critical review has been presented by Bellesort and Migniot (1972), and many other studies appeared thereafter. We shall outline some of them, as more or less typical examples, although a huge number of versions can be invented.

Defined as *unconventional* are measures and systems that are of no established usage in coastal engineering such as headland control introduced in recent decades. Under this category we also mean low-cost, environment friendly, temporary and emergency undertakings.

6.1.2 *Sea Wall and Spur Groynes*

In Poland, experimental spur groynes were constructed in the years 1967...1968 along a spit type coast at Dziwnów, protected earlier with a sea wall. Subsequent measurements have identified the following features:

- The beach and shore were restored and stabilized in the first four years after construction. A wide beach was generated, and the total volume of accreted sand was assessed about $30,000 m^3$.
- A weak trend towards shoreline retreat was noted in the next years of 1973...1977. This was pronounced on the downdrift side of the system, with predominant westerly winds. The newly generated beach was eroded by about 1 m (as of 1977)
- The N...NE storms of 1978...1979 brought about strong erosion of dunes to

the west of the system, together with erosion of the beach at the spur groynes reaching some 1.5...2 m, and considerable contraction of the beach, by some 25 m, on the west side (in 1979)

- A strong storm from W, NW and N, lasting four days in 1983 caused a surge setup about 1.20 m. Heavy waves brought about severe damage of dunes on the eastern side of the spur groynes, along an 800-m strip of coastline, thus posing potential flooding hazards for the hinterland settlements. The width of the beach decreased to 10...15 m, and sand was eroded to 0.7 ... 1 m below the crest of the structure. Distinct erosion coves were generated on both sides of the groyne-spur system, although attempts employing additional groynes for prevention of the erosion were undertaken.

Fifteen years after the construction it can be concluded that the system failed as an insignificant improvement of sedimentation processes was reached in comparison with the situation prior to construction (1967). The status observed in the early eighties was

- a fairly narrow and steep beach (1:15...1:20)
- pronounced differences of sand levels on beach and between the groynes
- erosion coves on both sides of the system constructed
- 2.5-m holes in front of spurs, attaching at places to a segmented underwater bar, on depths of 1.2 ... 1.4 m
- accretion between groynes in the summer, followed by erosion due to storms in the automn and winter.

Nonetheless, one should not draw final conclusions on the effectiveness of the groyne-spur system, judged from the above example, as it can be highly site-specific. *Inter alia*, another spur system has been proposed by Anderson, Hardaway and Gunn (1983). In their implementation, the spur is a short, shore-parallel structure attached to a groyne near the beach. The resulting beach planform is similar to that found with a "T"-headed groyne. Construction materials varied depending on the wave climate. Materials used were wood sheeting, riprap, sandbags and gabions.

Of particular interest were episodes when waves approached the terminal groyne from the "updirft" side (Fig.6.1 top) Although the fastland end of the terminal groyne is in the lee under these conditions, the effects of refraction and diffraction allow waves to erode the offset corner. During such events the beach trapped. between the groynes realings to nearly parallel the incoming wave crests. Due to the high ends of most of the groynes, at or above mean high water, the incoming waves are diffracted at the end of the groyne. The smaller waves travel along the groyne and break on the beach. In storms, the diffracted waves erode the beach

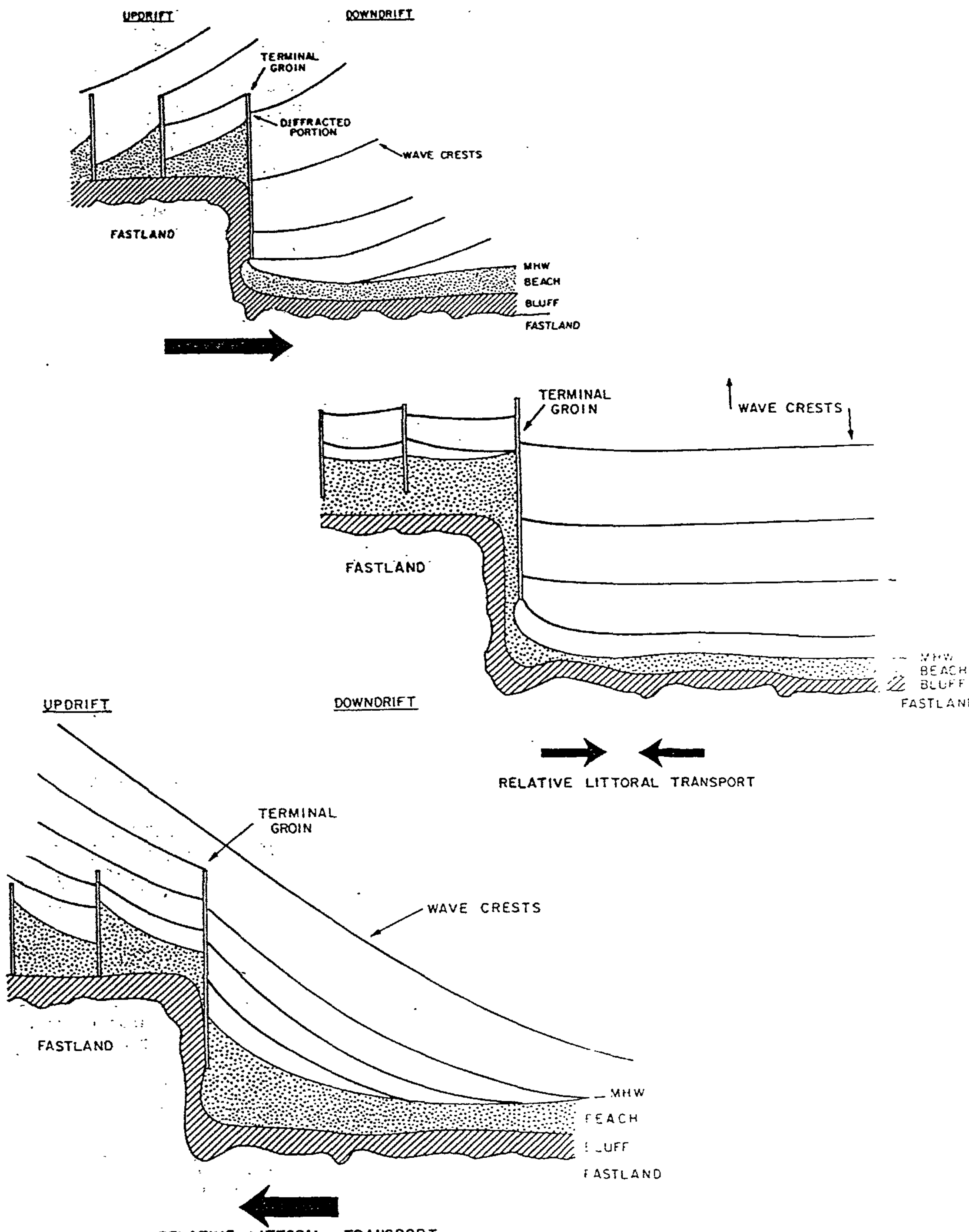


Figure 6.1. Performance of Groyne System without Spur, Anderson et al.(1983).

and the fastland bank at their juncture with the terminal groyne. When shorem waves approach parallel to shore the fastland bank is eroded, further removing it from the landward end of the groyne (Fig.6.1 centre). During the times when waves approach from the "downdrift" side, sand is moved into the offset (Fig.6.1 bottom). The offset itself is an indication of the rareness of events moving sand from the downdrift area back towards the terminal groyne.

Placing a spur on the downdrift side of the terminal groyne causes several adjustments in the beach planform. When waves approach from the updrift side of the offset, as indicated in Fig.6.2 (top), three things happen to the incoming waves. First, the wave is diffracted at the end of the groyne. Second, the wave is forced to break when it encounters the spur. Finally the wave is again diffracted which results in reduction of wave energy in the lee of the spur and a significant change in the direction the wave crest moves. Although the wave in the lee of the spur is much smaller than the adjacent non-diffracted portion, it can move sand into the lee of the spur. Under these conditions, the spur changes the eroding area into one of deposition.

The spur acts on waves approaching parallel to the shore in two ways. First, the portion of the wave encountering the spur breaks. Second, diffraction occurs at the end of the spur resulting in a smaller wave moving towards the offset. The net effect is a reduced movement into the offset. This effect is illustrated in Fig.6.2 (centre).

In this simplistic and qualitative description of the wave-spur interactions, the last condition is a wave approach from downdrift (Fig.6.2 bottom). Under this condition sand is moved into the lee of the spur from the adjacent beach face. The result is accretion of sand in the offset.

Guidelines

The low wave energy behind the spur enhances the residence time of the entrapped material. With the passage of time, one or more of the described diffraction/refraction effects force this ridge of sand into the offset. Taking maximum advantage of this process requires two design considerations. First, the spur height should not exceed that of the terminal groyne. Second, the spur attachment point should allow overtopping events to deposit sand in the lee of the spur.

The shoreline planform of the original offset, and the "stable" post-spur planform suggest such areas are similar to spiral beaches or crenulate beaches in the lee of a headland. With further definition of the characteristics of existing installations, it may be possible to use the formulas for spiral beaches to aid in determining the position of a spur. The critical elements are length and position along the terminal groyne. Althrough the spur appears successful in maintaining the attachment of a terminal the full suite of design parameters and beach response patterns remains incomplete. Until data becomes available, placement will have to be by empirical judgement.

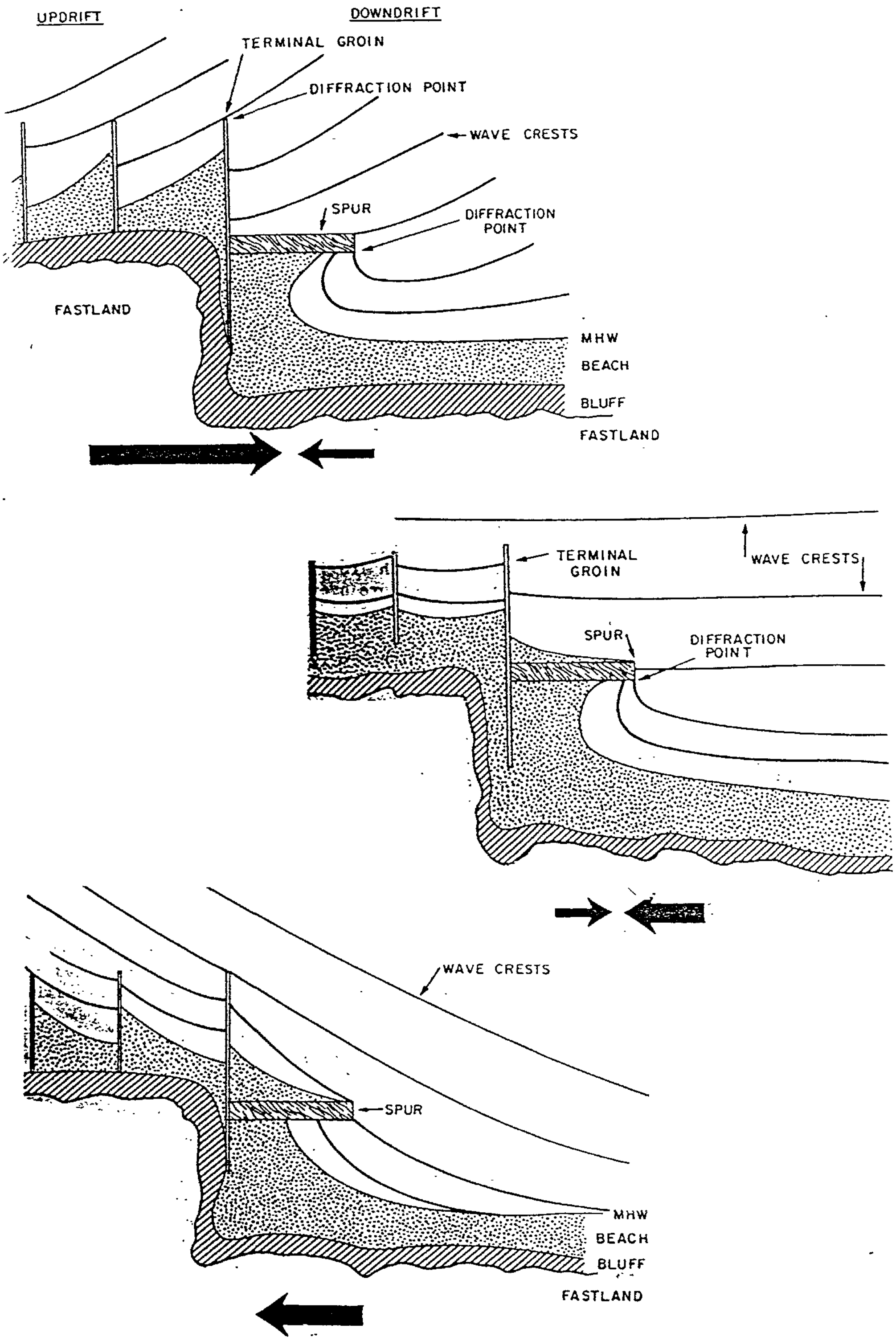


Figure 6.2. Performance of Groyne System with Spur, Anderson et al.(1983).

The spur concept is a useful, remedial measure for extreme offsets resulting from a groyne field. Although the total useful life of the structure will vary from site to site, the spur in the case study has weathered a moderate northeaster and prevented detachment of the terminal groyne for over four years. Spurs have performed equally well at the ten other sites not included in this discussion.

Also, it is not known whether spurs should be installed when the groyne field is originally constructed or at a later time. Clearly, much work remains before maximum utilization of spurs can be achieved.

Specific design parameters, whether they result from field studies or application of spiral beach formulas, need additional study.

6.1.3 *Artificial Beach Nourishment and Additional Structures*

Groynes

The beach at Cannes (France) was eroded after construction of the harbour. As a remedy, three groynes, reaching a depth of -2 m, were constructed along 860 m of the beach. The nourishment employed 70,000 m^3 of sand, coarser than the native one ($D_{90} = 2mm; D_{50} = 0.7mm; D_{10} = 0.3mm$ versus the natural $D_{50} = 0.15...0.5mm$). The beach was widened in 1960, and has remained in satisfactory condition for many years.

A marina built in Pornic (Yugoslavia) in 1972 required the construction of two groynes, 70 m and 100 m long, which divided the 200-m beach. Sand was supplied by hydraulic transport, in a quantity of $40,000m^3$, on depths of 0.5 ... 6.5 m.

Following the placement of 10.000 cu m of sand fill in a groyne compartment at Willoughby Spit, VA, changes in volume occured as a result of sediment movement in two shorewise zones: in Belt 1 (swash and wave dominated) by groyne overwashing at times of sufficiently elevated water level; in Belt 2 (beyond the groyne ends) by action of asymmetrical tidal currents. Unidirectional transports in and out of the compartment in Belt 1 were episodic and initially were equal but later an output exceeds input leading to losses. Sand that is shifted offshore from Belt 1 to Belt 2 moves along and out of the system. From a model, effective life of the fill is estimated at 8.8 yr. (Ludwick et al., 1987).

Detached Breakwater (and Groyne)

A system aimed at simultaneous accretion of nourished sediment and alongshore conveyance of its part has been put forth by Dean and Pope (1987). It was designed for Redington (USA), on a basis of extensive analysis of coastal changes using an m-line mode. The output criteria of the modelling procedures included primarily the stabilization of shoreline, with minimum downdrift erosion. The design resulted in the so-called "dog-leg" breakwater layout, with a groyne on the downdrift side. The crest of the breakwater was low enough (0.5 m above MLW) to permit overtopping

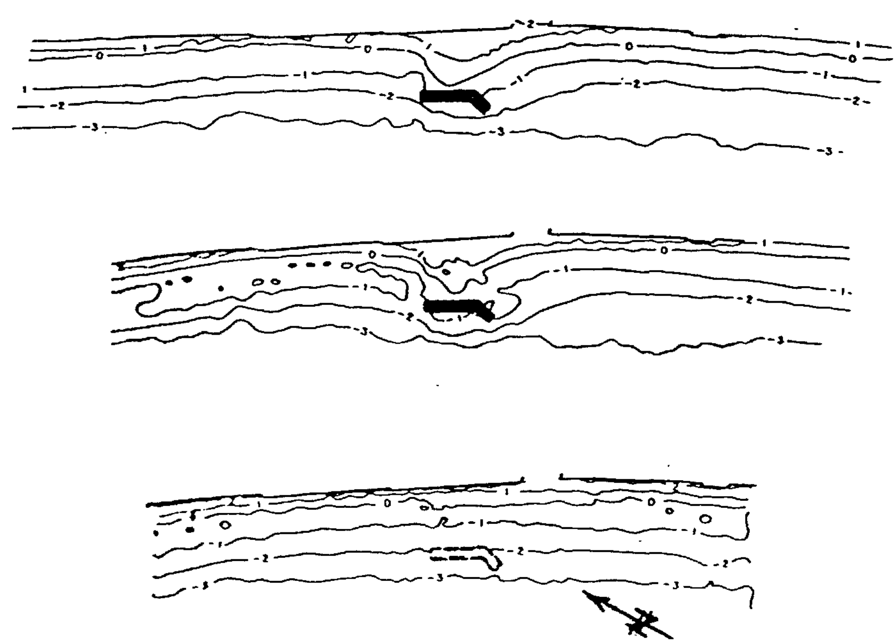


Figure 6.3. Beach Changes due to Construction of "Dog-Leg" Breakwater off Redington, Dally & Pope (1987); (a) November 1984; (b) April 1986; (c) December 1986; (predesign, four months, and one year after construction, respectively).

and penetration of sediment to the shoreward side of the breakwater during periods of higher water . The minimum amount of sand for nourishment was assessed as $20,260m^3$, while the optimum was put at $60,780m^3$.

Implementation of the design consisted of a modified version, without the groyne and with nourishment of 23,000 m^3. The breakwater (Fig.6.3) was constructed of 4500-kg units and sited on a depth of - 2.4 m, 91 m off shore. The alongshore segment measured 79 m and the diagonal segment (at 45° vs. shoreline) measured 46 m, its head protruding 120 m off shore.

Fig.6.3 summarizes the morphological effects of the dog-leg design in three stages: (a) prior to construction; (b) four months after construcrion; and (c) one year after construction. The sand entrapped or eroded in the vicinity of the breakwater is indicated in Table 6.1.

It can be inferred that the downdrift erosion was pronounced, but it decreased with time. The most intensive accretion occurred in the vicinity of the breakwater, on its updrift side, four months after construction. This has been attributed to artificial nourishment and entrapment. This accretion slows down subsequently.

Table 6.1. Volume Changes from Bathymetric Surveys

		11/84 11/84	4/86 4/86	4/86 4/86	12/86 12/86
Area	Length of Shoreline (m)	Vol. Change m^3	Per Shoreline Width (m^3/m)	Vol. Change (m^3)	Per Shoreline Width (m^3/m)
Far South	229	-12.802	-56.0	-2.469	-10.8
Near Sotuh	152	1.219	8.0	1.692	11.1
Breakwater	152	21.519	141.2	- 168	- 1.1
Near North	152	610	4.0	1.364	9.0
Far North	229	-3.658	-16.0	597	2.6

6.1.4 *Systems of Groynes, Offshore Breakwaters and Beach Nourishment*

Larvotto beach in Monaco was genereted in 1967...68 on a very steep slope. Three groynes and an offshore breakwater joined the design. Crushed limestone was filled on depths of 2.0...2.5 m, in a volume of 50,000 m^3, to create a three-pocket beach, 500 m long, sloping at % towards the sea. The beach is referred to as "suspended"(Fig.6.4).

Similar design was devised for Mourillon, where the beach was constructed in the years 1964...65. The sand is preserved in three pockets by the use of groynes and offshore breakwaters (Fig.6.5). The beach is 1000 m long, and its area is 80,000m^2. It is sloping towards sea at 3.5...5.5 %.

The same concept has been employed for other locations (Fig.6.6). Bellesort and Migniot (1972) suggest the following guidelines

- for beach slope of 2.5...3.5 % the spacing of the two groynes can be 245 m to reach the emerging breakwaters, 160 m long, on a depth of 4 m. the opening of the pocket is 90 m
- for beaches sloping at 3.5...5.5 % the pockets are smaller, and the spacing of the groynes should not exceed 120 m. The offshore breakwaters are 80 m long, at the spacing of 40 m.

Pope & Rowen (1983) provide an example for Lake Erie. Earlier protection with a sea wall and groynes failed. Therefore the recent project involved three rubble-mound breakwaters 76.2 m long and spaced at 48.8 m, together with two groynes at the extremities of the protected area, one short (45.7 m) on the updrift side and one long (106.7 m) on the downdrift side. The beach nourishment was designed as 3,823 m^3/yr to make up the anticipated yearly deficit. The breakwaters were emerging +1.8 m above SWL.

Although the amount of sand actually supplied was smaller than the design quan-

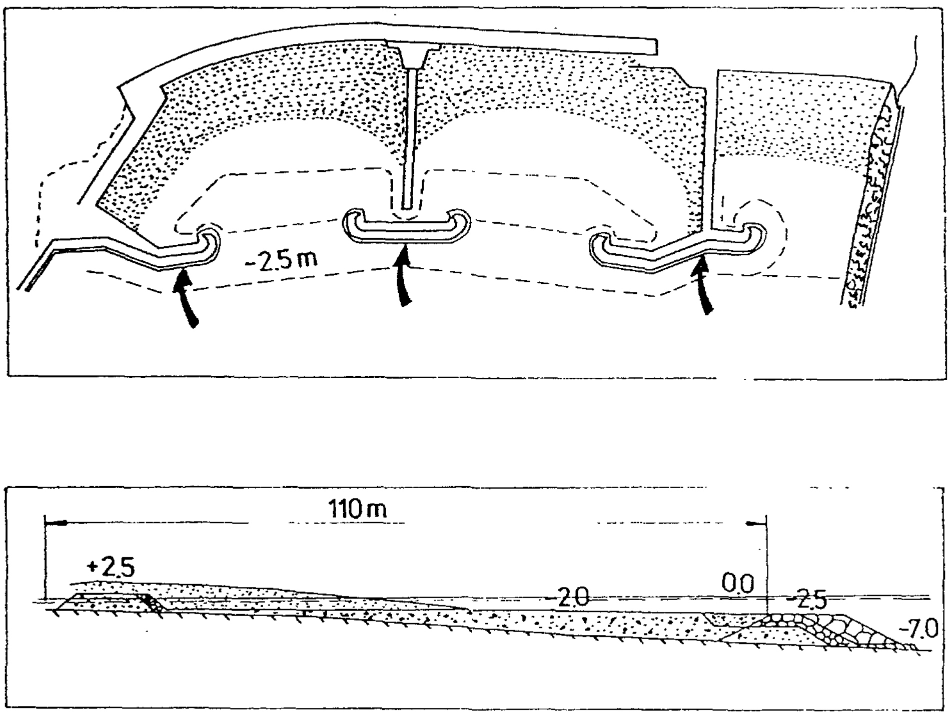


Figure 6.4.Artificial Beach at Larvotto Bellesort & Migniot (1972).

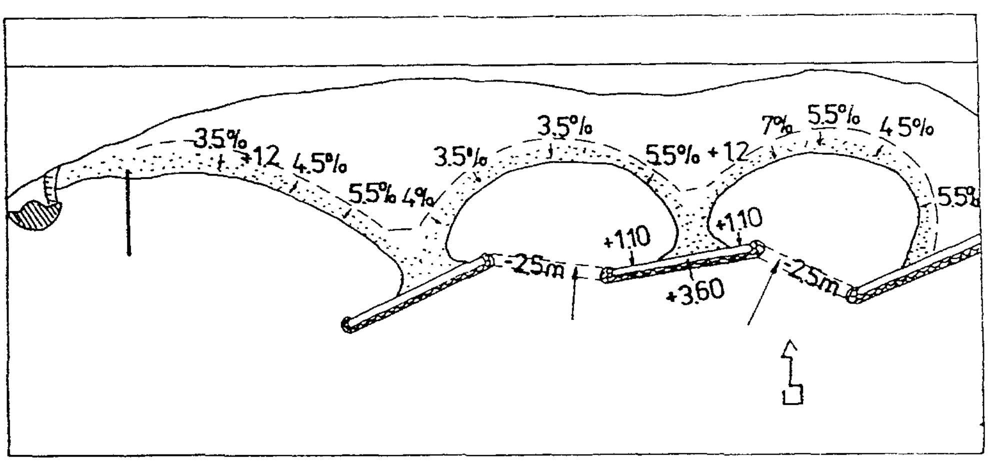


Figure 6.5. Mourillon Beach, Bellesort & Migniot (1972).

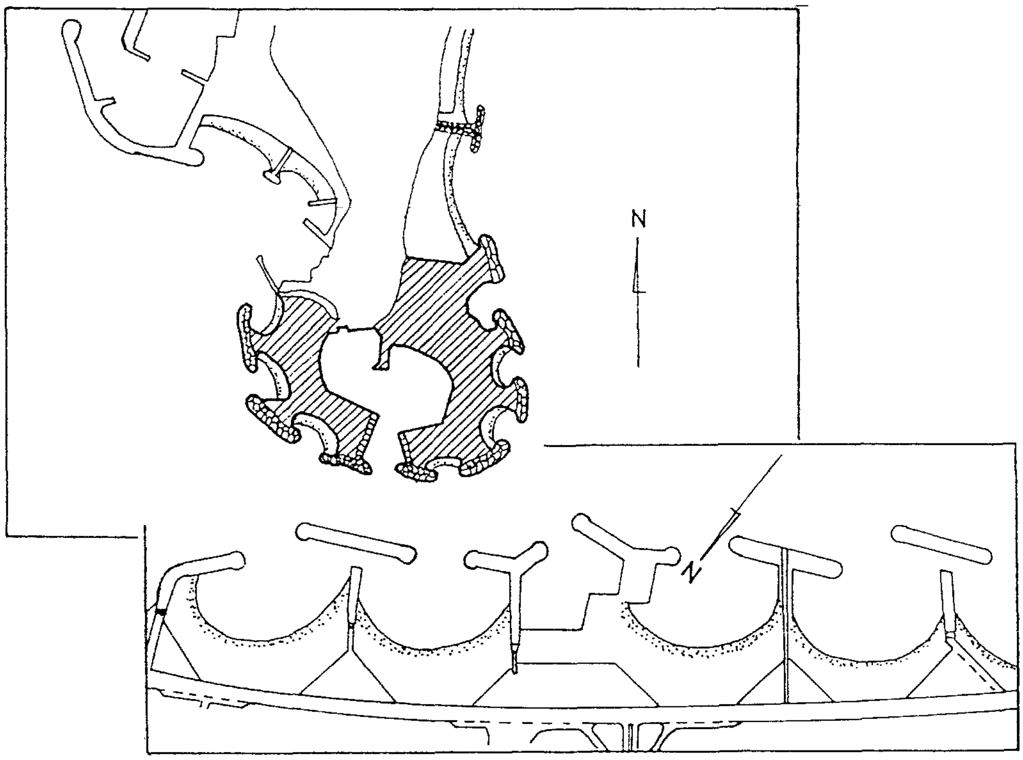


Figure 6.6. La Croisette and La Bocca Beaches, Bellesort & Migniot (1972).

tity, accretion has been observed in the protected area.
A few further examples have been shown in Chapter Two.

6.1.5 *Beach Nourishment and Diffraction Cones*

An ingenious system consisting of diffraction cones and submerged breakwaters has been proposed by Bellesort and Migniot (1972). The diffraction about the cones ensures favourable sedimentation patterns. Hydraulic model tests suggest that the cone diameter should be one-quarter of wave length, while the soacing of cones should equal one-half of wave length. The breakwater was submerged 1 m below SWL (Fig.6.7).

6.1.6*Other Systems*

A variety of different configurations of groynes, breakwaters, spurs and beach fills (Fig.6.8) have been considered by Spataru (1983) who presented a graphical method to estimate the sand filling capacity of longitudinal or oblique groynes (Fig.6.9).

The new beach area, which may be produced in the presence of the groyne (or a couple of groynes) is considered to be limited by circular arcs the centre (0) and radius $(r, r_M or r_m)$ of which are determined as shown in the drawing. This method

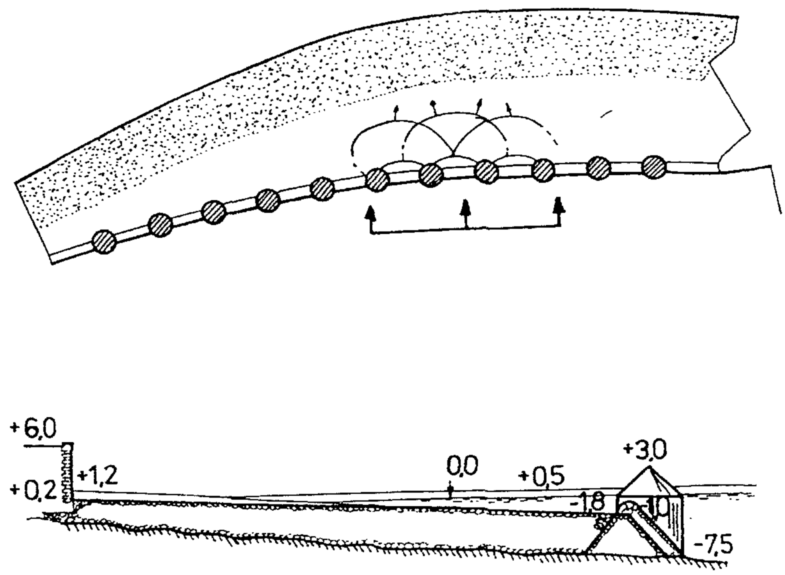


Figure 6.7. System of Diffraction Cones, Submerged Breakwater and Beach Fill, Bellesort & Migniot (1972).

was elaborated after the analysis of more than 150 tests performed on models to different scales.

The comparison for 35 groynes constructed on the Romanian shore of the Black Sea provides evidence on good agreement between the forecast of new shorelines (by the graphical method presented) and the field results.

These works were executed in the zone of the Romanian coast stretching from Constantza to the south, where the shoreline, with high cliffs, consists of a succession of concave segments, among rocky capes protruding in the sea. The design resulted in a very large area of new beaches generated at Eforie, Olimp, Jupiter, Saturn and Mangalia.

HEADLAND CONTROL

Headland control is a system of coastal management in which different shore protection measures are used to generate alternating headlands and crenulate bays in the state of dynamic equilibrium under the attack of waves.

Silvester (1972, 1976, 1980) has put forth the idea that crenulate shaped bays can be kept in equilibrium by use of a system of headlands. The persistent waves which sculpture the coast are diffracting around the upcoast headland and refracting in such a way as to arrive normal to the bay shoreline along its entire periphery. Hence the littoral drift is prevented and the shoreline remains stable. The headland system is in feedback with coast, and combines the advantages of groynes and detached breakwaters (shore-parallel or oblique).

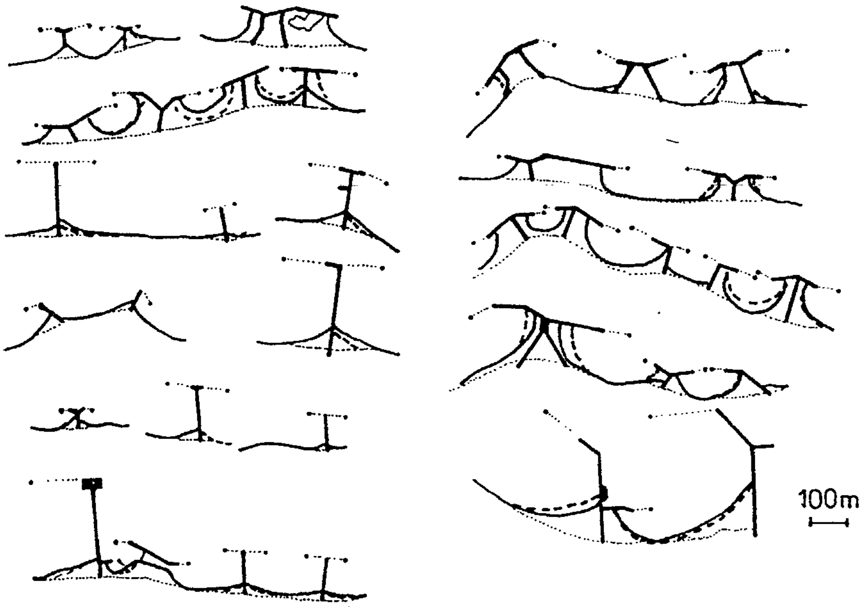


Figure 6.8. Various Layouts of Coastal Structures, Spataru (1983).

It must be stressed that the concept of headland control cannot serve a universal remedy for any coastal erosion (accretion) problem. In the writers' opinion Silvester neglects at least a few factors that might modify significantly the headland control concept, as noted in Chapter Two. For instance, constant wave obliquity may often prove an unrealistic assumption for natural environments with a wide two-dimensional (frequency- -direction) spectrum of incident waves.

Silvester (1984) himself agrees that substantial losses of beaches, in view of the erosion caused by handland downcoast of them, must be allowed for, and are in fact inherently built in the scheme. This voluntary retreat can barely be considered an advantage of the method.

Outflanking, undermining and other modes of scouring at headland structures are not exceptional as they also occur at other defence structures. However, the reliability of headland structures is of paramount importance as the feasibility of the whole concept depends largely on them, failure of a headland means collapse of the entire coastal protection scheme.

Some other **beach protection schemes** have been discussed by Swart and Horikawa (1986) on the basis of eleven poster presentations on beach protection schemes and/or devices, at the 20th International Conference on Coastal Engineering. The various levels involved in the design and evaluation of beach protection schemes are:

(1) the assessment of the problem and processes operative in the area;
(2) the establishment of an overall master plan for development;
(3) the design stage;

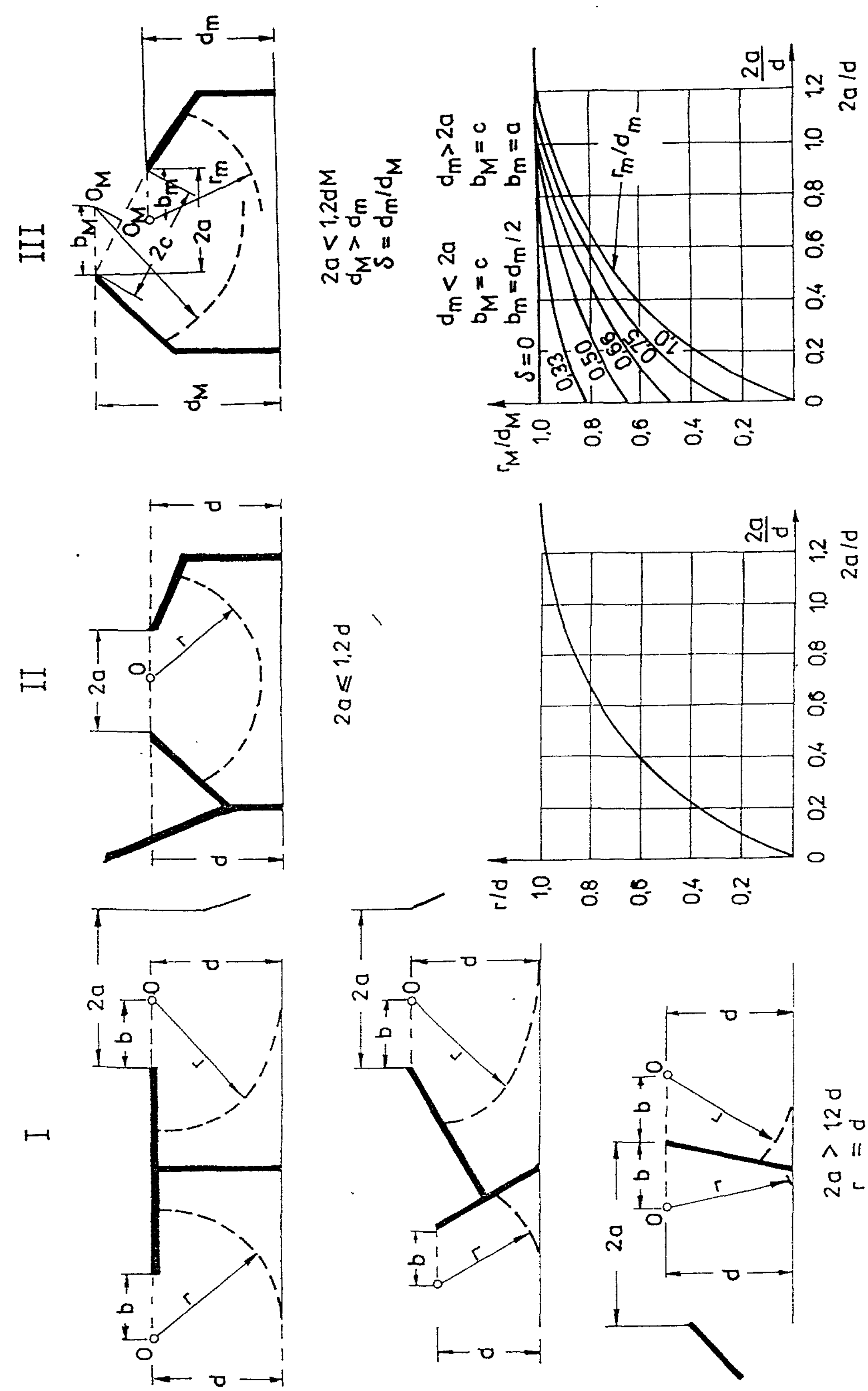


Figure 6.9. Spataru's (1983) Design Graph.

(4) the construction phase, and
(5) the post-construction phase.

It is concluded that it would be useful if some form of international co-operation could be instigated to allow the collation of techniques used at the various levels in one handy location for dissemination by all contributors. It is worthwhile to examine those presentations.

Moöller et. al. (1986) describe a diamond mining operation on the west coast of Africa in Namibia, where a sea-wall of normal beach sand has been built out to a distance of more than 300 m seawards of the original coastline. The wall which runs alongshore is maintained in the high energy environment, which is characterized by northbound longshore transport rates, by means of artificial suppletion at a rate of more than 300 000 m^3 month.

The individual survey results were used to establish a sand budget for each survey period. These sand budgets were used as input to a N-line littoral, morphological model. This model was preferred to a one-line coastal model because of the large alongshore difference in profile shape. During the three years that the project has run to date this technique was used to project the required volume of sand to complete the project by the expected completion date of March 1988. The project can serve as a one-to-one scale beachfill test and could prove extremely useful to evaluate techniques for the design of beachfills.

The paper by Yeh and Ou (1986) discusses the application of **offshore breakwaters** to the Redhill coast of Taiwan in the Republic of China. The coastline in the area consists of sandy eroding cliffs 6 to 10 m high and has a limited natural sand supply. In 1978 seven detached, offshore breakwaters with a length of 80 m each and a gap spacing of 30 m were built at mean sea level to protect the beaches. Field observations were done over a period of 3 years covering pre- and post-construction phases.

Substantial improvement in beach stability has been experienced since the construction of the breakwaters.

Combined beachfill/structures have also been analysed as it was realised that beach fill is often used in combination with structures, especially in areas where it is expected that the nattural littoral drift will transport sediment out of the area too quickly, thereby reducing the effective life-time of the beachfill.

Deguchi and Sawaragi (1986) describe how sand was placed in the lee of a 80 m long offshore breakwaer in 1984, one year after construction of the breakwater was completed. Surveys were done 4 to 6 times a year. Immediately after construction of the breakwater both longshore and offshore sediment transport took place as the beach reacted to the changed littoral regime. Thereafter longshore transport dominated until the beachfill was placed. Within two months after placement fifty per cent of the beachfill was lost from the area behind the breakwater, and a further ten per cent of the initial volume lost eventually in the subsequent two months.

Pui (1986) shows how as a result of land reclamation to meet the increasing needs of the fast-growing Singapore population, the shore was built out to such an extent that the littoral processes were accelerated, thereby necessitating the simultaneous construction in some areas of revetments or headland breakwaters. In a few cases additional measures wewre not required owing to the favourable alignment of the beachfill.

Kana et al. (1986) give a very good example of a combined beachfill/ /structures approach. It is most probably one of the most extensive recreational waterfront projects ever designed. The results of field-monitoring programme, which started in 1977, were used to drawup a master plan for beachfront improvement as well as a series of design criteria. The scheme includes artificial beaches, promenades, revetments, outfalls which also serve as headland breakwaters for the artificial pocket beaches and an artificial island. On the basis of the wave data longshore transport calculations were done and were used to orientate the pocket beaches to the waves, thereby largely eliminating longshore sediment losses. An interesting aspect of the design of the beachfill is that a berm elevation of 1 m above mean high water springs was used. This allowed washover and as a result the beaches attained a natural appearance soon after construction.

The monitoring shows that the sand losses from the scheme are low and it is not expected that renourishment will be required later.

Suyama et al. (1986) describes a field experiment on the Shimoni-Ikawa coast on Toyama Bay in Japan. Downdrift of Miyazaki fishing harbour the coast is eroding owing to the interruption of littoral drift by the harbour. The purpose of the field experiment was to investigatge the effectiveness of sand bypassing as a method of combating beach erosion and to evaluate the degree to which offshore breakwaters in the downdrift area can serve to minimize sediment redistribution after replenishment of the beach.

Slow longshore movement of sand took place without apprecaible sand losses through the gaps between the three breakwaters, which were situated about 820 m offshore. The artificial sand infill into the area behind the breakwaters helped to create tombolos, which would not have formed otherwise, and reduced wave overtopping of the structures due to shallower water at the structures.

When dealing with shore protection schemes on **river delta coastlines** the general problem is compounded because of the added processes associated with the river delta itself. Here specific attention should be focused on current patterns related to the river delta formation and orientation, decrease in the shore area, relative sea-level rise (relative shore drop) and shoreline protrusion with the associated effect on morphological processes.

The papers by Kadib et al. (1986) and Gambardella (1986) serve as reference.

Kadib et al. describe the development of the Nile river delta. Up to the end of the 19th century the supply of sediments to the delta generally exceeded the losses due to wind, current and wave processes in the shore area. After construction of the lower Aswan Dam on the Nile in 1912 beach erosion at a rate of 30 m/year commenced. This incrased to more than 170 m/year after 1966, when the Aswan High Dam was built.

The Egyptian Government recognized the seriousness of the situation, and instigated an extensive investigation programme the basis of an analysis of the field data gathered as part of this programme, the 350 km coastal area was subdivided into six morphological regions. Various shoreline protection schemes are being evaluated against each other, each taking into account the specific morphological behaviour of the region under consideration. Protection devices which are being considered are shoreline hardening with the use of armour units, sand nourishment, groyne systems or detached breakwaters in conjunction with sand nourishment, "no action " with assessment of the economic consequences and other options.

Gambardella (1986) shows that the shoreline of the Po river delta in Italy receded dramatically since the 1950's. Two main processes can be identified as being causative factors. Natural subsidence of the more than 2 000 m thick, unconsolidated post-Pliocence sediments which is about 1.5 mm/year, increased to about 15 mm/year due to water and gas extraction, reaching a maximum of 3 m/40 years locally. Secondly hydraulic works, industrial installation and extensive reclamation works altered the natural equilibrikum of the outer coastal area, thereby giving rise to shoreline recession. It is not always possible to shift back the protective sea-walls because settlements have been established in the coastal area immediately behind the sea-walls. Due to the substantial erosion of the delta beachfront the dike/breakwater systems proved ineffective to prevent beach erosion. The application of 1 m to 1.8 m diameter sand-filled "Longard" tubes in a configuration as shown in Section 6.2 has to date proved very effective and is considered the mechanism to combat future erosion.

For **other beach protection techniqes**, using the Hawaiian experience as an example, Hayashi shows that sea-walls with a nearly-vertical seaward face used to protect properties and maintain the certified shoreline, cause that beach erosion is frequently enhanced and extends to the adjacent properties, thereby giving rise to the so-called "domino" effect. The use of beach-walls, a concept developed by Hayashi (1986), however, leads to much more natural beach processes. Experience over two years has shown that beaches are retained more readily in front of beach-walls than in front of sea-walls or other hard structures. Slope revetments employ the same rationale.

Kawata & Tsuchiya (1986) discuss both theoretical and experimental studies to investigate the effect of a sub-bottom filtration system whereby water is drawn into the bed. As is to be expected, the increased inflow velocity into the bed causes the threshold velocity of bottom sand to be increased, thus preventing erosion.

Although the laboratory experiments showed conclusively the benefit of such a filtration or draw-off system, the main problem that remains is the manner in which such a system can be constructed and maintained on a natural beach.

6.2 UNCONVENTIONAL DEFENCE: LOW-COST, ENVIRONMENT FRIENDLY, TEMPORARY & EMERGENCY MEASURES

6.2.1 *General*

For a variety of reasons, new approaches or solutions to the shore protection are proposed and marketed from time to time. The first of these reasons is the tendency to diminish cost although one should remember that practically any method of coastal defence, if properly implemented, is expensive. Significant investments are required to achieve the durability needed to resist even small waves.

On a rapidly eroding coastline, any protective structure built to withstand direct wave attack will probably fail eventually. Even a well-designed structures is likely to fail once its design life (commonly 20 to 25 years) has been exceeded, especially if it is not properly maintained. Spending large amounts of money on the installation of a coastal engineering structure does not secure long-term, or even short-term in some cases, protection for home and property.

In this context, **"low cost"** simply means that the various measures are commensurate with the value of individual residential or commercial properties. The total cost of implementation will vary with the different alternatives, but in all cases, there should be a suitable (and affordable) range of solutions.

The low-cost technology described in this chapter is usually appropriate for use only in sheltered areas, and is generally not intended for open oceanic coastal that would be exposed to the unrestricted attack by large waves. Use of most of low-cost structures in such areas is definitely not recommended and entails a considerable risk of failure (US Army, 1981).

A number of lightweight coastal protection structures built on the Lake Erie are discussed by Kamphuis (1975). The paper describes the performance under normal and highwater conditions. It is found that inexpensive low-key structures are sufficiently strong to survive normal conditions, but fail by overtopping and flanking under conditions beyond their low design limits.

The experience gained by Brater (1979) in more than 25 years of observations and laboratory research on the effectiveness and durability of low-cost shore protective procedures has demonstrated that they may be economically feasible if compatible with the shore processes at individual sites.

Low-cost measures may be considered a certain compromise between two extremes. At the one extreme are massive, carefully engineered concrete seawalls, which function effectively for over fifty years. At the other extreme, slabs of broken concrete or asphalt, or other construction debris is simply being dumped at the base of a cliff or any other endangered type of coastline. Most efforts of this more or less non-engineered sort have been relatively futile or very short-lived. They nevertheless solve the coastal erosion problem temporarily at a relativelly minor expense. Sometimes such measures are applied on emergency basis when one is left with no solution at all if he gives up the idea of stacking a pile of sand-filled bags in front of his property at risk. Accordingly, included in this chapter are measures classified as "**emergency**" and "**temporary**".

Some of the structures used in shore protection pose an environmental, or at least aesthetic, threat either direct or indirect. Broken debris, no matter how unwanted on the construction site, becomes by no means a welcome scenery if disposed of on beach where it may however contribute to preservation of adjacent sections of coast. On the other hand, an invisible submerged structure, apparently environment-friendly, may cause, through enhanced erosion, serious domages to its downcoast neighbour. An "**environment-friendly**" are then these measures which not only protect the environment but also do not pose any threat by themselves. Such measures embody i.a. dune stablization by use of vegetation, beach fills etc. These more or less natural measures are discussed in Sec. 6.2.2.

One may apply different criteria in classifying the measures being described in this chapter, viz. structure type, structure material, degree of novelty, shoreforms, shore uses, etc. The first orientation has been chosen, and the following discussion will accordingly reflect the division already accepted for the other chapters, i.e. for (1) groines, (2) breakwaters, (3) bulkheads, seawalls and revetments, (4) beach fills, slope flattening, perched beaches. The low-cost etc. solutions for these types of structures, in addition to dune stabilization and some other areas of shore protection are presented for the above classes, although some repetitions may be unavoidable due to the fact that similar measures, e.g. sand bags or sand sausages are used alternatively in different types of structures.

6.2.2 *Dune stabilization*

General

Dunes are formed directly on the upland partions of sandy beaches. They play an important role in the coastal processes for they accumulate sand which can subsequently protect their hinterlands. Hence, the protection of coastline can be accomplished through the creation of barrier dunes and the stabilization of present dunes. Vegetation used to initiate the building of barrier dunes is specially adapted to the more severe environment of the beach area. Barrier dune formation can occur naturally, but is usually slow and in some areas does not happen. Utilization and proper management of the natural processes can accelerate the development.

The beach provides a generally harsh environment for plant growth. Plants must tolerate rapid sand accumulation, flooding, salt spray, sandblasts, wind and water erosion, wide temperature fluctuations, drought, and low nutrient levels. Plants capable of stabilizing coastal dunes do, however, occur in most coastal regions where there is sufficient rainfall to support plant growth. These regions and several of the most succesful species are discussed below.

Sand driven by wind accumulates at a barrier on beach and may create various accumulative forms, which often disappear if wind stops to blow or eventually generate a dune. For constant supply of sand the dune reaches its state of equilibrium and then begins to move in the direction of wind. HUECKEL (1975) indicates that dunes may travel landwards at a speed of 5 m per year and burry the objects on their way (forest, farms and even hamlets). The height of dunes along the Polish coastline reaches 56 m, while the highest dunes encountered are up to 200 m (Tripoli, Libya).

Man controls the formation of dunes and thereby provides natural shelter for their hinterland. **Dune-forming fences** are used to build new dunes (so-called white dunes) or to raise and replenish the existing ones. Upon the formation of dunes one continues with dune stabilization, initially by building stabilizing fences and later by planting various suitable species of **grass** (usually in spring). If grass survives one year the dune is deemed stabilized. Since the dune transforms into a grey one, with the variety of vegetation changing in time, some three years after stabilization the dune is ready for **forestation**. Hedgerows are recommended at the seaward limit of the forest, and robust or dwarf trees, such as willow or sea-buckthorn (Hippophae rhumnoides) are found most useful for this purpose. Resistant shrubs and trees should be planted on the seaward side of the forest, gradually mixing with evergreen and deciduous trees upon landward transgression. Trees are planted in springs; older seedlings must be used and cultivated over two years.

The importance of dune stabilization and forestation was realized in America as early as in the 17th century. The Province lands at the northern end of Cape Cod (Massachusetts, USA), was forested in 1620, according to the earliest Pilgrim writings (Mc Caffrey & Leatherman, 1979). Land use restrictions through legislation showed that the colonists removed the wood from that fragile ecosystem and grazed livestook on the dunes. The latter soon became barren and susceptible to aeolian transport - and the area was described as comparable to the Arabian desert. When Province town and the harbour were threatened by advancing sand, attention was finally focused on this issue and plantings to stabilize the dunes were undertaken.

To halt eastward shifting sands 20 miles south from Fort Stevens, Oregon, where severe erosion and inland migration of coastal dunes began in the 1930's, a sand stabilization program was initiated in 1935. Plantings of European beachgrass, Ammophila arenaria, were used as the primary sand stillers.

The changes that occurred before 1935 and results of the stabilization plan are reported on by Meter & Chester (1977).

Phillips (1980) has provided an intensive review of the historical and present work on transplanting seagrasses, including elgrass, turtle grass, shoalgrass, manatee grass and ditch grass. He has listed the best seasons, recommended methods of transplanting and suggestions for use of each species on the coasts of the United States.

A report by Knutson (1977) presents techniques and guidelines for efficient propagation of beach grass to form and stabilize dune systems as a natural barrier to the inland penetration of waves and storm surges. The guidelines include i.a. selecting plants and planting methods; obtaining plants, storing, planting and maintaining plants, estimating labour and requirements for planting projects.

Monitoring of foredunes formed from grass plantings was conducted in Texas during 1969 to 1973 (Dahl & Goen, 1977). The beach and foredunes, a 200 m cross-section, gained 4.4 m^3 per linear feet of beach from August 1975 to August 1976, although the north end of the study area acquired no sand.

Planting, 15 m wide seaward of the existing foredunes proved to be an effective dune-widening techniques, as they provided a 38-m dune base versus 24-m base resulting from the initial 30 meters of plantings.

The study on dune stabilization by Brisbane-based Beach Protection Authority (1983) contains reports on field stabilization and mulch trials in preparation for the growth and nutrition of various dune-stabilising species, Horsetail sort (Casuarina equisetifolia var. incana) and sand spinifex grass (Spinifex hirsutus) are investigated amoung the best surface stabilising flors. The effect of different forms and rates of nitrogenous fertilizers on the establishment and growth of horsetail she-oak is shown together with the effects of mycorrhizal inoculation and urea plus superphosphate combinations on the growth of spinifex grass. A long-term research programme conducted by the same authority (Mc Kenzie & Bar, 1984) concentrates on dune forming fences and the impact of various dune management mesures. Among others the programme has produced information on the effectiveness of semipermeable dune forming fences.

Stabilization of bluffs is another problem of the same category. Often, structural corrections are required in line with vegetation. Once the structural stabilization is accomplished, vegetative cover will aid in preventing erosion, reducing seepage, and slowing runoff.

The type of vegetation which can be established on bluff slopes is dependent upon the slope angle. Slopes steeper than 1 on 1 generally preclude successful vegetation, but slopes flatter than 1 on 3 can be planted as lawn and maintained in the usual manner. Slopes between 1 on 3 and 1 on 1 can be planed with grasses which will not be mowed, ground covers, trees and shrubs, or combinations of these three.

Coastal marshes are those herbaceous plant communities which are normally inundated or saturated by surface or groundwater. They may be narrow fringes along steep shorelines or they may cover wide areas in shallow, gently sloping shore regions typically found in bays and estuaries. In saltwater marshes, salinity is generally equal to or slightly less than seawater. Freshwater marshes expericence water level fluctuations resulting from groundwater table and seasonal climatic changes.

To establish a coastal marsh, the site must be evaluated based on geographic area, tidal eveluation and range, salinity, fetch length, and soil properties. Planning specifications are summarized in Table 8.2. The suitability of a site of marsh plantings can be evaluated using the patterns presented by US Army Corps of Engineers (1981).

Knutson & Woodhouse (1983) also provide guidelines on using coastal marsh vegetation. This erosion control alternative is suitable for relatively sheltered shorelines such as those found on bays, sounds, and estuaries. For various seasons this alternative has not been found to be effective in the Great Lakes, Alaska or Hawaii. Criteria are given for determination of site suitability, selecting plant materials, planting procedures, assessing impact, and estimating project costs.

Operation of and computations for dune fences

Numerous authors have investigated sand entrapment by dune fences. Hotta, Kraus & Horikawa (1987) described the patterns of air streamlines about an impermeable fence. For a fence of height H, air streamlines have been found to deflect at the distance L_I = 3...5 H. On the downstream side the eddies are larger and the size of the circulation cell L_o is 7...10H, with maximum return velocities recorded at the distance K_r = 2...4 H. The flow about the obstacle terminates at the distance $L_I = 9...17H$ but it is only L_s = 25...H that the effect of the fence is not felt.

For impervious fences the circulation patterns change. The eddies on the upstream side disappear for a 20-percent permeability. Bacause of wind penetration through the fence, the downstream circulation weakens and moves downstream. With increasing wind speed, the circulation cells disappear at the fence. Eddies are not detected for a fence permeability of 70...80 percent.

Hotta et al. (1987) reviewed the wind tunnel and field studies on sand entrapment at a dune fence. Sue (1969) compared different fences having permeabilities of 38 and 60 percent, subject to winds of low ($5ms^{-1}$) and high (9 ms^{-1} speed. Sand was found to accumulate upstream and downstream of the fence if wind was weak and permeability low, while for higher permeability and faster wind sand was entrapped primarily downstream of the fence. For increasing permeability dune crest moved away from the fence. The upstream slope of the dune was the steeper the higher

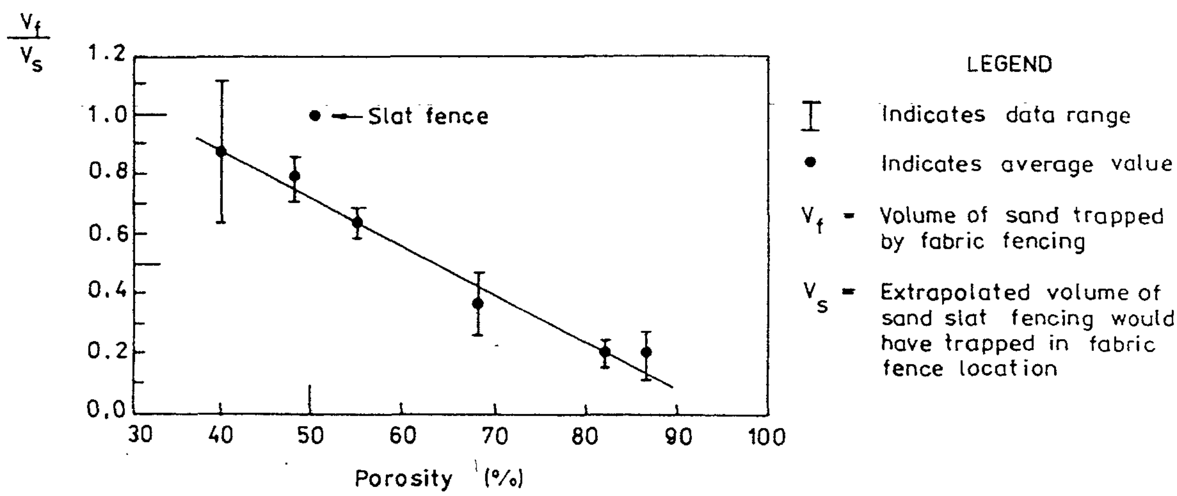


Figure 6.10. Effectiveness of Sand Entrapment by Plastic Fences of Different Permeability, Savage (1968).

the wind speed. The dune slopes flattened for increased sand entrapment.

Manohar & Bruun (1970) described the stages of dune growth about fences. An initial V-shaped scour immediately behind the fence was followed by the formation of a secondary dune on the downstream side and eventual merging of both forms as a single large dune.

Fence exposition in relation to wind direction also controls the patterns of sand entrapment. For an upstream inclination of the fence, the dune grows centrally above the fence, while for a downstream-deflected fence the principal acceleration area grows behind the fence.

Hotta et al (1987) assessed the efficiency of sand entrapment for various types of fences. Their conclusions are based on measurements with natural as well as various man-made materials metallic scraps, plastics etc. Small-permeability fences made from natural materials such as fern, reed, grass, wicker and even narrow strips of timber are found to be more effective than fences of identical permeability and large clearances.

The efficiency tests of synthetic fabrics (meshes) having different permeabilities (40, 48, 55, 68, 82 and 86 %) have shown lower efficiencies with increasing permeability (small quantities of sand were entrapped for permeabilities above 80 %) - cf. Fig.6.10.

Hotta et al (1987) also provide a method for computations of fence entrapment rates. It is assumed that the entrapment is primarily controlled by the distribution

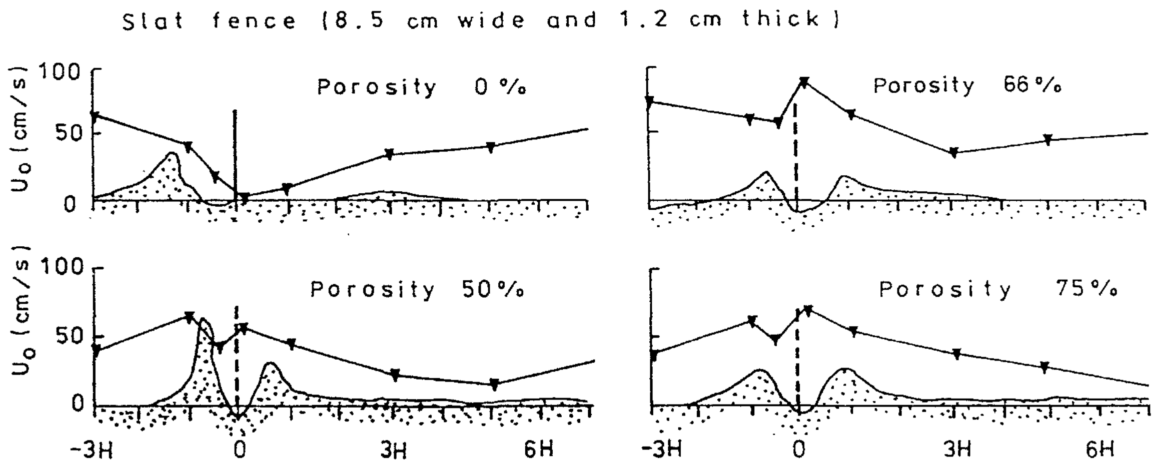


Figure 6.11. Friction Velocity and Changes in Beach Topography after Hotta, Kraus and Horikawa (1987).

of friction velocity across beach. The friction velocity u_* can be found from the logarithmic profile of wind speed u_z

$$u_z = 5.75 u_* log_{10}(z/z_o) \tag{6.1}$$

in which
z_o = roughness height.

The aeolian transport rate is taken after Kawamura (1951), cf. Horikawa, Hotta & Kraus (1986):

$$q = k\frac{\rho_a}{g}(u_* + u_{*c})^2(u_* - u_{*c}) \tag{6.2}$$

in which
q = sand transport rate
k = empirical coefficient (about unity)
ρ_a = density of air
g = acceleration due to gravity
u_{*c} = friction velocity for incipient motion of sand

Using Kawamura's formula and knowing friction velocity distribution some authors computed sand quantities upstream and downstream of fence. Fig.6.11 illustrates such computations for dune fences of different permeabilities.

Sand entrapment about a dune fence is a complex phenomenon depending on fence parameters (permeability, inclination, height) and characteristics of sand and wind. Hotta et al (1987) conclude that:

I. inclination of the upstream slope of dune increases for higher wind speeds while the dune crest moves in the downstream direction

II. fences control the growth of dune up to a certain critical wind speed; sand is no longer entrapped above this speed

III. for a certain type of fence, the permeability of about 50 % is most effective

IV. neither plastics nor metallic scraps are recommended; the best materials should easily decay under a cover of wet sand.

In Poland, permeable fences protruding 70 to 100 cm above ground, made from timber scraps, fascine or reed pierced in vertical rows to a depth of 30 to 40 cm have proved most effective. More and more common are also fences incorporating fairly dense plastic nets spred between vertical timber poles spaced every 5 metres. The fences are arranged in parallel rows spaced 1.5 to 2 m, oriented normally to the prevailing wind, sometimes parallel to shoreline. The rows are interconnected with perpendicular fences placed every 10 meters. Upon fence burial new fences are constructed on top of the dune as long as needed to reach the desired dune height.

Fences operate well if sand is supplied from a wide (at least 20 m), dry sandy beach. Under moderate climatic conditions, such as those in Poland construction should be initiated in spring to allow for a sufficiently long period of entrapment. Dune may be stabilized by covering with seaweed, dry twigs or fascine.

Stabilizing fences, 30 to 40 cm high, with rod spacing equal to four reed diameters are also in use. They are usually arranged in squares two by two metres.

6.2.3 *Artificial Seaweed and Grass*

In calmer marine environments one may endeavour different measures which simulate, reproduce or follow the natural patterns of sedimentation caused by water or wind, such as those in the preceding section. We will exemplify two approaches using artificial seaweed and grass.

An artifical seaweed system claimed to combat coastal erosion has been developed by Shell and Nicolon, a Dutch manufacturer of special fabrics for hydraulic engineering applications. The system has been tested at various locations on the Dutch coast in consultation with the Rijkswaterstaat, and it has been found that for some applications considerable economies can be made over the cost of traditional methods of coast protection and bottom stabilization, Bakker et al (1973).

If synthetic tapes or yearns with a density of less than water can be suitably anchored to the sea bottom, the flow rate of the water near the bottom will be reduced, giving a degree of control over the migration of bottom material. Consequently, erosion of the bottom can be suppressed and deposition of suspended solids is enhanced in a relatively simple and inexpensive way, thus adding to design value.

The Shell/Nicolon system is based on stretched polypropylene (PP) foam made via an extruder-gassing system. The following properties render the material eminently suitable for artificial seaweed;
- Low density giving high buoyancy.
- High yarn strength, resulting in a long lifetime under water and trouble-free handing and processing of the yarn.
- In spite of the open structure, there is little penetration of water into the mterial due to its hydrophobic nature, surrounding sheath and the very low diffusion rate of water through the sheath. - Resistance to the influence of bacteria, fungi and most chemicals.

The strands should be arranged side by side, thus forming a kind of screen. Such an arrangement is much more effective than, for instance, tufts of strands spaced at intervals, between which sturbulence can occur, with possible adverse effect.

Secondly, once installed under water, the strands in the screen must be able to movefreely and independently to ensure that not too much buoyancy-reducing foreign material is cought by the curtain.

Thirdly, the strands must be attached to an effective anchorage. Taking into account that in coastal-protection projects one is dealing with huge quantities of material, it is clear that the side-by-side arrangement of the weed threads and the attachment to the anchoring device must be simple and economic.

Typical characteristics of complete artificial seaweed screens are:
Length of weed strands 1 - 2 meters.
Width of anchoring tube attaching seam 5 cm.
Width of screen 1-several-hundreds of metres.
Number of weed standards per cm. ca. 4.
Basic weight of screen $70g/m^3$.
Diameter of anchoring tuble 15 cm.

Since laying of the weed, the test area has been regularly and carefully surveyed by echo-sounding. These measurements revealed that the bottom had become stable in, and close to, the weed field. From time to time sand accumulations of the order of 30-50 cm were measured but this sand usually disappeared again during heavy weather. Erosion, however, no longer takes place. This trial can therefore be considered successful.

Grass slopes or sods can often prevent moderate erosion, slow down run-up, help siltation, and generally develop favourable conditions of coastal protection.

The Delft Hydraulic Laboratory was commissioned to assess the stability of a grass dike by means of a full scale model study which was an absolute requirement as grass cannot be scalled down. Two investigations have benn performed.

In the Delta Flume, a five metre wide section of the grass dike was reproduced on full scale. The model consisted of a sand core covered with a clay layer on a slope 1 on 8 .
Sods of grass with the depth of the roots of approximately 40 cm were laid on top of the clay layer (the grass was taken from an existing dike that was reinforced ten years ago).

During the tests, the wave heights and periods and water levels (tidal cyclus) were varied continuously according to predetermined boundary conditions during the design storm surge. The maximum H_s was equal to 1.85 m with $T_p = 5.6\ s$ (plunging breaker falling on a water cushion).

The measured maximum velocity on the slope (1:8) was about 2 m/s. After 30 hours of continuous random wave attack the condition of the grass dike was still exceptional well. The surface erosion speed of clay protected by grass was not more than 1 mm per hour. In a number of additional tests, the durability of the grass and the enlargement of holes previously dug in the grass were studied. Although wave action considerably enlarged some of these holes, the residual strength of the dike was such that its collapse was far from imminence.

The second investigation was carried out in a large (site) flume on slope 1 on 4. Special equipment was used to simulate the run-up and run-down velocities on this slope. Two qualitatively different grass-mats on clay were used. The grass-mats were tested with the average velocity of 2 m/s (average over 40 hours of test) and the thickness of a water layer of about 0.6 m. The maximum velocity was about 4 m/s. Erosion speed of the clay surface was 1 to 2 mm per hour up to 20 hours depending on quality of grass-mat. After 20 hours of loading the erosion speed started to grow much progressively for a bad quality grass-mat. Similar process took place for a good quality grass-mat but after 40 hours of loading. The detailed information on the results and grass-mat specification can be found in DH reports.

Hence the results of the physical model tests carried out at Delft Hydraulics with respect to grass mats have proved suitability of the latter. In addition, it has also been indicated that if the block thickness was 1.5 times the opening size of the holes, sufficient material in the holes would remain.

6.2.4 *Low-cost and temporary sea walls*

In this chapter emphasis is placed on low cost and effectiveness of coastal structures. Accordingly, slightly modified terms may be used. The denominations "bulkhead" and "seawall" are often used interchangeably. In a strict sense, however, bulkheads are retaining walls whose primary purpose is to hold or prevent sliding of the soil while providing protection from light-to-moderate wave action. Sea walls, on the other hand, are structures whose primary purpose is to protect the backshore from heavy wave action. Their massive size generally places them beyond the low cost range. Also, they are not generally needed in sheltered weaters where large waves are not generated (except perhaps in the Great Lakes).

Bulkheads can be used to protect eroding bluffs by retaining soil at the toe, thereby increasing stability, or protecting the toe from erosion and undercutting. They are also used for reclamation where a fill is needed in advancement of the existing shore.

Finally, bulkheads are used for marinas and other structures, where deep water is needed directly at the shore.

Construction of a bulkhead does not secure stability of a bluff. If a bulkhead is placed at the toe of a high bluff steepened by erosion to the point of incipient failure, the bluff above the bulkhead may slide, burrying or moving the structure toward the water. To increase the chances of success, the bulkhead should be placed seaward of the bluff toe, and if possible, the bluff should be graded a flatter, more stable slope.

Bulkheads can be built as sheet pile, post supported or miscellaneous structures, cantilevered or anchored, while revetments about them may be classified as flexible, semirigid, or rigid. Flexible armors, such as quarrystone, riprap, or gabions, retain their protective quantities even of the structure is severely distorted, such as when the underlying soil settles, or scour causes the toe of the revetment to sink. A semi-rigid armour layer, such as interlocking concrete blocks, can tolerate minor distortion, but the blocks may be displaced if they are moved too far to remain locked to the surrounding units. Once one unit is completely displaced, such revetments have little reserve strength and displacement will generally continue to complete failure. Rigid structures may be damaged and fail completely if subjected to differential settlement or the less of support by underlying soil. Grout-filled mattresses of synthetic fabric and reinforced concrete slabs are examples of rigid structures.

In the following we discuss various more or less non-conventional materials, methods and measures applied to bulkheads and revetments, although some references to the traditional implementations are unavoidable.

Treated timber in well-designed and built structures have long been recognized as viable and economical material for bulkhead construction, for both sheet piles and posts.

Horizontal, pressur-treated timber planks can be spiked to the landward side of posts which are anchored to deadmem or poles in the backfill. The planks must be backed by filter cloth or graded stone to prevent soil losses through the cracks. Riprap toe protection should be provided. Horizontal untreated logs can also be used occasionally.

Wooden seawalls are used for purposes similar to concrete seawalls, and may behave as bulkheads, holding back fill materials. Their effect on incoming waves is also similar to that of concrete walls, and they suffer from many of the same problems of undermining and overtopping. They are often cheaper to install than concrete, however, which propably accounts for their continued use.

Numerous designs for wood walls have been tried over the years, including the use of reilroad ties and steel H-beams as vertical mumbers. Wakefield piling bulkheads use vertical boards deeply embedded in the sand in a tongue-in-groove arrangement, and reinforced with one or more horizontal planks on their seaward forces. These walls were relatively common between the turn of the century and the 1950s. Currently, the most common design within the study region incorporates vertical wooden pilings six to eight feet apart embedded in the sand with horizontal boards (usually 3 or 4 inches by 12 inches in cross section) nailed or bolted to the lendward side of the pilings. In the last decade, such walls have also incorporated filter cloth behind the horizontal wooden planks or lagging, and tie-backs into the fill behind the wall.

The Army Corps of Engineers(1956) observed that wooden walls cppeared to be effective when new, but disintegrated rapidly. Their Low Cost Shore Protction study (1982), which dealswith more protected environments than the exposed Pacific const, found wooden walls to be most successful if treated properly to reduce rot, fastened together with corrosion-resistant bolts, and sealed from sand leakage using filter cloth or other means. Chemical treatment of wood walls is a critical necessity, but even pressure-treated wood loses its resistance to decay after about ten to twenty years of exposure to salt water and spray (Moffatt and Nichols, 1968, 1983). Where wooden walls are fronted by rip rap, even though some fill may erode, the planks often stay in place at levels below the top of the rip rap. Where rip rap in front of wood walls settles or has not been piled to the top of the walls in the first place, both walls and property may be damaged. In the more successful cases, the wooden wall may provide a better support for the rip rap than would fill alone, and the rip rap appears to provide protection from battering and scour for the wood wall.

Historical data indicates that even treated wooden walls more than twenty years old may appear sound, but are often internally week. They will continue to stand for decades, until a severe storm tests their strength, and then they may fail catastrophically.

Used rubber tyres can be strug over two rows of treated posts set in a staggered pattern, with the tyres abutting each other and filled with gravel (Fig.6.12). The posts can be tied back to logs buried in the backfill with filter cloth placed behind the tyres before backfilling. Under wave action, the gravel tends to wash out of the tyres, and the backfill can then escape. Although used tyres can generally be obtained free, the cost of the structure is probably comparable to other bulkheads because of the requires close port spacing.

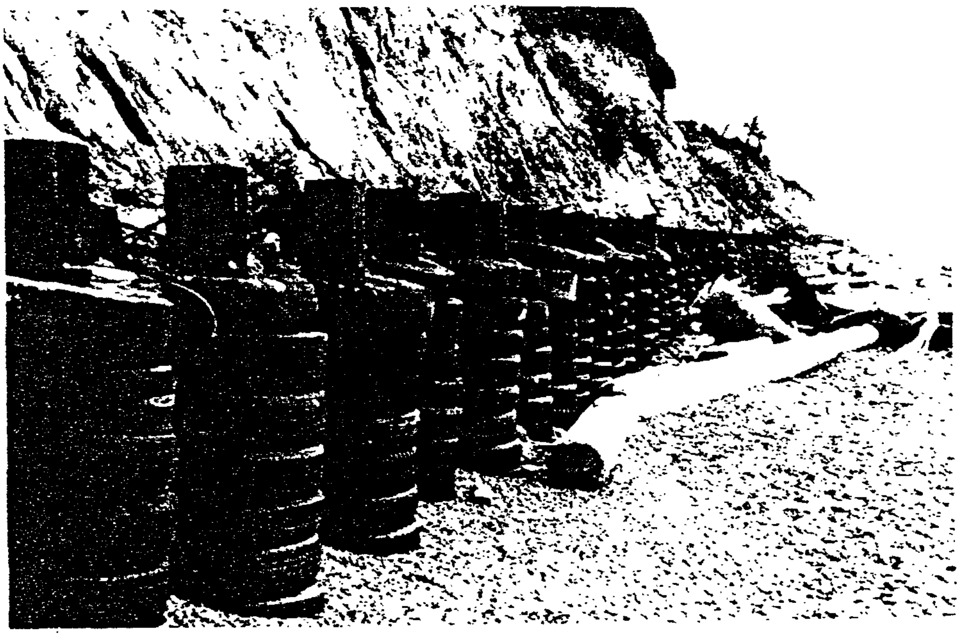


Figure 6.12. Used Rubber Tire and Post Bulkhead.

Because used tyres are readily available at most sites at no cost, many have tried to use them for shoreline protection devices. Bulkheads can made with scrap tyres interconnections (both vertically and horizontally) by galvanized spikes and push-nuts. The tyres are stacked in a staggered pattern over a filter cloth, and granular material is used both as backfill in low areas, and as fill in the tyres.

Three rows of galvanized steel anchors secured the structure to the beach. The structure progressively failed (US Army 1981) because the interconnections between the tyres were inadequate to hold it together. The gravel washed out of the tyres, evantually allowing them to be lifted by waves. This system is not recommended in view of better and less costly alternatives.

This structure illustrates a common problem with using scrap tyres. While their availability is a strong temptation to use them in shore protection devices, tyres are extremely rugged, and usually cannot be securely fastened together expect by considerable lavor and expense. In almost all cases, failure results because interconnections do not perform as expected.

Three rows of posts with scrap tires have been arranged as a sea wall at the settlement of Lesnoye on the Soviet Baltic Sea (Kurshskiy Spit). The prestressed-concrete piles, 35 x 35 cm, were driven 3.5 m below beach and protruded 2.5 m above it, and spaced every 80 cm along a 200-m strip of shore. The three-row wall was positioned originally eight metres from the protected dune. The rate of dune erosionhas been claimed to fall from $10m^3$/yr to $5.6m^3$/yr (Lashchenkov & Rabkova, 1987).

Used tyres may be used for revetments. Under a study conducted by Svensson

& Sweeney (1979), eight scrap tyre revetments have been installed to combat different erosion control problems. Each revetment had a different arrangement of tyres bolted together and secured to a shoreline or streambank in such a fashion as to reduce erosion by waves, drainage, and/or littoral currents. The revetment design was dependent upon such factors as the nature and severity of wave activity, soil composition, slope of the existing bank, and amount of hydraulic loading. One primary advantage of a tyre revetment over conventional methods of shoreline protection, such as rip-rap or sheet piling is the cost. [1].

Three sites on the Black Rock Channel were completed in 1978. The shoreline of the Channel, which is major shipping lane, is subjected to waves induced by vessels up to 460 feet in length. This site had been previously rip-rapped in 1950; and portion of the stacked cement had toppled and eroded to such an extent as to leave the shore unprotected. The primary consideration at these sites was wave attack: consequently multilayered structures were utilized. The tyres were laid down horizontally and bolted side-to-side and top-to-bottom, to form a structure four tyres high and four-five tyres wide. Steel rods, six feet in length, were driven in every fourth or fifth tyre.

The finished structure was covered with backfill and seeded with a mixture of long-rooted grasses to anchor to soil tyres. With such a structure, the impact of waves would be absorbed and dissipated through the structure due to its elasticity and interlocking construction. The structure is extremely porous, which allows runoff water to pass through and, thereby, reduces hydraulic loading. A disadvantage is that some backfill may be needed to compensate for soil which may wash off the top of the structure.

The fourth site was at the top of a 30-foot bank on Cayuga Creek, in Cheektowaga, New York. The primary problem at this location was bank slumpage, with accompanying loss of trees and other vegetation. The finished area is, as indicated, by most opinions, more aesthetically pleasing than the more commonly used methods of bank stabilization.

The next three sites were completed on Ellicott Creek, which is a stream with medium flow and bank erosion problems.

The last revetment site completed was at Wendt Beach, on the lake Erie shoreline in Angola, New York. This revetment was placed at the base of a sandy bluff which was eroding and exposing the roots of a 40-foot red oak tree. The primary objective was to save the three. A structure was designed which, when viewed from the side, was triangular in shape. Sections of this structure were prefabricted and bolted in place in a 100-foot semicircle at the base of the tree. Stakes were driven in at other sites and backfilling over the structure was initiated. This structure has been designed to absorb wave impact and trap sand. While the structure was under

[1]The latter may cost upwards of $ 1,000 per linear foot, while a tyre revetment, which may be equally effective in erosion prevention may be installed for as little as $ 50 per linear foot

construction, a storm hit the area and the structure effectively trapped from 6 to 10 inches of sand.

In summary, tyre revetment should provide an inexpensive yet effective means of erosion control. The application of these structures may be used successfully in a variety of situations, although severe storms may pose a threat to the stability, and even existence, of the used tyre revetment.

SAND BAGS

Sand bag is a generic name given in coastal engineering to all cloth or plastic bags filled with sand, mortar, grout, concrete etc. to protect terrace deposits, cliffs, bluffs, dunes etc. Their more elaborate version, sand sausage, is discussed separately for it occupies an important place among the modern low-cost sea defence measures.

Hogwire fencing attached to posts can be used to support sand bags stacked on the landward side of the fence to form a relatively inexpensive structure. The sand bags are vulnerable to rearing, however, if after being undercut by toe scour, they slide against the hogwire fencing.

For best peformance, use small-mesh wire with a PVC coating, because bare wire fencing tends to cut the bags. Tearing of the front row of bags can be prevented by filling them with a sand- cement mixture. Burlap bags can be substituted for the more expensive bags when a sand-cement mixture is used. The material and seams of all sand-filled bags must be resistant to ultraviolet light.

Place the bottom bags and fencing in a trench excavated to at least the depth of anticipated toe scour. Anchor or brace the posts, or embod them deeply, allowing for less of support because of toe scour. Provide adeuate drainage of the retained embankment and place stone at the toe of the bulkhead.

Several manufactures produce bags and mats in various sizes and fabrics that are commonly filled with either sand or a lean mixture of concrete for use in revetments. While no special equipment is required for sand-filled units, a mixer, and possibly a pump, are needed for concrete-filled units. Bags should be filled and stacked against a prepared slope with their long axes parallel to the shoreline and joints offset as in brickwork. Grout-filled bags can be further stabilized by steel rods driven through the bags.

The advantage of a bag revetment is its ease of construction andmoderate cost. Sand-filled bags arerelatively flexible and can be repaired if some the original bags are dislodged. In addition, stacked bags are suitable as temporary emergency protection measures. Among their disadvantages, they are limited to low energy areas, have a relatively short service life compared to other revetments, and generally have an unattractive appearance. Since concrete-filled structures are rigid, any movement or distortion from differential esttlement of the subgrade can cause a major failure that would be hard to repair. Sand-filled bags are highly suceptible to damage and possible failure from vandalism, impact by water-borne debrie and determination of material and seams by sunlight. The smooth, rounded contours of bags also present an interlocking problem and they should be kept flatter and underfilled for stability.

Grout-filled mattresses are designed to be laid flat on a prepared slope, joined together, and then filled. They form a large mass of pillow-like concrete sections with regularly spaced filter meshes for the passage of water. They should alawys be installed according to the manufacturer's recommendations.

Bags or mattresses should be placed only on a stable slope. While a stacked bag revetment can be placed on a steeper slope than a mattress, it should not exceed 1 vertical on 1.5 horizontal. A stacked bag revetment should be at least two bags thick, preferably with the outside layer concrte-filled and the interior bags sand-filled. When sand is used as filler material, the bag or mat fabric, and its seams, must be resistant to ultraviolet light. Where vandalism or water-borne debris are likely, only concrete-filled units should be used.

Some form of toe protection should be provided, or the toe should be buried well below the anticipated scour depth. Also, an adequate filter system, such as a properly installed and sized filter cloth, should be installed.

Some types of bags and mats which have been used in the past are described below (US Army 1981).

Burlap bags are recommended only when filled with concrete because of rapid deterioration in the shoreline environment and the ease with which they can be torn.

Sand Pillows are ultraviolet-resistant bags made from a woven acrylic fabric. They weigh approximately 100 pounds when filled. Bacause of their resistance to sunlight, they are suitable for sand-filling in some areas.

Dura Bags are large ($4 \times 12 \times 1.7 feet$), and must be filled in place using a pumped sand-slurry or concrete. Their large size makes them more resistant to movement under wave attack. Fabricated of ultraviolet-resistant material, they can be used in installations exposed to sunlight.

Fabriform is designed to be filled with a highly fluid, lean-cement mixture. the exterior cloth envelope serves primarily as a form until the grout hardens. Fabriform is a patened procuct, available in several fabric styles, including some with filter points (weep holes) to provide slope drainage. Fabriform mats should be installed according to the manufacturer's instructions.

Concrete-filled sacks placed over steel reinforcing rods have been used to protect terrace deposits and sea cliffs. Some of these walls have been outflanked, in part bacause they allowed water to collect and then run off to either side, even where shallow flanking wall used. Surface runoff from streets and sidewalks combined with water from overtopping waves to cause these problems. Several walls of this type failed completely under heavy storms, other were threatened. Those which were not exposed tofrequent wave splash did not suffer from severe outflanking problems. The stability of the foundation material beneath these walls is also an important factor in their survival.

6.2.5 *Engineered and Non-Engineered Revetments*

Engineered revetments embody not only the aforementioned materials and construction elements, arranged in an orderly manner (in contrast to disorderly non-engineered revetments, just dumped on beach a at seacliffs). On the contrary, they primarily incorporate carefully placed layers of different sizes of rock, excavated foundations or keyways, and/or filter cloth, and have been used with increasing frequency for small-scale protective structures over the last decade.

Stone revetments are constructed of either two or more layers of uniform-sized pieces (quarrystone) or a gradation of sizes between upper and lower limits (riprap). Stone revetments are a proven method of shoreline protection. Given bellow are examples of some other engineered revetment implementations.

Concrete blocks for semi-rigid armor layer are designed with various intermeshing or interlocking features, and many of the units patented. Blocks have the average of a neat, uniform appearance. Many units are light enough to be installed by hand once the slope has been prepared. The disadvantage of concrete blocks is that the interlocking feature between units must be maintained. Once one block is lost, other units soon dislodge, and complete failure may result.

Gobi (Erco) blocks are patented units that weigh about 13 pounds each. Erco blocks are similar but they are offered by a different licensed manugacturer. Jumbo blocks are large-size Erco blocks that weight about 105 pounds each. The units are designed for hand-placement on a filter cloth or they are factory glued to carrier stips of filter cloth.

Turfblocks are designed for hand placement on a filter with the long axis parallel to the shoreline. Each block measures 16 $\times$24 $\times$ 4.5 inches and weighs approximately 100 pounds. Filed installations have not yielded conclusive results, but their performance should be similar to Jumbo Ercoblocks.

Nami Ring is a patented concrete block shaped like a short section of concrete pile, 2.5 feet in diameter by l-foot high, and weighing 240 pounds. The rings are placed, side-by-side, on a slope over filter cloth. Better percormance has been observed when the rings are jointed together with tie rods.

Control blocks come in various sizes and are similar to standard concrete construction blocks, except that protrusions in the block ends provide a tongue-and groove interlock between units. Designed to be hand-placed on a filter cloth with the cells vertical, the blocks can be aligned with their long axes parallel to shore, but optimum performance probably results from placement perpendicular to the water's edge.

Standard construction masonry blocks should be hand-placed on a filter cloth with their long axes perpendicular to the shoreline and the hollows vertical. Their general availibility is a primary advantage, but they are highly susceptible to theft.

Shiplap blocks are formed by joining standard concrete patio blocks with an epoxy adhesive. AT 100 pounds or more per unit, they are designed for hand placement on a filter.

Lock-Gard blocks join together using a tongue-and-groove system. the 80-pound, patented units are designed to be hand-placed on a filter, their long axes perpendicular to the shoreline. Since a Lok-Gard revetment has a smooth surface, increased runup heights must be considered in the design.

Terrafix blocks are patented units that join together with a mortise and tenon system, and have two cone-shaped projections which fit holes in the bottom of the adjacent block.

The uppermost layer of riprap, either at shoreline or on deeper water, where it serves as a structural element of breakwaters, jetties, dikes etc, often consists of heavy man-made blocks such as tetrapods, tribars, doloses, quadripods, etc.

Their principal task is to provide stability of the structure under direct impact of waves and to dissipate wave energy. They are not treated here in detail since they are heavy and costly, and therefore do not fall into the categories dealt with in this chapter.

Revetments may be permeable or impervious, and rough or smooth. Bituminous materials and gunnite are among those intended for smooth impervious revetments. The former are becoming increasingly widespread (e.g. in Denmark, Belgium and the Netherlands) for lightweight coastal protection structure with elastoviscous properties. Garderen and Mulders (1983) distinguish the following three groups of bituminous materials: overfield, exactly filled, and underfilled. The first group includes substances with a high percentage of asphalt thus highly elastoviscous while the third group is characterized by predominance of aggregates. The thickness of a bituminous layer on a 1 : 2 slope varies from 0.1 to 0.35 m, depending on the wave impact.

Gunnite consists of viscous concrete sprayed or trowelled over a steel mesh and/or reinforcing rod framework, and is oftem laid over relatively steep, erodible slopes. It has most commonly been used to protect terrace deposits and erodible bedrock from erosion by wave splash and spray, as well as by surface runoff. The coastal protection literature seldom mentions gunnite, probably bacause it is not viewed as a stable long-term solution to erosion problems. Hawever, with continued maintanance and patching, gunnite at some sites has lasted for 10 to 20 years.

There are still many problems, however, in adhering gunnite to weak, loose, powdery materials, such as terrace deposits, where detachment can occur.

One can take three important recommmendations for gunnite in coastal areas:
(1) Extend the coverge as far below low tide as possible;
(2) Us a carefully engineered system of tie-rods and wire mesh to anchor the gunnite in loose materials;
(3) Install subsurface cutt-off walls and/or drains to keep water from undermining the concrete facing or exerting high hydrostatic presures.

Davis & Rutherford (1985) have successfully utilized epoxy-coated reinforcing bar with gunnite or shotcrete to prolong the life of a structure.
Non-engineered revetment are often constructed during emergency situations. An interlocing system of concrete or wooden rails, cribbing, is commonly used to support potentially unstable highway cuts. Moffatt & Nichols (1983) feel that selected concrete rubble is acceptable as an under-layer beneath armor stone.

A concrete rubble revetment utilizes a waste procuct that is otherwise difficult to disposes of in an environmentally acceptable manner. The concrete should have the durability to resist abrasion by water-borne debris and ice pressure. In addition, all protruding reinforcing bars should be burned off prior to placement. Numerous concrete rubble revetments have failed in the past, but this is attributted to neglect of filter requirements.
Gabions may be classified as a transition between engineered and non-engineered revetments.

Gabions are rectangular baskets or mattresses made of galvanized, and sometimes PVC-coated, steel wire, in a hexagonal mesh. Subdivided into approximately equal sized cells, standard gabion baskets are 3 feet wide, and available in length of 6 and 12 feet and height of 1, 1.5 and 3 feet. Mattresses are either 9 or 12 inches thick. 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 4- to 8-inch diameter stone. 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.
Miscellaneous bulkheads and revetments include Longard tubes, gabions and steel fuel barrels. Longard tubes are patented, woven, polyethylene tubes, filled with sand at installation and available in 40- and 69-inch diameters, and lengths up to 328 feet. Like sand-filled bags, performance depends on the fabric remaining intact, and the tube completely filled. When filled, the tube is dense and heavy yet flexible enough to settle if depressions occur. A properly installed Longard tube is

greater than those of bulkheads and revetments, most of the low-cost measures, methods and materials utilized close to shoreline may also be used further offshore. They are reviewed below.

The general tendency in groyne construction has recently shifted towards cheaper solutions. Heavy groynes made from stone rubble or concrete blocks, often in bulky implementations between two rows of piles, have been gradually replaced by lighter permeable single-row palisades, frequently with their permeability growing in the downstream direction (i.e. the piles in downstream rows being spaced more widely than the upstream ones). This tendency is common for many European countries. The practice in the Northerlands (Bakker et al, 1984) shows that a permeable groyne system, consisting of single or double rows of timber piles, perpendicular to beach, without bottom protection, costs only 10 to 25 percent of the expenditure on the impermeable stone groynes which were used in the Netherlands (and elsewhere) for centuries.

Model experiments (Bakker et al. 1984) confirm that wave-induced currents in the areas protected by permeable groynes are reduced to 65 %, and tidal currents even to 50 %, depending on the pile screen configuration. Prototype measurements could not lead to straightforward conclusions with statistical significance: the effect of the pile screens on beach evolution is partly dissolved in natural fluctuations and trends. Wooden pile screens do not prevent the shoreward motion of tidal channels, which can cause washing out of piles. Furthermore, constructional failures, which in the future can be avoided, at some places resulted in negative experience. It is concluded that permeable pile screens deserve serious consideration as a first flexible and cheap phase in combating coastal erosion. Its application however should be based on a thorough analysis of the local coastal current climate.

Timber is used in sheet piling, post piles, and various structural elements.

Sheet pile groynes, an old and proven means of shore protection, can be constructed of timber, steel, or aluminium sheeting. Toe protection or adequate embedment is required to insure the structure's stability. The general recommendations fiven for sheet pile bulkheads also apply to groynes.

Timber piles can i.a. be driven into the bottom, so that every three form a triangular pattern, and used automobile tyres can then be stacked on the piles. Just above the top tyres, the triangularly grouped piles should be interconnected using 2 × 6-inch planks bolted to the piles. The structure, whose stability depends on the depth of pile penetration, has proven effective against mild wave action.

A brush breakwater is constructed of two parallel rows of posts deiven into the offshore bottom, connected across the top with timber crossties, and filled with brush. Brush should be cut longer than the space between the posts and placed parallel to the structure alinement. Not suitable for permenent protection, this breakwater can be used for temporary sheltering of young vegetation.

Used tyres have just been mentioned for the timber pile arrangements. They can also be used as a buoyant material in floating breakwaters.

Scrap tyres can be used to create artificial reefs capable of protecting shores. A study by Stone at al (1974) summarizes
(1) methods of assembling into units that would be inexpensive, easy to assemble and handle and effective
(2) the cost, both to build the units and to transport them to thereef site
(3) how these reefs function in the marine environment
(4) the effect of reefs on angling success in the New York Bight
(5) the number of reefs that could be established, and the potential number of scrop tyres that can be used effectively in the marine environment.
Sand bags are used for both groynes and breakwaters. Sand-filled bag breakwaters are constructed of stacked bags a staggered pattern. The integrity of the structure depends on the individual bags remaining in place and intact. The bags and seams must be resistant to ultraviolet light to preclude deterioration from prolonged sunlight exposure. They should not be used where vandalism is expected or where the structure will be exposed to water-borne debris. Lighter bags (100-pound range), like those used for revetments, are displacement when exposed to even moderate waves. Larger units, such as Dura Bags, are recommended even though they are more difficult to handle and require filling in place. A filter cloth should be placed under the bags to reduce settlement in soft bottoms.

Submerged breakwaters built on sand-filled nylon bags on a sand bed, subjected to progressively more severe wave conditions were investigated by Ray (1971) in full-scale laboratory tests.

Properties of the bags, of effects of wave action on placement of bags, and the performance of bags and structure were determined for various combinations and wave conditions. The change in the sand bed at base of the structure and the wave attenuation by the breakwaters have also been assessed.

6.2.7 *Breakwaters*

Floating breakwaters can be constructed of virtually any buoyant material such as rubber tyres, logs, timbers and hollow concrete modules. Floating breakwaters are particularly adventageous where offshore slopes are steep and fixed breakwaters would be expensive because of deep water. They can also be used where the tidal range is large and fixed breakwaters would be subjected to widely varying degrees of submergence. Floating breakwaters are excellent for temporary installations, such as where vegetation requires protection while becoming established.

Floating breakwaters have several disadvantages as well. They are effective only against short-period waves (less than five seconds), which are those most commonly present in sheltered locations where low cost protection is most appropriate. Also, they may regarded as eyesores in some areas, they tend to collect floating debris, and they may require more maintenance than fixed breakwaters usually do.

Discarded tyres as a construction material for low-cost shore and harbour protection are examined experimentally by Amstrong & Petersen (1978) for an 18-tyre module. Engineering design paremeter, material cost and filed tests for both floating tyre breakwater (FTB) and tyre revetments are summarized. FTB are found to provide wave attenuation in marinas and small harbours in both salt and freshwater.

Two possible arangements of rubber-tyre floating breakwaters are shown in Fig.6.14. The upper configuration, known as a Wave-Maze, is patented and cannot be used without payment of royalties. The bottom configuration was developed by the Goodyear Tyre and Rubber Company for promotion purposes and may be used without royalties. The use of other configurations is limited only by imagination of the designer.

Increased depth of penetration in the water column increases the effectivenes of floating breakwaters. In general, the draft should be greater than one-half the design wave height. Two-layer structures or the use of truck or tractor tires will achieve greater draft.

The air trapped within the top of vertical tyres provides sufficient floation in most cases. In quiet water, the air is eventually dissolved by the surrounding water and the structure sinks. Wave action, however, replenishes the air supply, but care must be taken not to use tyres with puncture holes. More permanent floatation is possible with styrofoam blocks or foam injected into the crowns of the tires. In salt water, marine growth that is not periodically removed will eventually sink the structure. Sand also collects in the tyres and can sink them, but this can be prevented by drilling holes in the bottoms of the tyres.

Floating tyre breakwaters must be securely anchored to prevent displacement. Driven piles are generally the best means of stable anchorage over long periods.

Tyres have the property of elasticity: they are capable of deforming up to 30% without premanent change in shape. Due to the relative inertness of tyres, internal structural breakdown is slow. Therefore, they are capable of maintaining strength and flexibility characteristics over long periods of time, even under unfavorable conditions. The capacity of the tire to deform under large forces gives it potentially greater dissipative capabilities than more rigid materials. Erosion can be substantially reduced if the majority of the attacking wave energy can be dissipated before reaching the shoreline.

Tire structures used to attenuate wave action fall into two categories. Floating tire breakwaters have been proposed and built in locations offshore to attenuate incoming waves. Revetment type structures located directly on the shoreline have been built to dissipate wave energy remaining at the shore.
The fundamental characteristic of the FTB (floating Fig.1 fire breakwater) is ex-

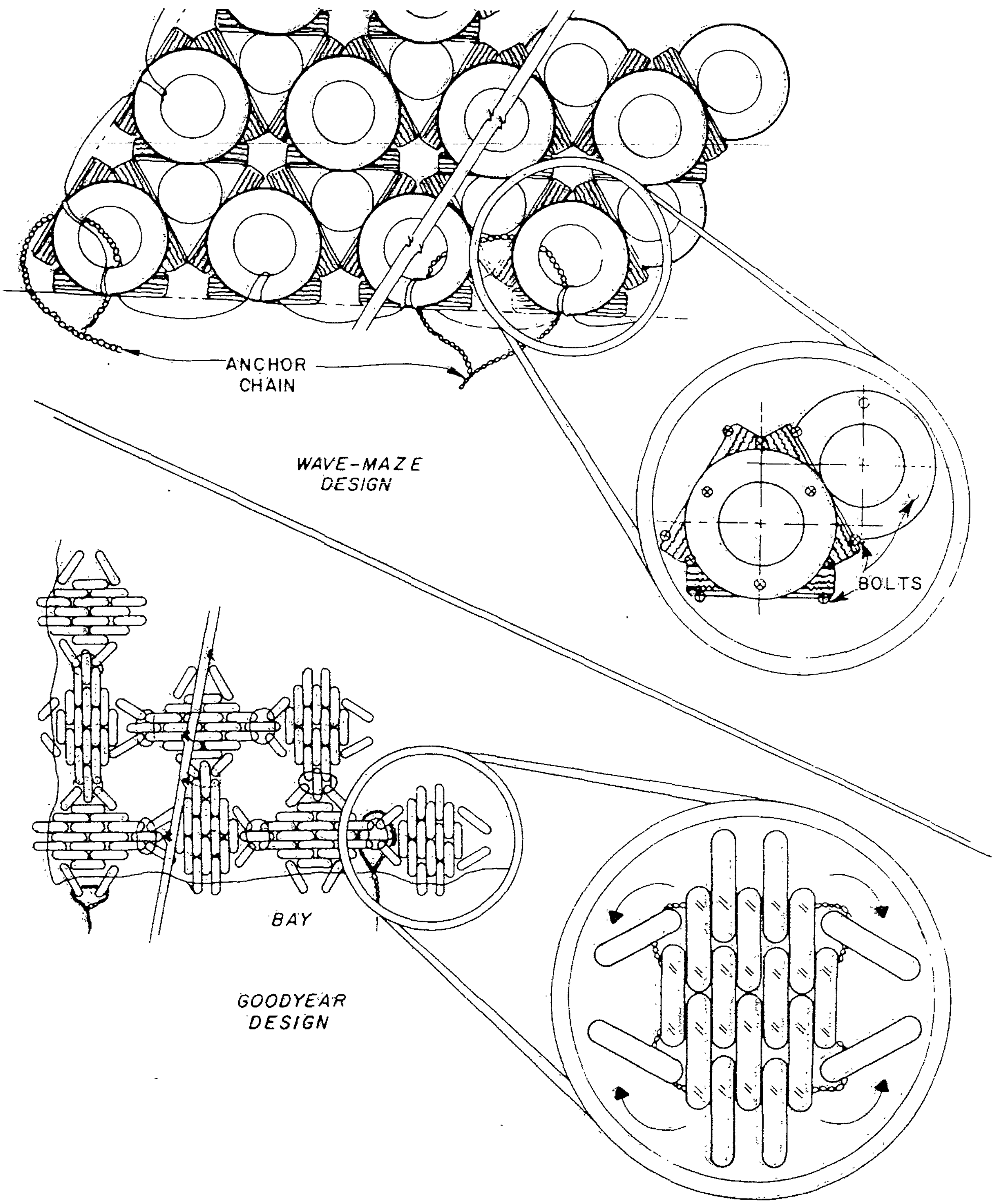


Figure 6.14. Floating Tyre Breakwater (FTB) Design Patterns.

hibited through the ratio of energy dissipated to energy reflected. The reflected energy is only between 7% - 20% as large as the dissipated energy, depending upon the relative wavelength $\frac{L}{L_B}$. The wave-transmission characteristics of FTB can be examined for various designs and patterns Harms (1979).
A fixed vertical plate simulating a wave barrier constructed of siding mounted on a pile structure is considered, versus a rigid horizontal plate fixed, and also floating. Comparison of the experimental data for the Goodyear FTB to that for the vertical plate indicates that a Goodyear FTB with beam size B=12D offers approximately the same level of wave attenuation as a fixed vertical plate of equal draft D. Comparison to the theoretical wave-transmission characteristics of the horizontal plates indicates that the performance of the Goodyear FTB lies approximately midway between that of the fixed and floating plate.

Width, depth, and length are important parameters in a floating breakwater effectiveness. In general, as width or depth, or both increase, the breakwater's wave attenuation characteristics increase. The span of shoreline in need of protection from wave attack will determine the length of the floating breakwater. The width (measured perpendicular to the shoreline) of the breakwater tested by Kowalski was greater than half the design wave length. These test breakwaters reduced 3-ft to 5-ft waves (0.9 m to 1.2 m) by 70 % - 85 % and smaller waves were almost completely attenuated, Amstrong and Petersen (1978).

Recent evaluation of tying materials for floating tire breakwaters yielded the following recommendations. Rubber conveyor belt edging fastened with nylon bolts received the highest ranking. Galvanized welded 1/2-in. diam or larger chain is second, but in a marine environment. The special 1/2-in. unwelded breakwater chain, produced by Campbell Chain Company, has survived several years in freshwater situations. Floating tire breakwaters' constant movement will not allow a protective layer of corrosion to build up on ungalvanized chain. A silted-in revetment mattress moves much less and would be a better environment for ungalvanized chain. Revetment situations where there is less movement of the tire bundles would allow a protective layer of corrosion to build up on the special 1/2-in. ungalvanized chain. Polypropelene line is available but must be protected from ultraviolet degradation and spliced as knots do not hold well. Wire cables are not recommended as the large surface area of the wire allows rapid corrosion, followed by failure. Natural and synthetic ropes are not recommended as they fray and fail rapidly in this abrasive environment, Amstrong & Petersen (1978).

To facilitate construction of large protection structures, either floating or fixed, Amstrong & Petersen (1978) suggested the 18-tire module. The tires are interconnected with high strength rope, cable, or chain, using appropriate fasteners. The fastening or connecting materials should be of suitable strength and corrosion resistance to withstand wave forces over the expected service life. Nylon rope, corrosion-resistant cable or chain, and formed steel rods have been examined, but each has certain operational limitations.

Once constructed, the modules are stronger than the individual tire. Due to the tire's flexibility characteristic, loads are distributed throughout the structure giving these modules up to 55,000 lb (24.948 kg) tensile breaking strength.

The modules then can be connected to construct the desired protection structure. To form a single-layer tire "mattress", modules are connected to each other by strapping material similar to that used in their assembly. Mattresses are built to meet length and width requirements specific to each site application. Additionally, the modules can be oriented in different configurations to provide the best preformance at a given site. These various configurations have yer to be studied to evaluate the arrangement that will give the best attenuation efficiency.

The modular concept provides distinct advantages in assembling the mattress. Tire modules can be assembled either in the vicinity of the designated wite or at a remote location. Labor and equipment usage cost may increase substantially if a remote assembly location is used. Most of the construction can be preformed prior to placement in the water. Work done in the water includes tying the modules together, locating the mattress in the desired position and securing it in place. Heavy construction equipment may be required should the tires become silted in before reaching their final position.

The modules can be and should be built in advance of the construction of the mattress. Completing module assembly before starting mattress assembly will allow for rapid assembly of the mattress, which is important in order to avoid damage by storms during installation.

The following conclusions and design guidelines are given by Harms(1979):

1. Laboratory measurements indicate that FTB function predominantly as wave-energy dissipators, transforming into turbulence far more of the incident wave energy than they reflect.

2. Wave-transmission design curves were generated from measurements on one-eighth-scale and one-quarter-scale FTB models and found to be in good agreement with available full-scale data. Wave-attenuation characteristics of FTB may therefore be studied with confidence using model breakwaters constructed of flexible tires with diameters as small as 3.25 in. (83 mm). These results suggest that changes in Reynolds number were not of importance for the range of conditions considered. Consequently, for most practical considerations, the Froude scaling law remains appropriate for this problem. It should remain appropriate as long as the Reynolds number is sufficiently high to maintain the turbulent character of the energy-dissipation mechanism.

3. Peak-mooring-force design curves for the Goodyear FTB were generated from model data. Available full-scale data were found to agree well with these curves,

even though proper scaling of elastic properties and damping characteristics of the mooring system could not be assured.

4. Simple semi-empirical model was found to simulate measured wave-transmission characteristics for H/L-0.04 with sufficient precision to be useful for many engineering applications:

$$C_1 = \frac{H_1}{H} = exp(-0.84\frac{C_d}{P}\frac{L}{B}........(...) \tag{6.1}$$

in which
$C_d/P = 0.69$ and 1.22 for the Goodyear and Wave-Guard FTB, respectively.

5. For $D/d = 0.065$, H/L = 0.33 - 0.06 and $L/B > 0.8$, the peak mooring force per unit length of breakwater, F, can be determined from the empirically derived relationship

$$F/\gamma H_2 = k_o(L/B)^{-1}tanh(L/B) \tag{6.2}$$

in which
$k_o = 0.093$ and 0.15 for the Goodyear and Wave-Guard FTB, respectively. For small values of L/B this simplifies to

$$F = k_o \gamma H^2 \tag{6.3}$$

Other Designs

A submerged plane can be used as a cheap breakwater (Patarapanich, 1980). Theoretical results show that it is suitable for small depth to wave-length ratios. The optimum submergence is found experimentally about 0.1 to 0.15 where highest energy loss is achieved by wave breaking. The optimum plate length is between 0.6 to 0.8 of the wavelength above the plate. Patarapanich (1980) also presents design criteria for wave force and overturning moment exerted on the plate.

Engineered and non-engineered riprap is incorporated in many breakwaters and groynes. A stone breakwater is structurally similar to a stone revetment. A major advantage of a quarrystone breakwaters is that the structure does not necessarity fail when differential settlement occurs. Through the years, stone has been used for more breakwater construction than any other material. It istime-tested and can be quite economical if suitable rock is available locally, although relatively expensive stone has also been in various types of groynes.

Miscellaneous breakwaters and groynes incorporate Longard tubes, gabions, steel fuel barrels, and several other components.

The advantages and disadvantages of Longard tube in bulkheads generally apply to breakwaters. An added disadvantage is that the protective epoxy coating cannot be applied to wet tubes so that damages are more likely. Therefore, they should not be used where the tube may be exposed to vandalism or water-borne debris.

The tube should be installed over a layer of synthetic filter-cloth with factory-sewn, 10-inches Longard tubes on each edge to reduce the potential for failure due to toe or heel scour. Where a 69-inch tube cannot provide sufficient height, an alternate breakwaters system should be used.

The basic design considerations for gabion revetments also hold for breakwater and groynes. The wire mesh should be placement vertical characteristic-coated, the baskets should be tightly packed, and a filter cloth should be used beneath the structure to help control settlement. A gabion mat 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. Stones enclosed in nets were tested in layers underlain by a sand and gravel filter (Pillai and Verma, 1981). The results of laboratory comparisons for typical stone walls have shown good performance of stones in nets. Possibility of filure due to removal of the base material and dislocation of individual armour stones are considerably reduced. The quantity and size of stones required in thenets are smaller, thus decreasing the general cost.

The use of steel fuel barrels for construction is only economical in remote arctic areas where used barrels are readily available and have no other salvage value. Barrel groynes have worked well where littoral transport characteristics are suitable for shore stabilization with a low groyne.

Among other device one may mention Z-walls, which are patented structures with vertical reinforced-concrete panels set on edge in a zigzag fashion. The structure is designed for placement close to the shore.

6.2.8 *Sand Sausages*

General

Efficient defence schemes should rely on local scources of construction materials, such as sand, gravel and stone. The latter is most often used for construction of various rubble structures: breakwaters, groynes, headlands and parts of seawalls and revetments. Sometimes the construction materials are not cvailable locally, and their transportation cost from remote suppliers becomes quite considerble. a economic alternative of stone rubbles consists of sand bags, already mentioned in the earlier sections, or socalled sand sausages, filled with sand, often in combination with grout, mortar or concrete, cf. Silvester, 1978, 1985, 1986.

Sand sausages are elongated plastic (usually polyethylene) bags, tubes etc. of 3-mm thickness and strength about 3600 kgs per 1 lineal metre (Liu & Silvester, 1977), having diameters up to 1 m. The mixture of sand mortar or concrete is added under water.

The advantages of sand sausages are evident. All basic materials are available on site; it is merely the pumps delivering sand mixture to bags that are required as a equipment. The common transportation problems and costs are then avoided.

The few designed problems connected with the sand sausages include the selection of size and strength, proportion of sand, mortar and concrete, and general configuration of structural elements in the breakwater or headland. These problems are discussed by Silvester (1986), who also shows examples of adequate sand and cement mixture to fill the sausages.

An optimum configuration of sand sausages in a breakwater is provided by placing them at 45° with regard to the structure axis so as to increase the relative length and weight of the sausages. In order to relative or reduce the wave pressures gaps should be produced between individual sausages (Silvester, 1986).

The stability of coastal structures made from sand sausages was invested only in a few field cases, Silvester (1978), Hydro Delft (1982). The latter study presents general results for the wave impact on a circular shore (in island), which was protected by various methods. Stone rubble and gabions were tested along with sand sausages 3 m long and 1 m in diameter. The study shows that for a slope of 1 : 3 the maximum wave runup on the gabion revetment was 60 to 65% of the wave runup for the defence scheme using and sand sausages. Hence the efficiency of wave damping was heigher for gabions than for sand sausages.

Characteristics of plastics and various other materials used in low-coast shore protection, in designing sand sausages and in the construction of sublayers (materaces) are given by Heerten (1981) and Saathoff & Kohlhase (1987).

Silvester (1986) provides an estimate cost of a breakwater having the following cross-section: 8.73 m, height, 39.4 m base width, 7- m crown width and 6 m height (up to water level[2]).

Design

Liu & Silvester (1977) give a theoretical analysis of cross-section suitable for sand sausages and of respective stresses. Following the notation shown in Fig.6.15, the optimum shape of the lower part of the sand tube is given by the equation

$$X = \pm\{[b - 1E(k) - E(k,\phi)] - \frac{b_3}{b_1}[K(k) - F(k\phi)]\} \tag{6.4}$$

in which
$F(k,\phi)$ and $E(k,\phi)$ = elliptic integrals of first and second kind, respectively.
$k = \frac{\sqrt{b_1^2 - b_3^2}}{b_1}$ = modulus of elliptic integral
ϕ = amplitude of the integrals $F(k,\phi)$ and $F(k,\phi)$

[2]Such a breakwater made from sand sausages costs about 64% of a stone rubble breakwater and 83% of a limestone rubble breakwater.

$$\sin\phi = \frac{\sqrt{b_1^2 - y^2}}{\sqrt{2(b_1^2 - b_3^2)}} \tag{6.5}$$

The shape of the upper part is described by the equation

$$X = \pm\frac{b_3^2 - b_2^2}{\sqrt{2b_3^2 - b_2^2}}[F(k\phi) - K(k)] - \sqrt{2b_3^2 - b_2^2}[e(k,\phi) + E(k)] \tag{6.6}$$

in which

$$k = \frac{sqrt2(b_3^2 - b_2^2}{\sqrt{2b_3^2 - b_2^2}} \tag{6.7}$$

$$\sin\phi = \frac{\sqrt{(2b_3^2 - b_2^2)(y^2 - b_2^2)}}{\sqrt{2y^2(b_3^2 - b_2^2)}} \tag{6.8}$$

The tube perimeter s is found from

$$S = \frac{2(b_1^2 - b_3^2)}{b_1}[K(k_1) - F(k_1\phi) + \frac{2b_3^2 - b_2^2)}{\sqrt{2b_3^2 - b_2^2}} \cdot [F(k_2,\phi) - K(k_2)] + b' \tag{6.9}$$

in which

$$k_1 = \frac{\sqrt{2(b_1^2 - b_3^2)}}{b_1}; \tag{6.10}$$

$$k_2 = \frac{\sqrt{2(b_3^2 - b_2^2)}}{\sqrt{2b_3^2 - b_2^2}} \tag{6.11}$$

$$\sin\phi_1 = \sqrt{\frac{b_1^2 - y^2}{2b_1^2 - b_3^2}}; \quad \sin\phi_2 = \sqrt{\frac{(2b_3^2 - b_2^2)(y^2 - b_2^2)}{2y^2(b_3^2 - b_2^2)}} \tag{6.12}$$

b' = width of sand tube contact with ground.

For a given value of b, corresponding to the total depth of water (Fig.6.15) and the perimeter s, from the above equations one can compute the quantities b_2, b_3 and b' characteristing the optimum dimensions of the sand tube.

The tube stress can be found from the following formula given by Liu & Silvester (1977)

$$T = \frac{\gamma}{2}(b_1^2 - b_3^2) \tag{6.13}$$

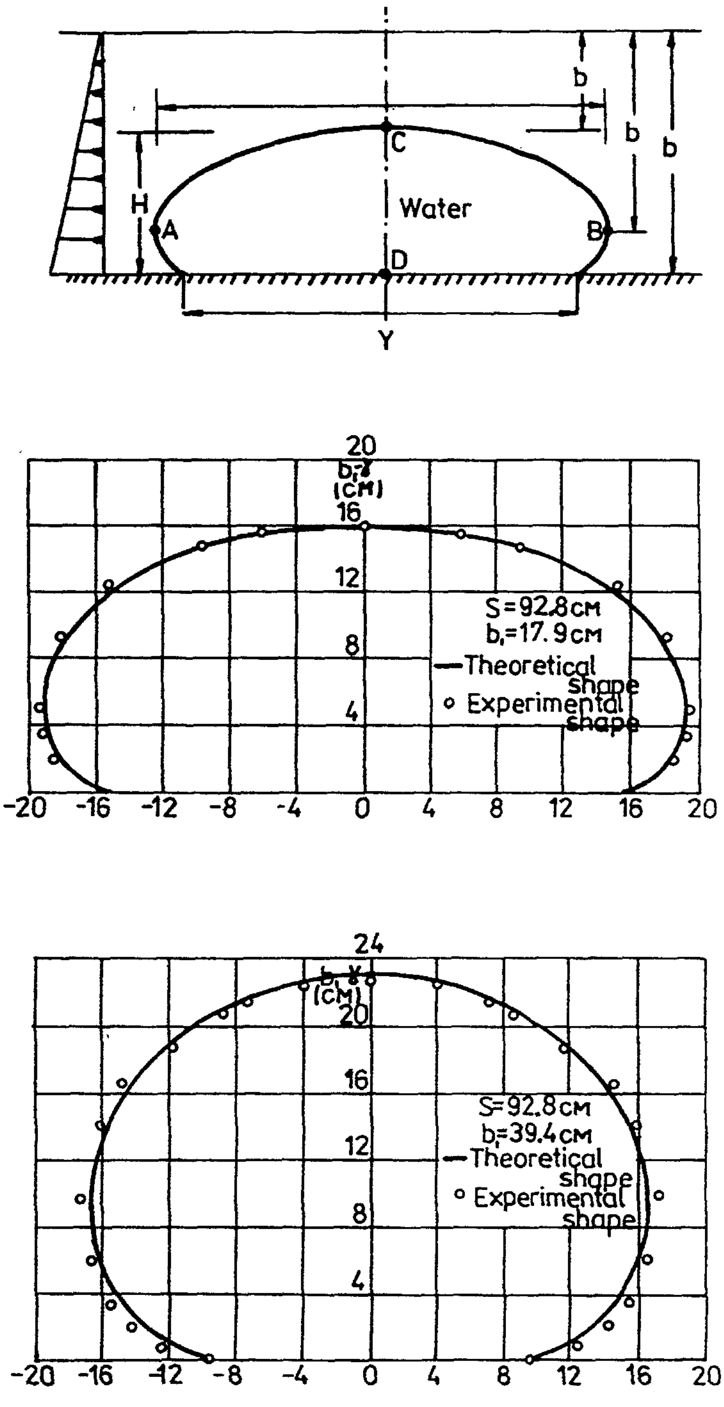


Figure 6.15. Notation (top) and Shape of Sand Sausage for Pressure Height of 17.9 cm (centre) and 39.4 cm (bottom).

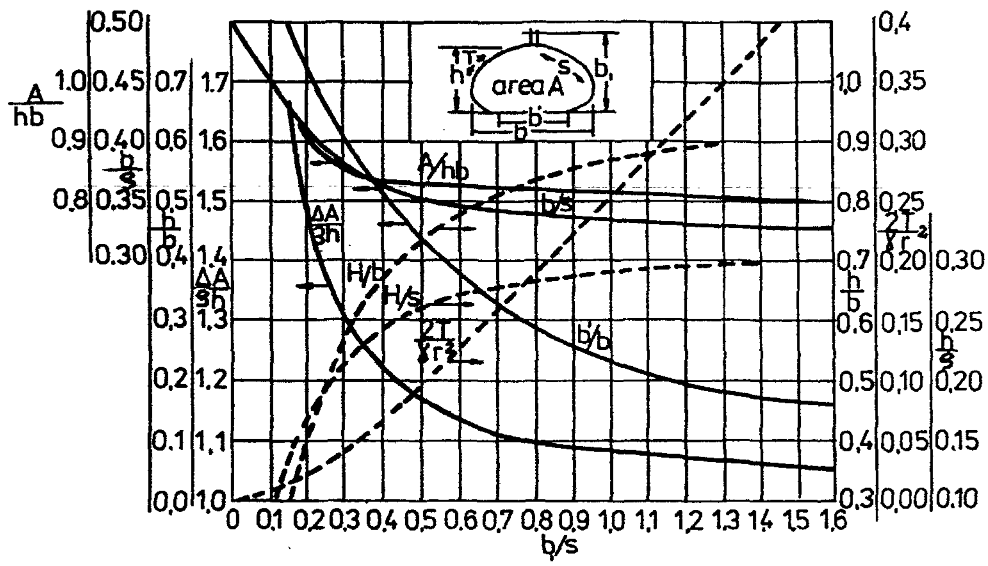


Figure 6.16. Characteristics of Mortar-Filled Sand Sausages (S.G. =2.0), Liu & Silvester (1977).

in which
$\gamma = \frac{p}{y}$,
p =pressure.

The experimental verification of the method proposed by Liu & Silvester (1977) shows a satisfactory agreement of theory and experiment.

The graphical form of the design equations for grout-filled tubes is depicted in Fig.6.16. For an initial value of b_1/s and the maximum height and length of the tube, H_B and L_B, being found grapically, one may determine the cross-section area and the contact length b'.

In summary, it should be emphasized that sand sausages offer on excellent low-cost alternative of coastal defence provided their materials are properly chosen and the sand to cement proportions are observed. To provide a sufficient stability, sand sausages must have enough contact with the bed. Wave thrust may be reduced by proper oblique placement of sausages and provision of pressure-relieving clearances at approximate levels. An additional advantage of sand sausages is that compared with heavy rigid structures, they make easier the control of wave reflection and scour.

6.2.9 *Geotextiles*

Coastal engineering was the starting point for the use of synthetic fabrics in geotechnics in the 1950s. Pioneers of coastal engineering first used synthetic fabrics to form huge bags for building groynes and closing dikes. Based on the advantages of these fabrics (low weight, high strength, long-term resistance) woven and, more recently, nonwoven fabrics have been of growing use in coastal engineering and, later, in waterway engineering, dam construction, road construction, railroad construction, tunnelling - all fields of geotechnical engineering.

In the 25 years of geotextile application many tests and investigastions have been carried out and many construction and design methods have changed owing to the growth in experience. Today we have the expertise for successful use of geotextiles in design and construction in coastal engineering applications (Heerten, 1984).

The following examples give a rough idea of geotextile applications in coastal engineering today:

I filters in erosion control structures as revetments of seadikes, seawalls or bottom protection structures, e.g. of sluices or storm surge barriers

II separator in the foundation of groynes and breakwaters item [III] fabric forms for sand filled bags or tubes as construction elements of groynes and dikes

IV flexible scour protection mats at different offshore and coastal structure.

Usually, in design drawings of coastal structures, a geotextile is only a line of black ink labelled "geotextile" or "synthetic filter cloth". But when selecting a suitable fabric for a special coastal engineering application the engineer has to learn that various types of fabrics are offered; woven, nonwoven and combinations, produced from different polymers. The properties of these fabrics are very different, influenced by the polymer properties and by the manufacturing process.

The following properties are important for geotextiles in coastal engineering:

I high filtering efficiency (soil-tightness and permeability);

II high robustness for safe installation (good resistance to puncture and to tear; sufficient stress/strain behaviour for the given installation method);

III sufficient long-term resistance to UV-degradation and to the marine environment;

IV high friction resistance with soil (on slopes)

Geotextile application in the harsh environment of coastal engineering is somewhat more of a challenge than the fabric uses in many other applications.

Entensive damage occurred, for example, to the bottom protection structure of a storm surge barrier at the Northfrisian coast, West Germany, after some years of service. After intensive investigations it could be shown that damage was not caused by insufficient longterm resistance of the polyamide multifilament woven fabric (200 g/m^2) but probably by unexpected instability problems in the armour layer and damage caused during construction by heavy bulldozers and excavators running on the gravel layers above the fabric. This example also underlines the demand for heavy weight fabrics in coastal engineering.

The most important parameters influencing the long-term resistance of fabrics in the salt-water regions of the North Sea (Heerten 1984) are ultraviolet irradiation and the raw material and fibre fineness of the fabric. Ageing by biological and chemical damage is of very low importance, but attention must be paid to ensure that there is no damage to fabrics during construction.
After geotextile and armour layer(s) have been carefully installed the successful service life of the structure depends mainly on the filtration properties of the fabric. The traditionally used filter materials such as sand and gravel are designed by the well known filtration rules, e.g. from Terzaghi of the US Corps of Engineers.

In discussing the filtraton properties of geotextiles we have to distinguish between the properties of woven and nonwoven fabrics. The filtration properties of woven fabrics are given by the mesh size or the fabric openings. The woven geotextile acts as a thin sieve. The filter conditions could be stable, with nearly all soil particles being larger than the mesh size, or unstable, with nearly all soil particles being smaller than the mesh size. Unstable conditions are often found on subsoil in the range from silty sands to clay and are found in most polders.

The filtration properties of nonwoven geotextiles are incluenced by the fibre size, the fabric weight and thickness.

The estimated permeability of the clogged geotextiles remained 5 to 12 times higher than the measured soil permeability, which is in the range of $k1.0to5.0x10^{-5}$ m/s. The remaining porosity of n' (0.32 to 0.74) guarantees an adequate long-term permability. In contrast to these results, for most of the investigated woven fabrics a lower permeability, compared to that measured in the soil, was estimated (Heerten, 1984).

To avoid permeability problems the use of thick needle-punched nonwoven geotextiles could be recommended in relation to the results of the investigations on dug-up fabrics.

Fig.6.17 shows the groyne cross-section with the foundation base above tidal low water using the new construction method as follows. On a heavy weight needle-punched nonwoven fabric (1100 g/m^2) the rip-rap core of the groyne is dumped. With an overlap of approximately 500 mm on both sides of the groyne a special scour protection mat is installed. Instability problems caused by scouring at the sides in the past led to crest sinking and damage of the groynes. By fixing the upper stones of the groyne core by grouting with concrete mortar an additional demand for high stability arises.

The filter mat in the foundation base of the groyne has to fulfil the tough requirements on geotextiles for hydraulic engineering.

The scour protecton mat is composed of four parts (Fig.6.17):

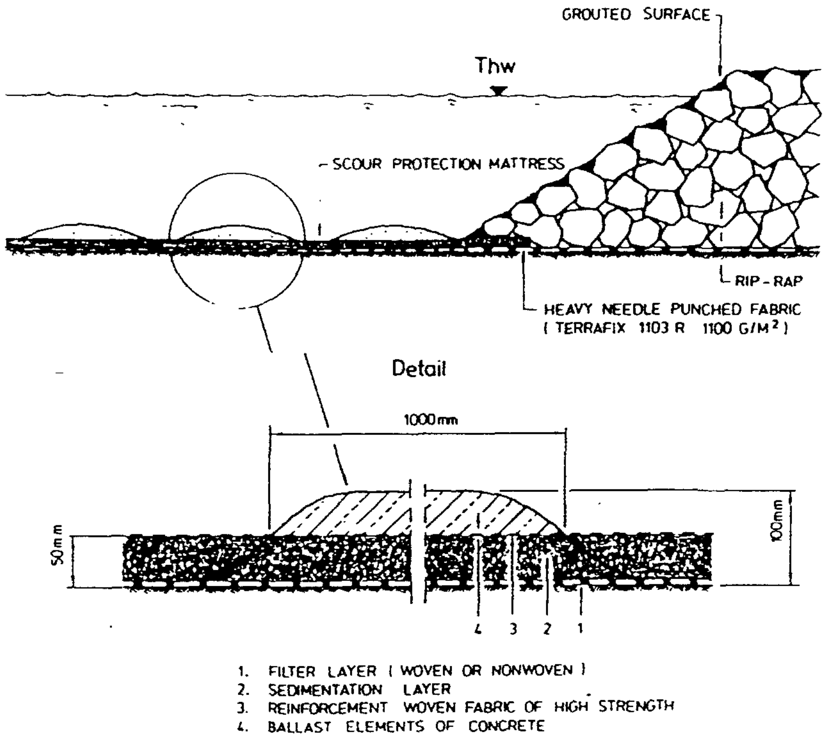


Figure 6.17. Groynes at the Weser River; construction details, Heerten, 1984.

I woven or nonwoven filter layer: soil-tightness, permeability and acting load have to be considered;

II sedimentation layer: approximately 5 cm thick, made of needle-punched and chemically bonded curled coarse fibres, it reduces the drag forces in the boundary layer of the seabed so that sedimentation takes place increasing the weight and stability of the structure;

III reinforcement fabric: by means of wide meshes (approximately 20 mm) and very high tensile strength the fabric combines and reinforces the ballast elements securely, providing a high degree of flexibility and adaptability with plenty of strength in reserve;

IV ballast elements: approximately 0.5 m in diameter and 0.1 m in height, with a minimum distance of approximately 0.2 m between the rims of the elements, they stabilize the scour protection mattress during the sedimentation phase.

CHAPTER 7

National strategies and policies of coastal defence

7.1 GENERAL

Coastal engineering has long been a conservative art. The concepts of different defence schemes are often implemented by reproduction and reference to the tradition being cultivated along a certain coastline. This must not necessarily be sound from a technological point of view because the response of a defence structure to a variety of coastal factors, in different time and space scales, may include long-term components, which are not evident at once. In other words, there must be a time span from completion of the construction period till the moment when the advantages of the structures outweigh the adverse effects. Since this duration often reaches decades, if not centuries, the real value of the defence scheme may be difficult to determine for the people involved in the project (who may simply be survived by the structure). The conservatism mentioned consists in repeating certain traditional patterns of coastal defence strategy without in-depth follow-up monitoring, in the fair conviction that the schemes repeated are good for they were applied by our predecessors.

Since experience is notwithstanding an important factor of coastal management, and learning from errors (particularly someone else's) should always be recommended, presented below are certain national shore protection strategies which, by and large, encompass traditional elements. They are by no means complete, nor clearly conclusive, yet provide a relevant contribution and supplement the rest of this report. *Inter alia*, the studies cited show the growing awareness of the pressing necessity to undertake steps towards integral coastal management on a large, at least regional, scale.

Given below is a short review of these bibliographical scources on national policies and experience which have been readily accessible to the writers. This review may certainly be up-dated, revised and made more systematic. Some regions and countries are not considered as much as they deserve to because, firstly, the references available for them are widely known and, secondly, the inclusion would require a special report for the abundance of the data. This refers, for instance, to the U.S.A. and the Netherlands who have contributed to the coastal engineering so much that it is simply impossible for the writers to summarize this invaluable work. The reader is addressed to Shore Protection Manual and many other American and Dutch sources. Fortunately enough, just recently there appeared a summary of the Dutch coastal policy (Louisse & Verhagen, 1990) so that we are devoting to it a special section at the end of this review (Sec.7.3).

The information on some other countries policies and practices, eg. Canada, France China or the South American states is dispersed and not easily accessible. On the other hand, some eastern practices may be interesting to the western reader.

The order of presentation is alphabetical, with the exception of the Netherlands (separate Sec.7.3).

7.2 NATIONAL EXPERIENCE and State-of-the-Art

7.2.1 Bulgaria

The Bulgarian coast on the Black Sea encompasses a few regions of intensive erosion (Bostanov, 1987), viz.

(I) Balchik: lanslides due to waves steadily endangers the premises and structures situated at the coastline. Numerous groynes, dykes and breakwaters, often to form docks and harbours, have been constructed there to protect the coast. Artificial beach nourishment has been used to create beaches
(II) Varna including the spas "Druzhba", "Zlotni Pyasnitsi" and "Kraniyevo": sea waves have activated erosion and dune sliding. Several recreation facilities and pump stations were destroyed. Groynes and dykes, the latter serving highway purposes, are under construction in the most vulnerable spots. Bulkdeads and revetments have been found suitable for sea defence in certain recreational areas
(III) Nesebar- Ravda - Pomorye - Saratovo - Burgas: groynes are mostly applied. Major attention has been concentrated on the defence for the ancient town of Nesebar and shore stabilization along some other sectors
(IV) Michurin and the village of Kiten.

The construction of sea defence structures was accelerated lately as severe storms in the recent years caused serious damage and flooded resorts, towns and villages. Storm surges as high as 150 cm above mean water level have intensified erosion and sliding of dune bluffs and cliffs.

A special authority has been established in Bulgaria to co-ordinnate and programme their coastal protection for the Black Sea. All complex problems of flooding-ports and beaches are under jurisdiction of this body. The studies have shown that the sediment supply from rivers and dune erosion has been reduced due to anthropogenic factors, thus causing the aforementioned problems of widespread shore erosion, activation of older slip surfaces, undercutting of highways etc.

The Bulgarian shore protection strategies include i.a. very heavy breakwaters and groynes. Typical cross-sections, as wide as thirty and more metres at their base, with two or three layers of rubble and tetrapods etc. are shown by BOSTANOV (1987) along with vertical-wall structures, such as massive bulkheads and caisson breakwaters. On the other hand, a 30-km strength of coastline will be protected by artificial beach nourishment from Kalyakra to Balchik.

7.2.2 Germany

The first settlement of the marshland areas on the North Sea and in the tidal areas of the Rivers Ems, Weser, Elbe and Eider in the period before Christ was possible without protecting habitations against storm tides since the sea levels were lower than they are today. As the water levels rose in the following period the inhabitants had to build mounds of earth, so-called "Wurten" appeared to protect people and their property against storm tides. As the number of inhabitants increased, individual settlements were enclosed by ring dykes, until by about the 13th century the first continuous embankment of the North Sea coastal area has been built.

With continuing rise of storm tide water levels up to the present day, the height of embankments also increased from around 2.0 m up to 9.0 metres above the level of the surrounding land with an increase in width from about 4.0 metres to 70 metres.

This also takes account of the recognition in recent decades that the inner slope should be built flat so that the highest waves breaking over it cannot wash away any soil.

After the very severe storm tides in the North Sea in 1953, 1962 and 1976 the raising and strengthening of all sea and river embankments in the North Sea coastal area was again begun and will be completed about 1988, Kramer (1978).

The protection of the Baltic coast against storm tides differs from that on the North Sea because here there are no broad areas of marshes and flats. The underwater bed slopes more steeply so that greater depths of water reach right to the coast. Of the $530km$ laong Baltic coast the greater part has shallow beaches in front of low lying areas which are broken in some stretches by steep cliffs. The land lost on the Baltic coast has always been more than the land reclaimed.

Dyke building began in the middle of the 18th century and was carried on more intensively after the very bad Baltic storm tides of 1872, when the aim was to achieve embankments with a crest height of NN + 5.0 m. Dykes along the sand bar were endangered by the further natural retreat of the beach so that they had to be secured with massive reinforcements of their footings and with groynes. In the long run, however, this rigid embankment protection was only successful when natural transport brought in sufficient sand to prevent undercutting.

In recent times beaches have been built up artificially in cases where the sand balance remained negative as it has been recognised that the sand requirements to maintain all the beaches cannot be maintained from the limited sand stocks in the coastal foreshore of the German Baltic.

The maintenance of the cliffs on the Baltic involves the same problems, which have mostly to do with negative sand balance. In the middle of the 19th century a start was made on the protection of individual stretches of shore with groynes and provision of the prevention of sand movement into shipping channels and harbours. The engineering works in many areas were ineffective, so that beach nourishment was also necessary here if further erosion of the coast was not to be allowed.

The first dune protection works were closed or open timber works. Their length of life was limited because of the poor durability of the wood. Very soon, therefore, solidly built paving works in steep profile or S-profile began to be constructed.

German Historical Experience

Petersen (1963) evaluates the experience accumulated by 120 prominent German coastal engineers and scientists as published in 210 bibliographic items, which extend over a period of 140 years. It offers an interesting insight into the opinions and attitudes of German engineers toward the historiclly controversial and highly disputed subject of coastal protection by means of groynes.

The first sea groynes in Gemany were constructed in the period 1818 to 1821. The first publications dealing with them are about one hundred years old. They may be found in manuals and in approximately fourty different periodicals. In addition to engineering papers, there are numerous publications by geographers, geologists, oceanographers and local historians, in which described are certain conditions and events useful for groyne investigation, or in which pertinent relationships and developments are revealed.

Petersen's (1963) classification of German documentation on groynes is divided according to coastal subjects, regions involved, and also chronologically.

The geographic location of Germany accounts for the regional distinction necessitated by dissimilar conditions on the Baltic Sea (a tideless inland sea), and the North Sea (a marginal sea characterized by its high and low tides). In the Baltic Sea the coastal currents generally are unimportant, while the tidal currents of the North Sea create a periodic movement of water masses. In addition, the currents there are augmented, when the filling and emptying of tidal flat areas (Wattenraum) occurs through submarine canyons (Strom gaten), tidal flat streams and river outlets.

Distinctions also exist between beaches with strong tidal or drift currents parallel to the coast with or without surge and those beaches with surge but without strong tide or drift currents. The surge itself often produces "a surge current" (Brandungstromung) parallel to the coast and of an intensity exceeding considerably that of the tide or drift current.

Prior to 1860, only a few reporters were writing on the construction and effect of sea groynes because not many such structures were in existence then, and consequently only a few observations were made and data collected.

Of course, data on the effectiveness of groynes are of limited value **if collected immediately or soon after the completion** of the structure. Usually a minimum period of 10 to 20 years is required for an objective evaluation of the groyne effectiveness.

(a). Construction period prior to 1900 - By 1900 several basic papers on coastal morphology, coastal engineering and a treatise on the big Baltic Sea storm tide of 1872 were already published in which the authors tried to find the causes and effects of coastal changes. They were good observers of natural phenomena on beaches. The groynes constructed during this period have a special experimental value, and for this reason researchers in subsequent years draw often on the ideas of older authors. They compare the old construction methods with the observations of a later period to see how the older design ideas materialized relative to their effectiveness.

(b). Construction Period 1900 to 1920 - In this period a distinction betwen sea groynes and stream or river groynes can be noted. However, full definition relative to functional limitation and structural forms involved is not made clear.

(c). Construction Period 1920 to 1930 - The German Harbor Engineering Society entered into the discussion on questions dealing with groyne construction. Of great significance to the German coastal problems was the XVth International Navigation Congress of 1931 in Venice. The recommendation made at the Congress by Coen Cagli clarifies the German attitude as follows: "Each plan for coastal protection must be preceded by a thorough study of the locality and all factors acting on the formation of the coast and how much it is exposed; the action set up by waves, currents of various kinds, atmospheric precipitation, and by floating ice; the origin and nature of material constituting beaches and the regimen of river deltas; the situation and regimen of the aquiferous sheets of fresh water flowing toward the sea; the influence on the sea by new scructures when they project from the coast". It was further stated at the Congress that the causes of the effect of the sea on coasts should be substantiated. The German model experiments during this period were successful, and their continuation under natural conditions was recommended.

(d). Construction Period 1931 to 1945 - A general reversal of coastal construction method took place during this period as nearly all of the groynes on the North Sea as well as the Baltic Sea were built of steel sheet piling. The economic life of this type of groyne structure is very short (10 to 20 years), particularly when single steel sheet piling is considered. The result of a series of investigations in coastal

morphology carried out in this period opened wide vistas into coastal zone phenomena and borught out that the knowledge of forces involved in those processes is very limited. The demands for more intensive and systematic investigations were thus substantiated. Differences were increasing between the practical builders, who believed in progressive success of groynes, and the research engineers and natural scientists. Also, contradictory experiences and opinions were noticeable among the practicing professionals.

(e). Construction Period 1945 to 1960 - Petersen's (1963) survey covers both parts of Germany, East and West, with considerable reference to East German scientists and engineers, including governmental agencies and institutes of higher learning. Following World War II, the incoming information was at first very scanty, consisting mostly of reports on the use of asphalt for construction of groynes. Ideas on the role of **sea groynes** continued to be **very controversial**. They have to serve for current deflection, beach protection and sand collection. It is confirmed by several authors that the groynes fulfilled their role as coastal current deflectors. The *groynes failed as beach protectors and sand collectors*, particularly on coasts with a deficient balance of sand. Following systematic experimental observation, it was found that the sand deficit could be balanced by artificial sand replenishment.

German Experience on Groynes

The overall survey of *experiences* on the **effectiveness of sea groynes in Germany** with special regard to engineering and economics is evaluated indicating that controversy on the value of groynes continues. Great progress in the structural strength of groynes relative to stability and material does not prove their functional effectiveness, which requires long periods of time for proper evaluation.

a. Motivation for Sea Groyne Construction - In most cases, sea groyne construction was motivated by damages caused by storm tides and the dynamic forces of the surge. Damages appearing at the foot of dunes and seawalls, as well as on longitudinal structures and on groynes themselves, resulted from lee-erosion and the effect of currents moving progressively beachward.

b. Current Groynes and Beach Groynes - Sea groynes are of two types; current groynes and beach groynes - each type having a different function. Current groynes are current-deflecting structurs and it has been proved that they are capable of fulfilling this role. Beach groynes are expected to maintain the surge-exposed beach and improve it as much as possible. On certain beaches it is considered, with some reservation, that groynes do retard the shore recession. The opinion on the role of beach groynes varies from that on the unverified extent of the reduced recession rate (success = 1 % to 99 %), to that on maintenance (success = 100 %), up to that on improvement of the beach (success > 100%).

On transitional reaches which are influenced by rivers and inlets from the sea, and on surge beaches, groynes may serve both the functions of current deflection and beach protection. Here the success can be assessed only when the beach groyne performance is clarified. The German technical literature does not yet provide a satisfactory clarification. Current-deflecting structures were constructed to an equal degree with those which would serve for beach conservation and improvement.

c. *Groyne Types* - Beach groynes have been tested in nature in an unusual number of cases. From the original procedure on constructing groynes as permeable structures, in the form of single or double-row pile groynes or stone cribs and fascine groynes, progress was made toward impermeable massive stone groynes, sheet pile groynes of steel or reinforced concrete, and recently toward flat stone pitch asphalt grouted structures. Permeable and impermeable groynes are found side by side in the same coastal reaches. The design of steel sheet-pile and reinforced concrete pile structures was apparently governed almost entirely by limitations imposed by structural engineering practices.

d. *Groyne Length* - Initially the length of the groynes was determined by structural engineering limitations. Only after the introduction of the steam pile driver did the groynes advance into deep water. Already Germelman in 1881 (mentioned by Franzius) had used the jetting procedure, which was found very suitable for long piles (also sheet piling). The water jet was applied by appropriate equipment. Current-deflecting groynes were constructed with good results, seaward up to and beyond the MTLW[1] and also through stream channels.

With regard to beach groynes, opinions as to their proper length were far apart. Groynes of limited length were considered adequate on dry as well as on wet beaches, while groynes reaching beyond the first sand bar were proposed when sand movement was the main problem involved. In the Baltic Sea this bar is located approximately 100 metres, and on the North Sea (Sylt Island) 300 metres, from the shore line. Only economic reasons account for the fact that such proposals regarding groyne length were not, or only in part, accomplished. The construction as well as maintenance cost increased considerably with the length of the structure.

e. *Elevation of Groyne Crest* - With regard to the most efficient elevation of the groyne crest, a variety of ideas were found to exist. Some authors had the attitude that a horizontal position of the crest was correct, others held that a certain minimum margin above the beach elevation was necessary; still others constructed groynes high above the beach, to the final elevation which they desired to achieve when the sand-catching capacity of the structure would be reached. New proposals point toward structures built so that they would follow the profile of the beach. This is possible only in the downward direction. During periods of heavy sand movement these groynes would be buried.

f. *Groyne Groups - Lee Erosion and Groyne Spacing* - Wherever groynes were

[1]Mean Tide Low Water

erected, it was found that single groynes never met the requirements and that more of them were always needed to provide protection or maintenance of beaches or to retard shore degradation. Always in the lee of surf currents (adjacent downdrift areas) new damages were found, requiring the expansion of groyne groups. This development logically led to a "Totality System", meaning that the entire coast had to be protected by groynes. However, Hansen (1938), considering the natural conditions of sand behaviour, believed such a limitless procedure should be rejected not only for functional but also for financial reasons.

In a series of investigations regarding methods to alleviate the lee erosion problem, no satisfactory solution had yet been found. This problem played a considerable role in deciding the terminal limits of a group of current groynes as well as for all beach groynes in general.

Within a group of groynes, spacing of the structures was determined in many ways; it varied from one to three times the length of the groynes, without any distinct recognized system. As during the course of development, lengths of the groynes were considerably increased in comparison to original concepts, a definite length--spacing relationship could not be derived on the basis of available engineering experience.

g. Direction of Attack (Groyne Alignment) - A number of authors have held it appropriate to place the heads (outer ends) of a group of groynes in a line conforming to the direction of current flow. In dealing with current-deflecting groynes, this attitude is unanimously accepted as correct. However, in observing the underwater groynes between Baltrum and Norderney, a difference in this attitude can be noticed which has not created disadvantages of a functional nature.

Some modern authors consider the line of attack in design of groynes; some attribute no meaning to the line of attack while other authors do not present the subject at all.

The reason that a pliant line of attack is required in river engineering is the effort to achieve a regulated flow, particularly for the purpose of navigation. On beaches we deal with one shore only. The currents change direction and intensity, and even though of low velocity, they develop, in combination with the surf, a considerable transport capacity, as the surf action provides the suspended sand.

The importance of the line of attack is doubtful with regard to short groynes which do not reach the offshore sand bars where the sand transport usually takes place.

h. Construction Material - A variety of construction materials used in building sea groynes is listed below: Wood: piles, spars, timber of available beach types, and brushwork (endangered by marine borers).

Natural stone.
Steel: single or multiple wall groynes built of steel sheet piling; connecting elements of structural parts (endangered by rusting and corrosion).

Concrete: used experimentally at the beginning of the 20th Century, since then systematically.
Asphalt: for several years used for sea groyne construction.
When one construction material is being replaced progressively by another, it is usually because of its economic life. This is influenced by changes of wetness, solar radiation, temperatures, surge impact (wave action), surge currents, elevation changes of the shore, sand abrasion, ice drift and sea bottom composition.
The conomic life of steel and reinforced concrete for sea groynes is generally only 10 to 20 years.
Wood can be destroyed in two to three years, when used in regions subject to marine borers or other wood-damaging phenomena. Without this deteriorating effect, the durability can extend several centuries.
Stone has an unlimited durability. It is being used as loading element in connection with brushworks or for bracing of steep banks or slopes.
As to the use of asphalt for coastal groynes it is still too early to express a conclusive opinion in view of existing experience and disappointments with other construction materials.
The economic life of a structure fixes its maintenance cost.

i. Artificial Beach Nourishment - The artificial addition of sand in regions with a deficient balance of natural sand supply is the most recent method of coastal protection, despite the fact that several decades earlier dredged material was discharged for such purpose on beaches of the Baltic and North Seas with good results. The time is too short to report on the results of this method of protection.

In conclusion, German sources (Petersen, 1963) are unable to clarify the complex problems occurring on sandy beaches and to interpret all cases properly. It is generally admitted, in view of the fact that the literature on the effect of beach groynes is encumbered with so many statements and hypotheses, that it is necessary to either substantiate or invalidate those statements analytically by measurements and figures. In view of the high cost of construction and maintenance, this procedure should be given primary importance. It has become obvious that only through such methodical measurement and systematic investigation, patterned after G.Hagen's ideas, would conclusive judgement be possible to some degree. Besides the necessary measurements in nature, theoretical hydrodynamic investigations, measurements and observations on model tests should be promoted. New measuring instruments will have to be developed and a good start has already been made.

Systematic observation of coastal development trends, carried out over a period of decades, may help to avoid obviously unsound investments.

Disappointment can be avoided at the construction stage if reports are critically drafted and failures made known.

Petersen (1963) postulates that all available literature on coastal regions, both domestic and foregin, be collected at the Coastal Commissions of the Baltic Sea and

North Sea and be publicized. A researcher of a coastal problem, in most cases, due to lack of time to undertake a survey of information sources, has not sufficient knowledge of previous achievements in a particular field.

Dette & Gärtner (1987) examined the history and experience of the sea wall constructed on the Island of Sylt, where the **armouring effects** discussed in Chapter Two have surfaced.As noted, more than 100 years ago men started on the Island of Sylt (North Sea) to interfere with a natural long-term coastal recession of sandy beaches and dunes. Groyne systems did not prove the expected success. In 1907 a 70-m long seawall was built by a private owner in order to prevent further recession in front of an endangered hotel. Already in 1912 an extension became necessary because of the downdrift erosion. Nowadays the chain of coastal structures has reached a length of more than 3 km in front of the city of Westerland, causing the armouring effect. Dette & Gärtner (1987) presented a chronological review since 1865 of all types of coastal structures, with special emphasis on the seawall.

Already in 1867 three heavy stone groynes were built in front of Westerland. Until 1900 another 44 of such works, with a spacing of 500 m, and two intermediate lighter groynes in between, were constructed for protection of the central part (22 km) of the island. These crossing works only could retard the recession process.

After another heavy storm surge in 1907, a 70 m long sea wall was constructed in Westerland to protect the Miramar hotel. After further storms the seawall was lengthened northward in 1912, southward in 1923 and once more northward in 1924. At this time it had become necessary already to strengthen the earlier seawall construction by means of toe protections.

In winter 1936/37 two heavy storm surges occurred and caused dune recession in between 10 m and 12 m northward of the seawall. After a 20-m wide beach fill in front of the dune as an immediate remedial measure in elongation of the seawall a 1 on 4 revetment, consisting of basalt blocks was built in 1937/38 over a length of 510 m.

In 1946 the northern end of the seawall was replaced over a length of 3,140 m by a revetment consisting of concrete slabs. This construction followed destruction of a 60-m long section of the seawall during a storm surge. In 1950 it became necessary to enlarge the toe protection in front of the seawall by additional width of 5 m; this was in the wake of continuous undermining of the beach. Due to considerable lee erosion at the northern end of the revetment it was necessary to extent the revetment in 1954 by 200 m. In December 1954 yet another heavy storm surge occurred and again heavy lee-erosion was caused, this time at the end of the elongated revetment.

After 1954 the length of coastal works had reached 1585 m (710 m long seawall and 875 m long revetment). Furthermore concrete pile groynes and 2 flat groynes were constructed in order to prevent further beach erosion. In 1961 a critical part of the seawall, a 100 m long part at the northern end, had to be strengthened. It was decided to place 6-ton tetrapods with a slope of 1 to 1.33 in front of the sea wall. From this measure it was expected that the incoming and reflected waves would be attenuated so much that the seawall would not be endangered any longer and further erosion of the beach could be prevented. In order to protect the sheet-pile toe protection which then was already 14 m distant from the seawall this toe protection was extended by an additional width of 10 m. By this means a "3rd generation of toe protection" was created (!).

In 1962 the German North Sea coastline was hit by a disastrous storm surge (classified as a centennial event). In order to prevent here a breach in the narrow dunes and a possible flooding of the city of Westerland, a 420 m long "tetrapod dam" was built as extension of the revetment. Also the southern end of the seawall had to be extended by a 300 m long "tetrapod dam" due to lee-erosion.

Dette & Gärtner rigthly identify as futile all these efforts against the erosion of the beach in front of a structure. Nobody is sure if the final attempt undertaken in 1970 as heavy riprap stones placed in front of the toe protection would serve the purpose of ultimate remedy. It appears that the **beach nourishment** programme, initiated in 1972 and repeated in 1978 and 1984 is as yet the best guarantee of the sea wall stability and survival.

The seven East Frisian Islands (Fig.5.37) protect, as a natural barrier chain, a part of the German coastline. On the seaside these islands have sandy beaches that are exposed to strong wave action and tidal currents. A littoral drift runs from west to east. The islands are separated from each other by tidal inlets that are crossed by crescent offshore bars.

In order to protect the health and bathing resorts against erosion, people constructed solid protection works and implemented artificial beach nourishment. Kunz (1987) reviews the projects for two of the islands that experienced a different evolution of the inlet and the shore line. On Norderney, for more than one hundred years people attempted to stop the erosion of its west end by seawalls, groyne and beach nourishment. On Langeoog however, most of the time there is a sufficient sand supply by approaching sand bars. Nevertheless for the last fifteen years, artificial beach fills have been carried out to reinforce barrier dunes and the beach in front of them. Our Section 5.6 describes the East Frisian experience with regard to artificial beach nourishment. In the present section we will try to provide some general observations on the regional defence strategies, concepts and designs.

The islands of Norderney and Langeoog are composed of sand and silt. The higher parts are covered by dune. A chain of barrier dunes stretches along the sandy beach on the seaside. The southern border is a salt marsh, naturally developed by siltation.

The islands have supposedly existed for several thousand years. The inlet between

Norderney and the western island Juist changed from a double to a single shaped profile. It was shifted towards the west end of Norderney by accretion of Juist and by extension of the inlet-catchment area to the east. Therefore man protected the west end of Norderney by alongshore structures and groynes. On the contrary, the inlet between Langeoog and the island of Baltrum to its west remained comparatively stable. Langeoog was sufficiently supported by sand bars that migrated from Baltrum in a crescent shapped bow to its west end. Therefore this part of the island naturally enlarged and no constructions had to be built. The east end of Juist and Norderney propagate to the east as a result of accretion. This phenomenon is known as the "eastward migration" of the East Frisian Islands.

If the west end of the island is not sufficiently supplied with sand, the bow of sandbars approaches the island a distance from its west end. On Norderney this stage had been developed about 200 years ago causing man to defend the increasing erosion of beach and dunes. On Langeoog this occurs periodically, since the beach approach pattern of the offshore Bars scatters.

Shore Protection of Norderney

On Norderney shore protection by solid construction began in the middle of the 19th century. About fifty years before, the island had become a health resort and therefore the settlement on its west part developed rapidly. In 1857/58, the first seawall was constructed on a 975 m long stretch. It was shaped as the so-called Norderney-S-profile. The construction made by quarrystone and lime mortar was so excellent that it still exists as an integrated part of the altered protection work. The seawall had to be extended and protected by groynes. The extension of the seawall by brushwood (1867) and by timber-work wave breakers (1877), were only temporarily successful. The maintenance of these constructions was expensive and they could not withstand severe storm events. Therefore, in 1883-k84, and 1897-98, they were replaced by a solid seawall.

Detailed description was provided by Fulscher (1905) and Backhaus (1938). At first the groynes stretched only down to the low water line (MLW). They could not stop the eastward directed migration of the tidal inlet that jeopardized the existing construction. Therefore, from 1897 on, the groynes on the west edge were extended into deep water. Up to now the system of the protective structures has been widened to a length of almost 6 kilometres and the number of groynes increased to 32.

Some of the constructions have been completely replaced and all have been extended, strengthened and altered.

Examples for alongshore structure cross-sections are shown in our Chapter Five. They might not be optimally designed by present standards, but in general they have proved to be successful. According to the historical development, they can be assessed as an optimum compromise between sufficient protection against storm tides and minimum building and maintenance costs. The newer structures are constructed as sloping seawalls, as shown in the cross-sections $P_1 - Q_1$ and $S_1 - W_1$ (Fig.5...). Theoretical principles, practical experience and limited money have led to a slope of 1:4, that is thought to be optimum.

The first slope of 1:4 was built in 1938, when part of a steep shaped beach wall that was damaged by surge and waves had to be replaced. The sloping seawall for the stretch $P_1 - Q_1$ was implemented in 1949-50, replacing another part of the steep shaped beach wall that had been constructed in 1919. The stretch $S_1 - W_1$ was secured between 1950 and 1953, with additional reinforcements after the severe storm flood of 1954. All longshore structures had to be reviewed and heightened after the "Orkan"-flood in 1962.

The first groynes were built in 1860-62, as shallow sloping structures with ashlar and basalt pavement. They only reached down to the low water line and did not prevent beach erosion as expected. This kind of groyne protects the entire western edge of the island. Since the end of the 19th century, the groynes stretching towards the tidal inlet were extended to a depth of 18 m. Thus the eastward migration of the deep channel was stopped. After 1948, the groynes were predominantly constructed as "crib groynes" and "pile groynes". Their main purpose was to turn back the tidal flow of the foreshore area down to NN-4.0 m. Some of them helped to reduce the scouring, but they could not change the trend of the beach development.

Artificial Beach Nourishment on Norderney and Langeoog

The "classical" approach to beach protection by means of embankment and groynes ended on Norderney in 1951-52. Upon recommendation from the "Working Group Norderney" (1952), the first artificial beach nourishment in Germany was implemented. This is described in our Section 5.6, together with the measures taken for Langeoog.

The island of Heligoland, lying in the bay of Heligoland occupies a special position in the German coast protection. The island with cliffs of soft rock, which has been reduced to a fraction of its original size by wave action, was given a protective wall, right round the island, in the first decade of this century. The small barrier (dune) islands lying close by, which are mainly used as bathing resorts, also require protection. Both are surrounded by protective works of various kinds, from vertical embankment walls and steel sheet piling to walls of tetrapods. These various protective works are accounted for not merely because of Heligoland's importance as a bathing resort, but because of its earlier role as a naval base and its present use as an emergency harbour during North Sea storms. An extensive system of breakwaters has been developed for the harbour installations which also serve as a protection for the built up land areas behind.

The East German (former G.D.R.) coast on the Baltic Sea is quite particular for its sections of roughly uniform morphology and climate are quite short. Hence the littoral drift consists of interrupted streams of sediment supplied from eroded marine formations. The erosion rate at those supply headlands is estimated at up to 2 m of a yearly retreat of the shoreline, as indicated by Khomitskiy (1983). Sea walls and offshore breakwaters have been used everywhere along those abraded headlands. This measure in turn brought about an intensified erosion of the adjacent areas as they were gradually depleted of the sediment being earlier supplied from the headlands. The construction of groynes in the erosion bays between the headlands has not produced the expected effects as the erosion was extended downdrift. Repeated storms, which rise the sea level by 3.5 m at places, and cause flooding and irreversible damage on the shore, tremendously worsen the situation. At present the GDR coastline seems to be defended along a considerable stretch, with shore protection measures thereon, but still the erosion processes are not under control, yet another example of coastal armouring, so conspicuous along the Sylt beach at Westerland.

7.2.3 India

The Nature's coastal protection is demonstrated everywhere in India at the headlands, reefs, rocky shores, dunes and at the points where rivers supply material to the shore. There are few places in the world which have such an abundance of natural protection as in India. The Nature's sea wall is most often made of lateritic rock which has proved to be quite safe against abrasion and weathering. From the point of view of coastal protection, the supplies of material by rivers to the shore of India, mainly during the monsoon time, is a blessing despite the fact that deposits at entrances create problems for navigation. The mud banks of the Kerala coast have also been an advantage but due to the fact that they migrate along the shore, their beneficial effect is only intermittent. They cause severe terminal erosion problem due to changes in wave characteristics brought about by them.

In India, protection of property on the beaches is most important as some shores, particularly in the Kerala State, are very populous. The fact that they are often located on narrow barriers separating the sea from lagoons or from low lying wetlands makes the problem rather difficult. Under such situations, little or no land can be lost to the sea. Rather, the need for recovering more land from the sea is urgent. Reclamation of land has been undertaken from the estuaries, backwaters, mangroves and rivers for centuries but seldom from the open seashore, although dunes and elevated sand platforms have been built or rebuilt at some places on the shores.

The type of protection which is most practical under the conditions prevailing today is the sea walls, or rather the so-called revetments or sloping walls. These are mostly built in India from coastal rocks such as granite and laterite. Groynes have been used in some "desperate" cases, e.g. on the downdrift side of the improved inlets (Cochin harbour, Kerala) and at the beaches downdrift of headlands (Moplah Bay, Kerala), but unfortunately they have not ben very successful although they have benefitted some localized points. The overall effect of groynes on erosion often is to do more harm than good due to the unavoidable leeside erosion. Groynes should, therefore, be avoided unless their main purpose is to keep a beach at one particular position at the cost of adjoining areas. In such cases the so-called, T-groynes which trap the beach material are preferable. But the adverse effect of T-groynes is similar to that of other groynes.

Shore parallel breakwaters have hardly ever been used for coastal protection in India but some harbour breakwaters extending from headlands such as at Cannanore and Vizhinjam in Kerala have faced adverse effects on the downdrift beaches and also on the maintenance of depths in the respective harbous. Artificial nourishment of beaches has not yet started in India but it will undoubtedly become the most widely used method for protection in India in the near future. There is hardly any place in the world where it is better justified than India because of the heavy population pressure on some highly developed shores of India. Nourishment will then mostly, if not always, have to come from the sea, comprising everything from man-powered draglines and winches to hydraulic dredging to be discharged on the beach. Wherever harbour structures have caused erosion due to their effect as littoral drift barriers, transfer of material by pumping should be initiated, e.g. at Madras and Paradip harbours. Special types of intermittent hydraulic transfers have been tested, e.g. at Nagapattinam while others are in the planning stage.

Choice of protective measures

A right choice of coastal protection for the conditions existing in India will perhaps rest invariably on a certain type of sea wall.

The most important basic factors to evaluate the need for a sea wall and its expected functions are the erosion situation of the offshore bottom and the availability of material for natural nourishment of the beaches. Table 3.6 in Bruun & Nayak (1980) (our Appendix 2-2) gives details of the sea wall design, with a certain reference to India, while Tables 3.7 and 3.8 (App.2-2) refer to groynes and offshore breakwaters. They are included for the purpose of general information about their performance although they may not deserve the same interest and practical use as sea walls and revetments. Table 3.9 gives the details of the performance of artificial nourishment. Furthermore special attention should be paid to bypassing of material at harbours and tidal inlets.

Brunn and Nayak (1980) regard some of the most important rules in planning of any coastal protective measure as:

(a) to look at the problem on a broader scale and to be farsighted
(b) to plan coastal protection in full consideration with what is in the neighbourhood. In other words, coastal protection should always be handled by a large group of people, a public organization or a city council which would be able to take an overall of the problem for the benefit of all concerned.

A well-coordinated programme with the participation of various intrest groups is required to consider multiple use of coastal resources. Although State governments are to administer coastal zone management programmes, both Local and Central interests should be carefully considered along with State interests. Short-term and long-term interests have also to be considered while planning any improvement programme in the coastal zone.

Administratively, one must distinguish between regulatory steps and steps towards improvements.

Regulatory steps are normally intended to control the construction or crection of all coastal structures to ensure that no structure will be placed or no action will be taken which will have harmful effects of an order of magnitude which will be against the interest of all concerned. Administration of such regulatory measures must be in public hands. Regulatory steps should include rules for the location of buildings on shores, particularly with respect to their minimum distance (set back) from an (eroding) dune, rules for the maintenance of dunes and coastal platforms and rules regarding the removal of beach material. Such removal should always be prohibited from all eroding shores.

The problem in India at present is connected with the enforcement of such rules, particularly in denesly populated shores where every metre of recession of the dune increases the congestion of houses, which are often saturated up to a maximum limit. In such cases, individuals or goups may become so desperate that they take actions which are not only inadequate for their own needs but dangerous to others who own property on the same beach. It is often very difficult to cope with such survival actions. The only practical way of handling such problems is by well-planned remedial measures based on both short-term and long-term considerations.

Steps on improvements include well planned coastal protective measures which are built to last long if maintained properly. As mentioned, the **situation in India favours the use of sea walls**, mainly of revetment type which stops the dune erosion, but it may not necessarily stop the beach erosion. Most often they do not, because erosion will continue to occur due to an imbalance in the material supply/loss system.

Administratively, action on coastal protection measures will have to come from one or more groups of people through a public agency- -Central and/or State or local municipality, which should ultimately consider the action on planning and financing. Improvements, however, need not necessarily have to be directed only as counter-attacks on the sea. It could well be the actions on the withdrawal of homes or other facilities for re-locating them further inland at public cost. The area affected by erosion may then be left as a recreational park for the public or for other usages by the public, protected and maintained at public cost.

Such steps, however, may easily infring upon the rights of individuals and they may take a long time to resolve the dispute.

Coastal protection should not become a passive defence strategy. India cannot afford to lose land to the sea in several of its populous states. It must win the battle by regaining the lost land by suitable measures and the ultimate answer to India's coastal protection problem undoubtedly lies in **offshore dredging of sand for the nourishment** of its starving beaches.

7.2.4 Italy

The problem of coastal erosion in Italy (approx. 7600 km of coastline) has recently reached alarming proportions. Sixty to seventy percent of the beaches suffer from accelerated erosion or are in extremely precarious conditions. This evolution has been caused by a combination of physical and anthropogenic factors, and has brought about significant changes in the environmental resources. As a result, a series of sporadic and uncoordinated coastal protection measures were taken, thus causing situations of disorder in neighbouring areas. Basing on these negative experiences, the authorities of the Marche and Abruzzo jointly performed a general study of most factors influencing the coastal dynamics. The general programme of defence works devised is now being implemented along 280 km of the coastline (Moretti and Pedone, 1984).

7.2.5 Japan

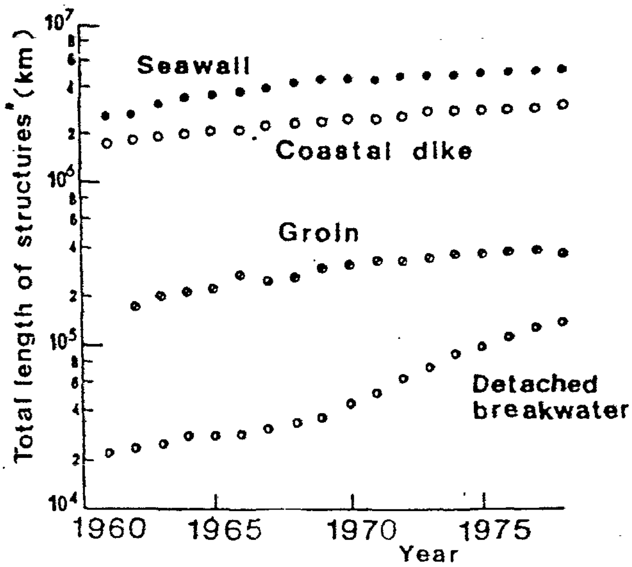
The coastline of Japan extends for a total length of 33,800 km. It is very long compared to the enclosed land area od 380,000 km^2.

The modern history of coastal stabilization works in Japan can be divided into two periods; the first is from 1945 to 1953 and the second extends from 1953 to the present. At the end of the World War II in 1945, the Japanese coastline lay in ruins.

The storn surge disaster in 1953 opened the eyes of the Japanese people to the importance of coastal protection. They therefore pushed to establish a new act for promoting coastal preservation. This was the Coast Act, issued in 1956. Until the concept of coastal management appeared in the 1970s, the Coast Act covered only a quite limited area of coastal protection.

Fig.7.1 indicates the total length of various coastal structures constructed in Japan till 1978. The main structures are sea walls and coastal dikes. Detached breakwaters have been utilized at a rather high rate since 1970. Fig.7.1 illustrates the percentage of various types of coastal protection structures constructed in each fiscal year at the sites controlled directly by the Ministry of Construction. The perception of the effectiveness of each structure has changed during the last twenty years:

(1) coastal dikes have maintained a share of 50% or more,
(2) sea walls have decreased in share tremendously,



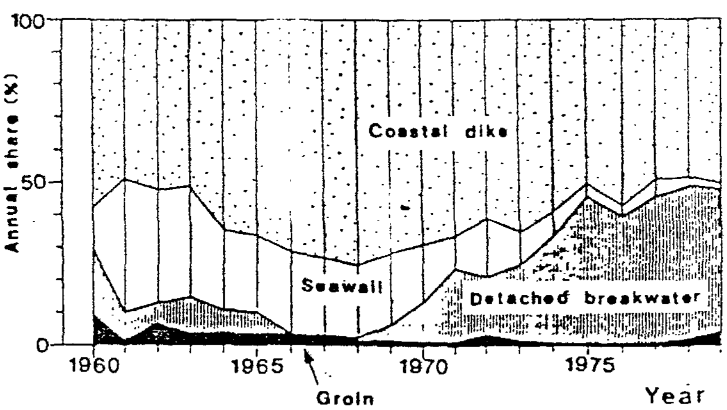


Figure 7.1. Coastal Protection Works in Japan, 1960...1978; Watanabe & Horikawa (1983).

(3) groyne systems have not been particularly successful and
(4) detached breakwaters have been applied very widely during the last ten years and have taken a major share of about 50%.

It may be said that beach erosion control in Japan at present is accomplished mainly through coastal dikes and detached breakwaters. In the following, we would like to point out trends and problems to be solved for each measure.

In the early stage of design and construction practices in Japan, structures with rather steep front faces were commonly used. However, it was found that severe erosion at the toes of such structures was a serious problem in maintaining the integrity of the structures themselves, and that the loss of foreshore sand was accelerated by the construction of these structures. Therefore, two modifications were made. One was to reduce the wave energy in front of a structure by setting armour blocks, and the other was to make a gentle slope on the dikes. The latter type of structure is favourable for providing access to the beach.

It is rather difficult to give an example of a groyne system which functioned positively without producing some adverse effect as well in Japan. The tendency of a detached breakwater to form a tombolo is at present of greater interest to coastal engineers in Japan on the basis of past favourable results. As yet, the determination of the location and alignment of a detached breakwater is based on experience. Intensive investi- gations are needed to explore quantitatively the effect of detached breakwaters on coastal stabilization.

A number of attempts have been made in Japan at various localities to establish artificial beaches to be used mainly for recreational purposes. The preservation and creation of natural beaches have become interesting projects for coastal engineers.

Coastal stabilization works in Japan have evolved considerably over the past several decades. Such "hard" measures as seawalls have given way to rather "soft" measures such as detached breakwaters and beach nourishment. This change has been brought about by progress in the understanding of littoral drift as well as by public demand for natural beaches, not just for protection of the coastline. The effectiveness and limitations of artificial beach nourishment and other soft measures, such as headland control systems, should be more intensively investigated. The history of success and failure of coastal protection works in Japan has also shown that coastal development, as well as other land development, often exerts a strong impact on the natural environment, Watanabe & Horikawa (1983).

7.2.6 *Lithuania*

The total length of the Lithuanian shoreline is about 100 km. This is the shortest coastal strip belonging to any country bordering the Baltic Sea. The Lithuanian coast stretches from the middle part of the Kursiu nerija Spit (a coastal barrier) northward to the border of Latvia. Located on the Lithuanian coast are Klaipeda

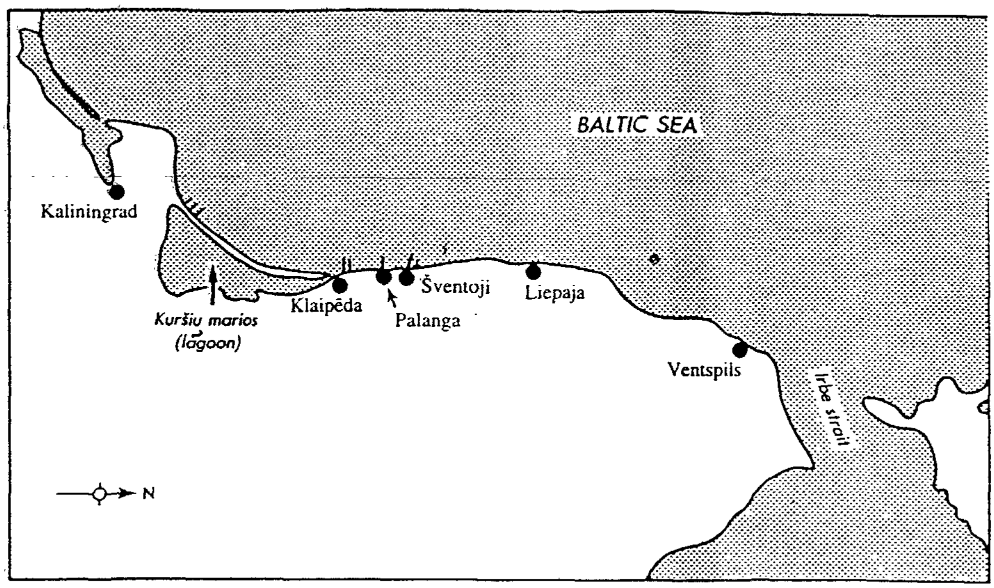


Figure 7.2. Locations of Anthropogenic Impact on Lithuanian Coastline.

(a large trade and ocean fishing harbour), Palanga (a famous sea resort agglomeration), and several recreational and industrial centres (Fig.7.2). The northern part of the Kursiu nerija Spit is a landscape preserve. The coastal zone is densely populated. During summer the population reaches about 0.5 milion.

The coastal zone bears all of the characteristic features of a tideless, low-lying, and shallow accumulative (aggradational) coast (Gudelis et al. 1979). Situated in the southern part of the coastal area is the so-called Kursiu merios Lagoon. It receives the discharge of the largest river of Lithuania, the Nemunas River. The Kursiu nerija Spit (98 km long and 0.4 to 4.0 km wide) separates Kursiu marios lagoon from the open sea. The highest coastal dunes in the entire Baltic area have developed on this spit. They reach an absolute height of 65 m. Almost half of this coastal dune ridge is mobile today. The Klaipeda Strait (400 m wide), which serves as a communication channel between the Kursiu marios lagoon and the Baltic Sea, has a mean annual discharge of about 26.5 km^3.

The nearshore zone is shallow with slopes ranging from 1:90 to 1:400. The predominant bottom sediment is sand, which, in some places, is mixed with gravel and shingle.

Usually, within the breaker zone there are 2 to 3 well-developed underwater sand bars. Exceptional are those places where the nearshore bottom consists of an abrasional platform covered by residual material, mostly gravel, shingle, and stones.

One can divide the Lithuanian coast into segments of erosion, quasi-dynamic equilibrium (longshore sediment transit), and aggradation-accumulation zones (Gudelis and Janukonis 1979). The zones of intensive beach erosion and aggradation are relatively short. Dramatic devastation of the shore zone is associated with heavy storms of long duration. During such times Baltic Sea beaches may lose up to 2 milion m^3 of sediment. Rates of beach erosion and aggradation vary in time and space.

Since the end of the Litorina maximum transgression on the Lithuanian coast, a powerful longshore sediment drift (the so-called East Baltic sediment stream) has been active. It originates on the northern coast of Sambian Peninsula and moves northward toward the Irbe Strait, which separates the Kurzeme Penisula from Saaremaa Island. The recent capacity of the East Baltic sediment stream is estimated as from 50 to 80.000 m^3 in the south, to about one million m^3 in the Irbe Strait area (Gudelis 1985). The structure and dynamic properties of this longshore sand stream have influenced the recent development of corresponding sea shore sectors (Knaps 1982). The presence of such a mightly longshore sand flow plays and important role in beach nourishment, and also in the siltation of navigation channels and harbous.

Anthropogenic Effects

The earliest techniques used for shore protection from storm surges and wind action consisted in the construction of sea dams and foredunes. The first attempts to construct sea dams were made as early as in 15th century (on the southern end of Kursiu nerija Spit). Regular foredune protection on the Lithuanian coast began in the middle of 19th Century and was completed during the first decade of the 20th Century.

A series of groynes for beach protection was arranged on the Kursiu nerija Spit in the 19th century and rebuilt several times thereafter. They were able to protect the eroding sand beaches within limits of 3 km.

Human impact of the natural evolution of the coastal zone is conspicuous at some places, and especially where large-scale water engineering structures (such as jetties and piers) have been built. Klaipeda Harbour, Sventoyi Harbour, and the promenade pier in Palanga sea resort are examples of anthropogenic effects.

Klaipeda Harbour

The initial harbour structures were built at the end of the 18th century. However, after some years they were demolished by storm waves. In the years 1834 to 1858 port engineering works were repeated. During that period a northern jetty was constructed and the southern one extended, reaching the length of 945 m. In 1884 the northern jetty was extended again. In 1901 and 1902 the southern jetty was further drawn to 1390 m (Żaromskis 1977).

Soon after the construction of the northern jetty in Klaipeda Harbour, the shoreline to the lee retreated about 200 m, whereas, to the south of the southern jetty, beach aggradation took place and the shoreline moved seaward about 0.5 to 0.7

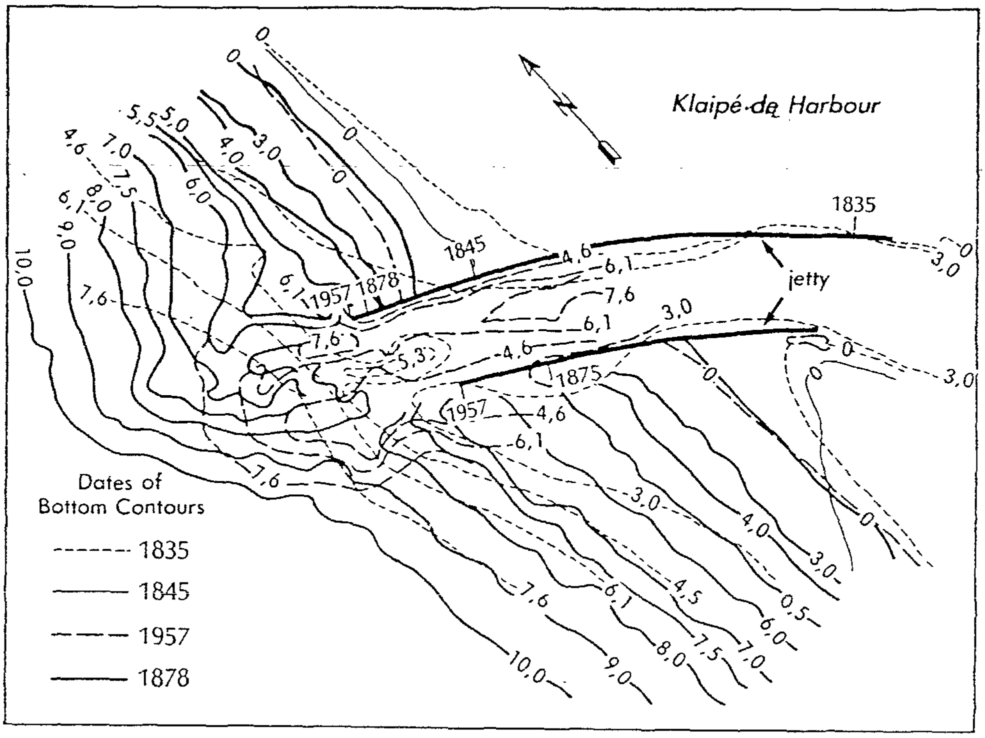


Figure 7.3. Depth Contours at Klaipeda Harbour, 1835...1957.

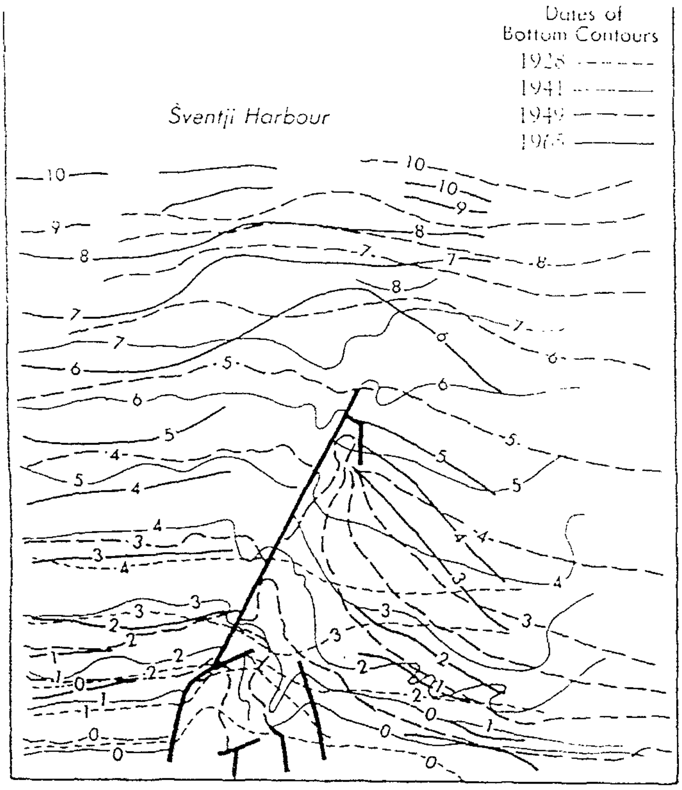


Figure 7.4. Depth Contours at Sventoyi Harbour, 1928...1965.

km. In front of the harbour entrance a vast submarine sand bar originated and continuous dredging was necessary. The land aggradation ratio, in the course of time, has been diminished. Thus, during the period of 1878 to 1957 (i.e. 80 years), beach progradation on the lee side of the Klaipeda Harbour amounted to no more than 50 to 60 m. During this time interval, farther to the north, shore erosion began (Fig.7.3). The bottom in the nearshore area was deepened 2 to 3 m and the shoreline shifted about 200 m backward, forming an erosional embayment. The accumulation of sediments to the south of the harbour stopped and in some places an insignificant wash-out occurred. During the 50 to 60 years since the construction of the Klaipeda Harbour jetties, the shoreline achieved a quasi-equilibrium state in the affected area (Fig.7.3). Silting of the entrance channel to the Klaipeda Harbour was intensified. Today approximately 0.7 to 1.0 milion m^3 of sediment must be removed annually be dredging.

Sventoyi Harbour

The harbour, constructed originally in the 17th century was destroyed by Swedish troops in 1701. However, in 1925 the Lithuanian government decided to rebuild it as a commercial and fishing harbour. During that year two jetties were constructed; a southern jetty (380 m long) and a northern one (230 m long) (Simoliunas 1933). Soon thereafter intensive silting in the harbour basin began, and the problem was more and more conspicuous. Within 10 years the shoreline grew southward from the harbour into the sea by 300 m. In contrast, to the lee of the harbour the shoreline retreated by more than 150 m (Fig.7.4). The Sventoyi Harbour basin soon became firm land.

In order to protect the harbour from siltation, a new 780 m long jetty was built and the reconstruction of the northern jetty began in 1939. Water depth at the head of the southern jetty reached about 7 m. World War II halted the reconstruction. The newly built southern jetty caused seaward displacement of the shoreline by some 10 m and the sea bottom has been deformed with a displacement of bottom contours (Fig.7.4).

The main reason for the dramatic silting of the Sventoyi Harbour was deemed the insufficient length of jetties. Even after their construction, bottom drift entered the harbour basin, especially during storms.

Sea walls (or revetments), dams, and groynes are absent on the open Lithuanian coast. However, on the lagoon shoreline of the Kursiu nerija Spit (in fishing villages of Nida, Preila, Pervalka, and Juodkrante) there are large systems of revetments, groynes, and harbour jetties. The total length of shore protection structures on the lagoon exceeds 5 km.

7.2.7 New Zealand

Long-term prediction of coastline is indeed an important strategic tool in environmental planning and shore protection. A method developed by J.Gibb from

Austrialian methods of determining buffer zones of beaches is described by Stuart (1984). The method is called Coastal Hazard Mapping and has been used to foresee where the New Zealand coastline will be in the next 100 years so as to prevent erosion destroying property and help to leave beaches in their natural state.

7.2.8 Poland

The erosion trends on the Polish coast have drawn attention to shore protection, which is aimed at defence and management of the economicall]y important dunes and cliffs, together with low agricultural lands adjacent to embayments, estuaries, lagoons or coast. Different technological means are used to achieve this goal, viz:

- faster natural growth of dunes and their stabilization using grass, trees and fascine
- artificial beach nourishment
- construction and stabilization of flood dykes
- construction of protection systems employing groynes, slope revetments, breakwaters, sea walls etc.

An outline of the Polish sea defence strategories has been provided by Basiński and Żmudziński (1988) and Onoszko (1984).

In 1960 the coasts of the open sea and the Gulf of Gdansk were protected along 20 sections measuring $88km$ altogether, i.e. 17 per cent of the entire length of the coastline. The defence structures included 822 groynes and 42 sea walls, revetments and other shore-parallel structures.

The first defence works of which we are aware were carried out in 1826 at Darlowo's breakwater, where a gravity shore wall using stone and fascine was constructed. First groynes, 40 m long and spaced every 50 – 70 m, were driven at several locations in the years 1873-1874. At the turn of this century groynes were redesigned: they became longer (to 10 m) and spaced more widely. In the thirties experiments were conducted as to groyne shape and design. The sixties have left openwork groynes. The plans to build 300 m long groynes and to protect Poland's all coast within a system of large groynes have never materialized. The longshore structures evolved similarly, from gravity retaining walls consisting of stone and concrete to leightweight temporary structures with stone and fascine. Fig.7.5 illustrates how the tendencies in the construction of groynes and longshore structures varied with changing concepts on efficiency of various types of defence works.

Tracking back the history of shore protection along the south coastline of the Baltic one sees that the defence works and policies were flexible and sometimes tailored in accordance with local interests. Zenkovich wrote in 1955 "... one can encounter coastal sections protected with gravity structures without clear economical effect and some other sections where groynes defend a forest in which the number of trees endangered by erosion in the nearest 10 ... 15 years is smaller than the number of piles in the groynes". Although the knowledge of coastal processes and protection

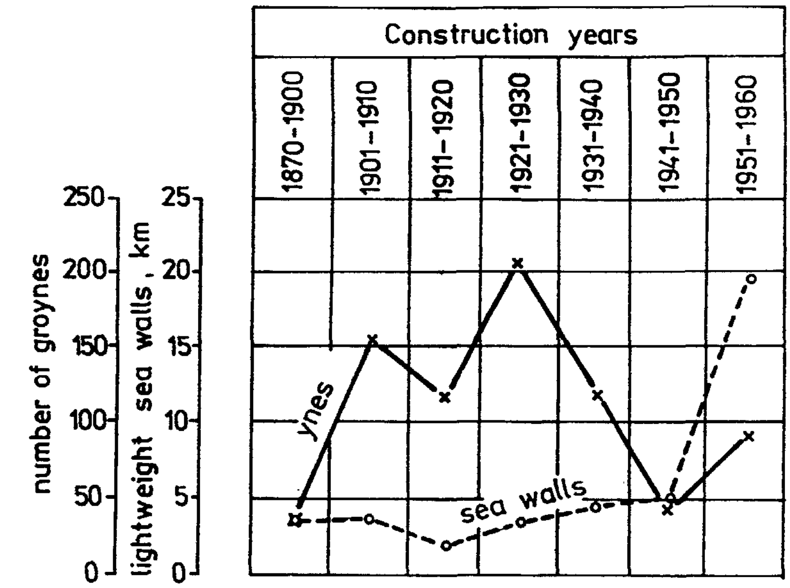


Figure 7.5. Shore Protection Works on th Polish Baltic Coastline.

mechanisms is appreciable, quite often the problem of coastal defence is a trial and error exercise, with the engineer's intuition and experience much in demand. The contemporary attitude bases on minimum interference with natural processes and use of "neutral" structures (Kowalski, 1980). Some engineers argue that groynes are inefficient or even harmful, that leightweight longshore structures must not be used at extremities of groyne groups and that sand bypassing and feeding is the optimum measure to delay erosion. A novel approach is also present that groynes bring about concentration of longshore drift in a narrow zone seaward of groyne heads, increase in current velocities and thus cause scouring of a channel between the groyne heads and the adjacent sand bar. As the bed slope in the channel becomes steep, this in turn induces loss of sand accumulated originally between the groynes. The storm floods in the years 1948, 1962 and 1983 have shown that groynes were unable to impede considerable erosion.

The evolution of shore protection tendencies can be exemplified by works along the open-sea coast of the Hel Peninsula. This spit is an accretional form subjected to erosion in its north-eastern part, particularly strong every several years due to catastrophic storm surges. The leigthweight stone-and-fascine sea walls constructed in 1936 were totally destroyed after a short time. A group of 44 groynes was constructed at Wladyslawowo in the years 1946-48, together with another group of eleven driven in 1948 about 4 *km* downdrift of the previous groynes. After execution of the works the shore became stabilized but deep erosion holes appeared at the extremes of both groups. The protection endeavoured with leightweight longshore structures did not save the dune but, on the contrary, intensified the erosion because of wave reflection and local currents between the structure and the toe of dune. Subsequently the structures were partly damaged and partly taken to pieces.

Some other erosion areas were also protected with groynes but the erosion phenomena at the extremities of the groyne group again repeated. In the course of this continuous construction of new groynes a 13 *km* long section of the Hel Peninsula was much reshaped. However, the shore was again seriously damaged during the winter storms of 1983. Experiments are under way to utilize for nourishment the sand dredged at the port of Wladyslawowo, but the groynes themselves remain highly controversial.

Heavy sea walls have been practiced since the seventies, mostly to protect the cliffs at Międzywodzie, Niechorze, Ustronie Morskie, Jaroslawiec and Ustka. The shore-parallel structures are given priority in the claiff protection while both longshore and cross-shore structures (lightweight sea walls versus groynes) are given equal chances in the protection of the Polish dunes (400 km vs 100 km of cliffs).

All construction works, after and prior to World War II, along the present Polish coastline have encompassed 990 groynes, 34.8 km of leightweight sea walls and 13.6 km of heavy sea walls, some of these units having been renovated after storms (Onoszko, 1984). Some 6 km of sea walls had to be destroyed because of their obvious harmful effect on the adjacent coastline. - Offshore breakwaters are sometimes connected with groynes to form T-groynes, such as those at Dziwnów. The history of the construction is illustrated in Figures 7.6 and 7.7. Low-cost measures are taken at places, such as at Karwia, where a rubble revetment at the toe of the dyke had to be founded on used rubber tyres, partly due to unavailability of fascine. Geotextiles are paving their way and are in wider and wider use. Artificial beach nourishment, taken up on several sites, such as Kolobrzeg Darlówek, Mrzeżyno and Hel Peninsula, has proved most successful in the sheltered waters of the Szczecin Bay.

The forecast tools available and applied in Poland to grasp and possibly control the coastal processes, including the most important one of shore ecolution, with or without protection measures, are described by Zeidler (1988 a,b) and associates (Zeidler et al., 1988).

7.2.9 Romania

In Romania the problem of intensified erosion has emerged along the major part of her coastline, as a consequence of the construction of harbours (Constanza, Mangalia, Middia) and river training (the Danube and its tributaries).

Model and field studies have been performed in Hydraulic Engineering Research Institute in Bucharest for more than twenty years to solve different problems of shore protection and beach formation or extension along the Romanian coast.

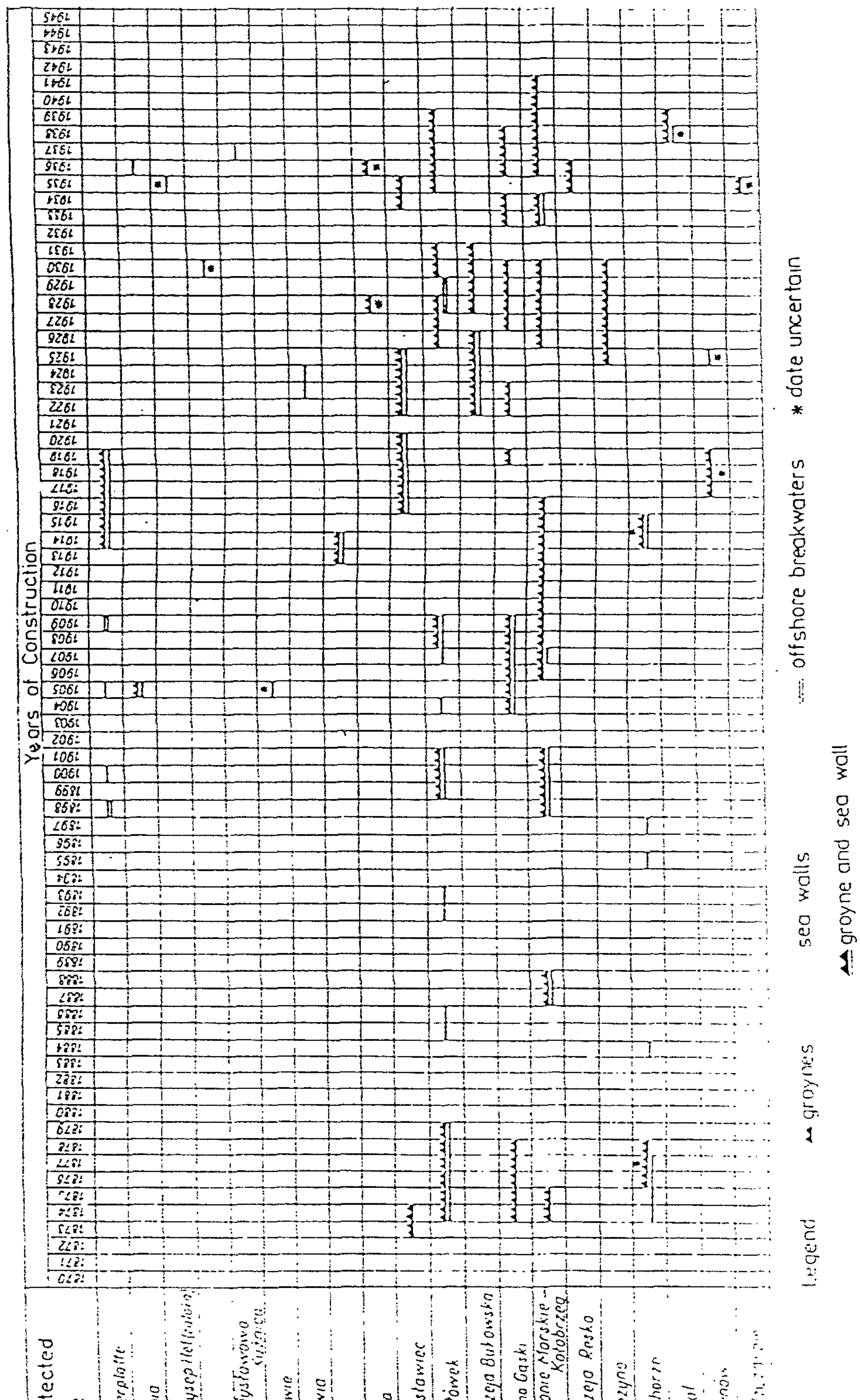


Figure 7.6. Polish Coastal Defence prior to 1945, Basiński (1963).

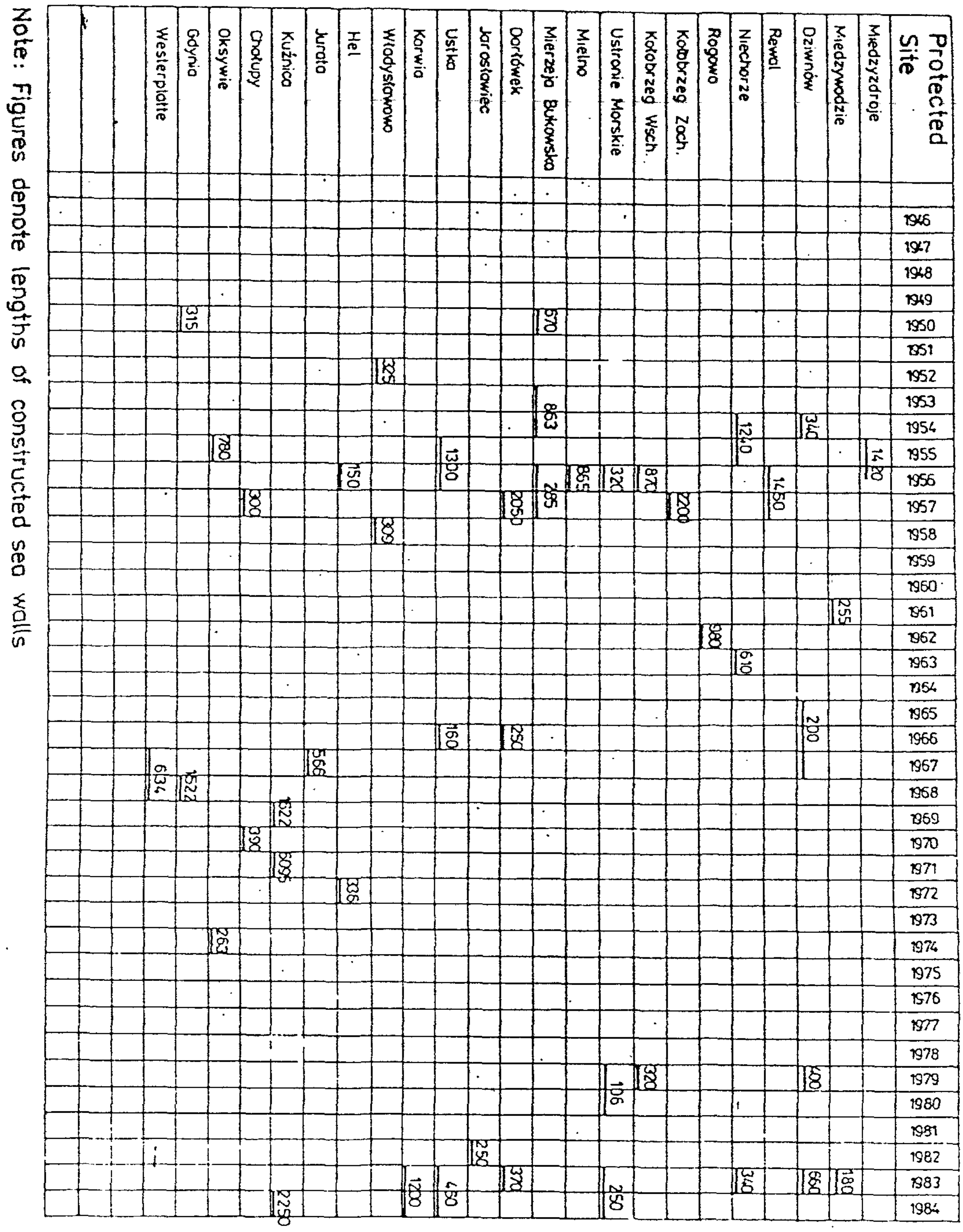


Figure 7.7. Construction of Sea Walls on Polish Coastline, Onoszko (1984).

Field observations and measurements have been used as an important element for the calibration of models and for better understanding the coastal morphological processes (Spataru & Simbotin 1983).

For long beach sectors under erosion, many experiments on large models led to the conslusion that the best protection work requires the division of the sector in several concave beaches with the aid of strong T-shaped groynes, well protected at their foundation. A shorter concave beach is easier to protect from erosion and sand losses.

For the zone of long sand spits at lake inlets, where the gaps produced by storms mean big losses of the sand transferred into the lakes, the most suitable protection work was found to be the construction of a sand dike, along the lagoon side of the spit.

The area between the spit and the dike must be divided with cross sand dams, to avoid the overflow of great water volumes during storms (Spataru & Simbotin 1983).

7.2.10 Soviet Union

The USSR strategies on coast protection and control are outlined by Khomitskiy (1983) who *inter alia* reviews the sea defence practices on the Baltic Sea and the Black Sea dating back to the early 20th century.

The first groynes in Russia were constructed in 1908-1911 on the Georgian and Crimean coastline, Zhoekvara and Yalta, respectively. A system of groynes was used in 1923-1925 to stabilize beach at Gagra. Since then groynes have been attempted at a variety of locations but the experience gathered is by no means clear, and the whole concept of groynes remains doubtful and controversial. Many other well-known measures have also been used. Artificial beach nourishment is now considered the most prospective and effective method of the protection from erosion, both in the seas and in inland reservoirs. Artificial beaches have been formed on the coastlines of the Caucasus (Gelendzhik, Poti), Crimea (Planerskoye), the Azov Sea (Eysk, Taganrog, Primorsko-Akhtarsk), and Siberian, Central-Asian, Dneper and Volga reservoirs. About 20 km stretch of coastline will soon be protected by artificial nourishment at the city of Berdyansk (the Azov Sea). It is considered economic if a most 10% of artificial beach is eroded per annum.

The method of beach protection used on the Soviet Black Sea coastline have been examined by Tsivtsivadze (1986). Particular attention is drawn to offshore breakwaters, both submerged and emerging, at different distances from shoreline. Simple formulae are proposed for evaluation of the effectiveness of these structures with regard to wave dissipation and sediment transport.

A modified version of beach fill is put forward by the Soviet engineers (Khomitskiy, 1983). A prism of rock and boulders is originally placed close to water line. It becomes washed out gradually, and its material is deposited in the inshore zone, thus helping waves dissipate their energy and defending shore indirectly. - Among other measures, segmented breakwaters have been made shore-connected along 1 km of coast at Berdyansk.

A distinction is made by Khomitskiy (1983) between sand-trapping and beach-preserving (-stabilizing) groynes. The former entrap the natural littoral drift, while the latter are created by filling the bays between groynes with mixtures of sand and gravel. The latter method was developed in the USSR in the fifties to combat the increasing erosion of the Azov-Black Sea coastline due to uncontrolled nearshore mining for construction purposes. They are in continued use, often in combination with detached breakwaters to connect groyne heads.

The adverse effects atributed to groynes in the USSR include:
(1) increasing cost of shore (up to 3 million roubles per 1 km in 1983)
(2) difficult and costly protection of groyne heads, subject to severe scouring
(3) connecting groyne heads by offshore breakwaters brings about poor water exchange
(4) environmentaal impact of groynes and T-groynes is often unacceptable from the recreational standpoint.

The effect of ports and coastal structures on the stability of coastline causes increasing concern of the Soviet environmentalists and engineers. The erosion to the south of the harbour of Sochi (the Black Sea) was possible to control only with a complex system of impermeable groynes and artificial beach nourishment. The Mokva River jetties at Ogamchire caused downdrift erosion and undermined the coastal structures at Galizagi. Among many other examples Khomitskiy (1983) cites the most spectacular example of the town of Poti, where partial mining in the Rioni River brought about disastrous erosion of the town's territory to the south of the river mouth, and subsequent accretion further downdrift, to interfere with harbour breakwater. The shoreline at Ventspils (the Baltic Sea) moved by $600m$ throughout 45 years.

The erosion on the northern coasts of the Azov Sea gives rise to siltation in the ports of Zhdanov, Berdyansk and Taganvog; the total volume of the settling sediments is estimated at 140 million m^3 per annum.

On the other hand, offshore breakwaters, jetties, groynes and other shore control measures are used extensively in the USSR to produce desirable effects, such as land reclamation, beach nourishment etc. Such breakwaters have a total length of 14 km at Sasyk; some further 20 km will be constructed upon completion of a project which encompasses the redistribution of sediment load between the rivers of Dneper and Danube, the two major tributaries of the Black Sea.

In their studies aimed at controlling the sediment transport and beach evolution by the use of breakwaters, jetties etc. the Soviet scientists have found, for the Black Sea coastline, that the seaward extremity of a jetty etc. should be on a depth of

$h_{cr}\alpha 1/6$ of wave length if about 80% of sediment transport is to be entrapped. Sixty percent of this volume of the sediment entrapped was recorded in the surf zone, and 40% on the depths between h_b and h_{cr}. Short jetties are found ineffective, quite in agreement with our findings described in Chapter Two.

Various sediment entrapment and control schemes have been devised, and the experience gained from the Soviet field studies and laboratory tests has been summarized by Khomitskiy (1983): Here are some major points:
(1) for oblique waves, sediment is arrested most effectively by a single jetty, a system of groynes or underwater trenches and basins
(2) detached breakwaters intended for diffraction and tombolo formation are best for normal wave incidence and depths greater than h_{cr}
(3) sediment bypassing is recommended if the ratio of the sediment transport transport rates on both sides of a jetty etc is greater than 3. For smaller ratios, sediment pockets must be provided on both sides of the obstacle.

Many other site-specific guidelines are also indicated by Khomitskiy (1983). A complex system of Chernomorniiproyekt at the Grigoryevskiiy liman, combining sediment entrapment, beach evolution, and land reclamation aspects and measures is one of the examples cited.

The Soviet organization of coastal management is fairly diversified. The coastline is under jurisdiction of different institutions. A recent trend is to establish administrative boards which combine the tasks of coastal management in the entire field of strategy, design, construction and maintenance of regional systems and measures. One of these boards operates in the Soviet Georgia ("Gruzberegozashchita") and another holds exclusive responsibility in the Kaliningrad District ("Upravleniye Beregozashchity"). Some others are to be enacted. The one in Kaliningrad is responsible for a strip of coastline stretching 3 km into the sea and 1 km on land. They have recently been involved in a large-distance artificial beach nourishment of 2 million m^3 annually.

The list of case studies, projects, practices, and strategies in use in the Soviet Union can be enriched by the data contained in Kanarskiy (1982), Shaytan (1974), Yakovenko (1986), and other books, papers and standard documents cited (see *References*). [2]

7.2.11 Sweden

The Swedish coastline may be learned from a review made for Scania by Lindh (1979). Scania (Skane) is the name of the southern part of Sweden, ecompassing the two southern counties which have coast reaches to the Baltic and to the Sound between Sweden and Denmark.

[2]The English versions of the books by Kanarskiy, Khomitskiy, Shaytan and Yakovenko, together with some Soviet standards (SNiP) are available

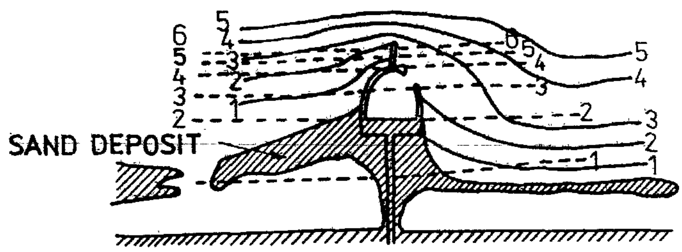


Figure 7.8. Accretion at Harbour of Skanor (Sweden).

As an almost classical example of interaction between man and the coastal zone, the consequences of constructing the harbour at the town of Skanor may be mentioned, where the expected effect has apparently failed to appear and where there has been continuous accumulation of littoral drift from the north (Fig.7.8). At the city of Helsingborg there are sand deposits at the Domsten Harbour. In the city of Malmøthere is sand accumulation in both ship channels leading to the harbour of Malmø.

On the other hand, at the beach at Ribersborg (Malmø) the shore has to be restored each year because of erosion processes.

In the municipality of Trelleborg it is reported that sand and seaweed are transported into three harbours for small boats. This necessitates continuous dredging of these harbours.

In the municipality of Ystad it may be noted that extensive erosion is taking place at both Ystad beach and at Lodetup beach. Many years ago the first erosion problem was satisfactorily solved by the construction of four groynes. At Loderup some regular funds are expended annually for private beach restoration, mainly without positive results. This is due to lack of knowledge as to how effective protection can be accomplished (usually large stones are placed along the shore).

The examples given from different parts of Scania show the results of combined human and natural activities, which are however difficult to separate. At the Loderup beach one observed a retreat of shoreline throughout the years 1918-1975, most of which occurred from 1950 to 1975. Even though it is difficult to make detailed analysis of the sediment transport (basing on the present knowledge), it seems reasonable to assume that the accumulation at Sandhammaren is due to transport from the Loderup seaside resort. There are many examples of badly planned and constructed groynes, especially along the Swedish southern coast. A situation occuring very often is one where serious erosion has developed on the downdrift side. This is sometimes also found in harbours. What is fundamentally wrong in these situations is that there is not sufficient artificial nourishment for feeding the groynes.

There exist many variants of shore protection in Scania. At Alabodarna there is no longer a beach, and the coast has been protected by means of a concrete wall and boulders. The same solution was chosen at Lerberget. The old promenade was often demolished during severe storms: today there is a wall of boulders. During heavy storms the promenade is covered by seaweed and the overtopping water is led back to the sea by holes in the wall. The western part of the south coast of Scania is also very much exposed to erosion. Fruitless attempts have been made to keep the beach intact. In an attempt to control a wanted accumulation of sand, some years ago big tubes were used as barriers in order to stop the sand transport. The result was not acceptable. In a new trial, steel sheets have been used. It is difficult to say how effective this may be (Lindh, 1979).

7.2.12 United Kingdom

The state of development of the United Kingdom protection practice is outlined in State of the Art (1985). The methods of coastal defence in use (earth wall, concrete or masonry walls, revetments, gabions, breakwaters, sheet piling, beach nourishment, groynes, grading of cliffs) are discussed together with the design and management of maintenance procedures.

Groynes have been constructed on the coastlines of Great Britain from the second half of the 19th century, with increasing activity from 1930 onwards. A variety of designs have evolved which have included permeable groynes, straight groynes with lateral spurs etc. Today the majority of groynes are rectilinear, constructed of hardwood piles and timber planking. Groyne heights are usually kept as low as possible commensurate with the needs to minimise scour, achieve required beach profiles and reduce costs. A distinction needs to be made between design for groynes on shingle beaches and for groynes on sand beaches. Further, the behaviour of groynes which are backed by sea walls or revetments needs to be distinguished from those which are backed by natural sloping beach material. For example, if a near-vertical wall is present and waves impinge on it, the beach material will usually be lost quite rapidly.

With new groyne construction or replacement it is advisable and now quite common to nourish the groyne bays artificially. In some recent coast protection schemes, mass beach renourishment has been carried out to the extent that the sea wall has been sufficiently buried to separate it completely from the active wave zone.

Although sand nourishment may need the inclusion of an existing or improved groyne system, experience suggests that for full beach recharge schemes using shingle it may not be necessary to provide groynes. Indeed, if groynes are installed in these circumstances they may create a saw-tooth configuration of the beach in plan whereas the smooth appearance of a full shingle recharge scheme without groynes may be preferable.

Generally design guidance is difficult to find, but the literature provides a review.

The most outstanding feature of design practice, a consequence of the division of responsibility of the coastline between authorities, is that coastline sections are all too often considered in siolation from the coastal system as a whole.

7.3 DUTCH DEFENCE STRATEGY[1] AND POLICIES

7.3.1 *Introduction*

7.3.1.1 *The Dutch coast*

The 350 km long Dutch coast along the North Sea is generally a dynamic coast. The dynamic coast is characterized by alternating coastal stretches of accretion and erosion resp. resulting in seaward and landward displacement retreats of the shoreline. Places of accretion and erosion also vary in time. Behind the dunes are low lying polders (very often with a ground level even below the low water line), in which millions of people live. The coastal erosion endangers the strength of the dunes as a sea-defence. Erosion of narrow dunes (at some place the dunes are less than 200 m wide) can therefore not be permitted in the Netherlands. At 40 km of coastline the dunes have no more than 10 m extra width available to cope with the erosion problem. A yearly erosion of only 1 m/year causes big problems within a few years. Since life in the Netherlands is so highly dependent of the quality of the coastal defence, structural solution for this problem must be developed and carried out.

In the past the only way to attack the erosion problem was to build groins. Several sections of the Dutch coastline are defended with these constructions (Fig.7.9). Evaluation of the efficiency of groins as tools for erosion was not stopped by these constructions. More recently (last decades) nourishment of sand beach and front dune has been practiced to compensate for the loss of sand dune to erosion.

For definitions of the various zones in the coastal defence zone is referred to Figure 7.9.

At some places it was not possible to maintain the dunes, and at those places the dunes are replaced by sea dike, mostly with very extensive bottom protection works (mattresses coverewd with stones, until a depth of 50 m below mean sea level).

Problems of coastal erosion and the resulting need for coastal defence are not new. Already in the Middle Ages the Dutch felt the need for organization of coastal defence efforts. In this period autonomous and independent authorities were formed with a special task to construct the dikes, to maintain all sea defences and to build and manage all the pumping works. These agencies, the Waterboards, still function. They collect their own taxes and have an own council and administration, which is chosen by the inhabitants of the polder. At this moment de Waterboards are responsible for maintining the dunes as a primary sea defence, and not for combating the coastal erosion. Erosion prevention is a national task.

[1]This section is a reprint from Louisse & Verhagen 'General strategy on coastal protection; the Dutch case' published in Short Course, Delft 1990

7.3.1.2 *Legal framework for the coastal defence system*

The protection of land against flooding was based upon a wide variety of regulations, laws, etc. At this moment a Bill is discussed in parliament. This 'Bill on sea-defence and river dikes' will also provide a basic legal framework for all coastal defence measures in the Netherlands. In this Bill the low lying part of the Netherlands is divided in so-called dike circles. A dike-circle is a low-lying area, which is surrounded by dikes, dunes and/or high grounds. Failure of one section of the sea-defence usually results in the inundation of a whole dike-circle. Each dike circle has a given allowable probability of inundation. In Figure 7.11 some of the 40 dike-circles of the Netherlands are presented with the allowable inundation frequencies.

The allowable inundation frequencies are laid down in the Bil on Sea-defence. The choice of an allowable inundation frequency is fundamentally a political decision. More recent studies are performed to find the optimal allowable inundation frequency, based upon the economic value of real estate and infrastructure in the dike circle. However, the values of human life, natural environment, historical and cultural values, etc. made it impossible to define the optimal value in an objective way.

It is interesting to mention that the results of these studies indicate that the economically optimal values are in the order of 10^{-4} to 10^{-5} per year. The frequencies proposed in the new Law on Sea-defence are a factor 10 higher.

In this Law it is also stated that the boundary values (such as water-levels) have to be recalculated every five year, and that dike managing authoruties have to certify evert five years that their dike still fulfills the requirements. So, they have to check the height of the dike, the quality of the slope protection, etc. In this way it its tried to prevent that the effect of climatic changes causes surprises, and dikes have to be adapted to the new situations regularly. This is the main reason that design water-levels, etc., are not given iun regulation, but only their probabilities.

7.3.1.3 *Need for a coastal defence policy in the Netherlands*

As has been already stated, care for the quality of the sea-defence as a means to protect the polders from flooding is the responsibility of the Waterboards. However it is the responsibility of the national government to the care of the 'foundation' of the sea defence. With other words, the national government is responsible for the battle against coastal erosion.

Also at locations where the dunes are wide erosion can cause problems. At those points there is of course no danger for flooding of polders during storm surges. The problem here is that the relatively wide dune area often have a very high natural values and besides at various places accommodate a number of human activities or functions. The dunes are considered as a high natural system of high national value that needs to be preserved. It is the longest more or less uninterrupted dune coast of Europe and exhibits a large variety of biotic and abiotic gradients.

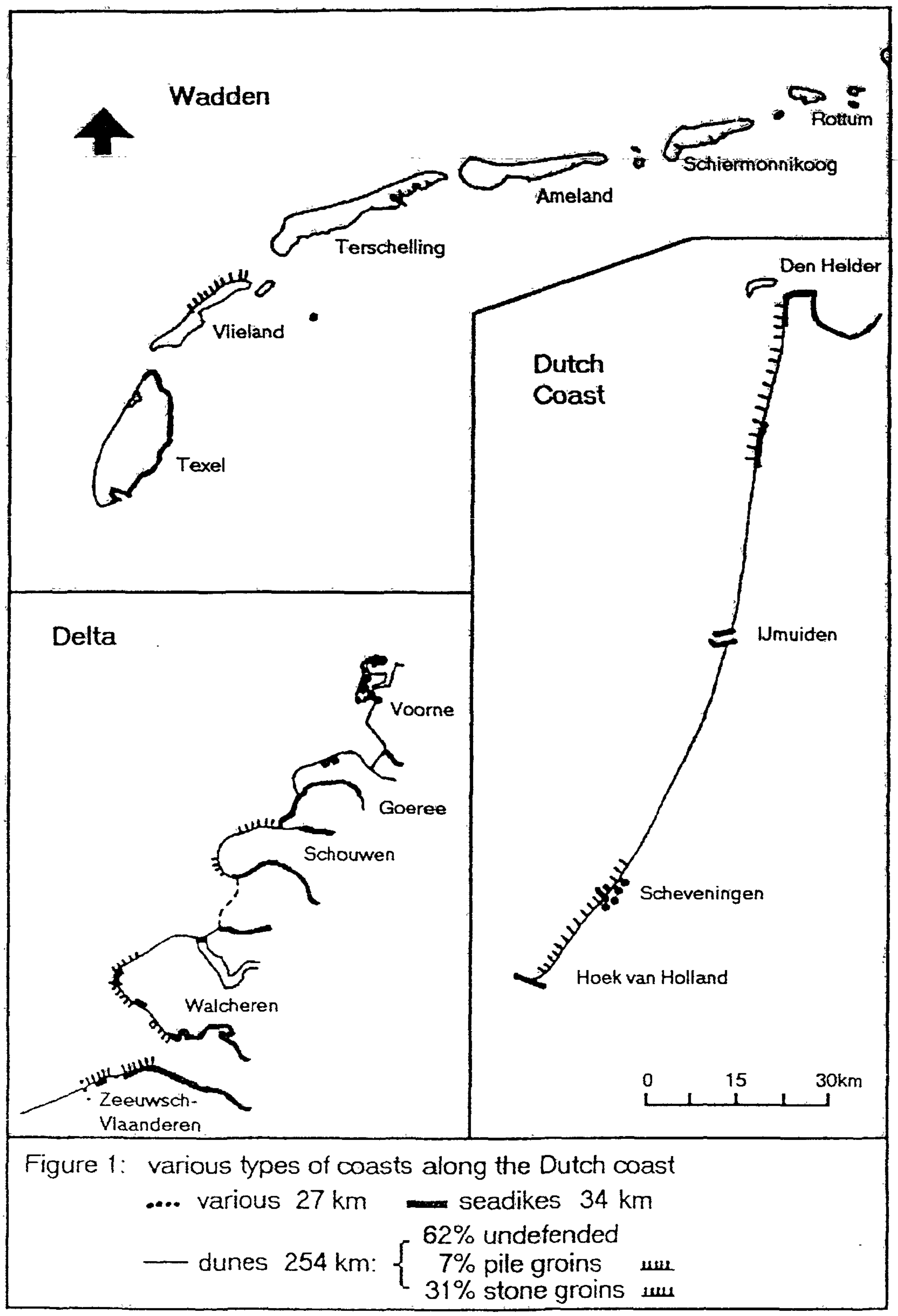


Figure 7.9. Different types of coasts along the Dutch coastline.

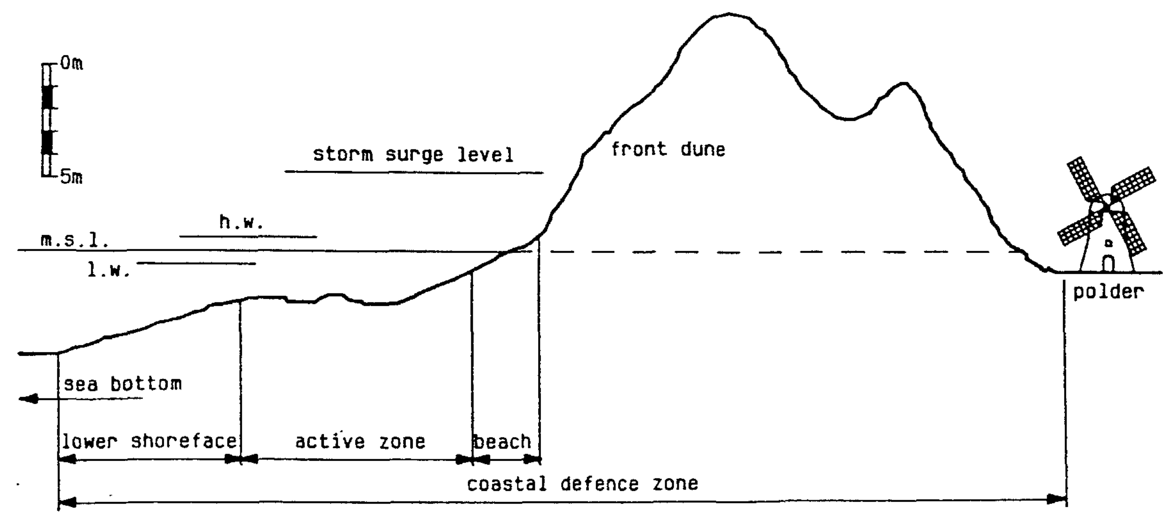


Figure 7.10. Definitions in the coastal zone.

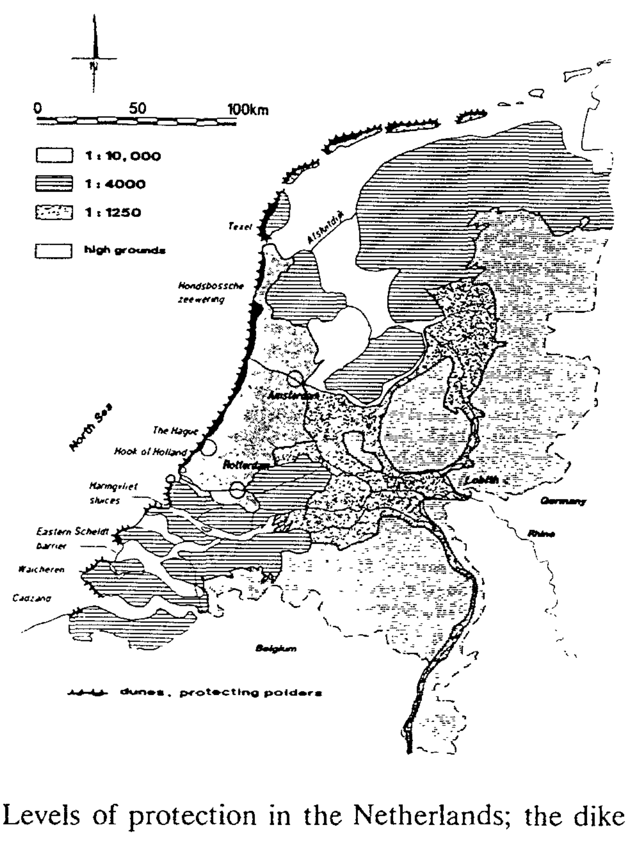


Figure 7.11. Levels of protection in the Netherlands; the dike circles are areas separated by dikes (mostly along rivers or estuaries).

Functions accommodated in the dune are: drinking water resources, recreation, living and such. At a frew places people live in the dunes. Historically dunes primary functioned as sea-defence. Therefore, many dunes became the property of the Polder Board. While some dunes are owned by drinking water companies, dunes remain as sea-defences because the Polder Boards have special jurisdiction over them, even if they do not own them. For the most part the Polder Boards do not allow construction of houses in the dunes. At present there is little private property and permanent housing in the dunes. Only temporary buildings were allowed, which must be removed before winter. Because of this policy, the dunes also became important natural reserves, although that was not the original intention. Dunes values for ecological important functions, remained in a seminatural state while the rest of the Netherlands became urbanized, or used for intensive agriculture. Beaches and dunes have very important recreational value not only for the 14 million inhabitants of The Netherlands but also for the densely populated Ruhr-area in Germany. For them, the Dutch coast is the nearest coastal recreation area. Beaches are generally not affected by coastal erosion. In principle coastal erosion only causes beach problems if a fixed structure such as a sea-wall lies behind the beach. In the dune areas there re recreational facilities such as camp grounds. Structures such as hotels and restaurants in the first dune-row are, of course, endngered by erosion.

Originally, some villages were built just behind (landward) the dunes. As coastal erosion occurred these villages became closer to the sea (several times in history this requires the removal of a village to a new location further inland). Today, in The Netherlands, demolishing houses because of coastal erosion is socially and politically unacceptable, although in some cases it would be economically acceptable. Therefore, the presence of villages near the sea requires a politicy that maintains the coastline at its present location.

In the Netherlands the dunes are also used for the production of drinking water. Because ground water in large parts of the Netherlands is brackish, is cannot be used for drinking water. In the 19th century the public water works of the big Dutch towns started to pump drinking water from the fresh-water lenses in the dunes. At this moment the natural supply from these lenses is no enough any more, and the lenses are supplied with river water (mainly from the Rhine river), which is infiltrated in the dunes and recovered later. Coastal erosion endagers the high investments in the drinking water pumping areas.

Since coastal erosion can be a treat for all these functions and for nature preservation, there is a need for clarification and qualification of this problem in relation to coastal defence.

7.3.1.4 *History*

In the past only the sea-defence aspect was very important. The authorities responsible for sea-defence therefore improved the dunes by placing more sand behind the dunes on the landward side, causing considerable damage to nature.

In order to control the erosion many groins have been build. In some cases the dunes became so narrow that is was necessary to construct a seawall or a sea dike. The consequences of the construction of aseawalls and dikes was that the safety of the polders was guaranteed, but the beaches in front of the constructions disappeared. Also the effect of groins was not quite satisfactory. In case where they were located near tidal gullies they were able to keep the tidal current out of the coastline, preventing extra erosion of the beach in that way. Near coastal sections where the tidal current is not so strong, the effect of groins in controlling erosion appeared to be minimal.
Erosion continued in spite of the construction of groins and dikes. In the south of Zeeland (southern Netherlands near Cadzand), several polders were permenently flooded due to coastal erosion in the 19th century. In many dune fields it was customary to maintain the dunes at the required strength by moving them landward. This was accomplished by mking the seaward slope of the dune-front somewhat more gentle, lowering the dune at the side of the sea, and moving the sand in a landward direction. The principle of this procedure is shown in Figure 7.10. This procedure does not stop erosion (in some cases it may even increase erosion), but the safety of flow lying polders behind the dunes ws guaranteed at the cost of the dune area. This was not considered to be a problem because of the low economic value of coastal dunes.

7.3.1.5 *Changes in Dune Appraisal*

Today attitudes are different and dunes play an important role in coastal zone management. First, better methods for determining safety of dunes were developed. In 1984 the 'Technical Advisory Committee on Water-defences' presented guidelines for the evaluation of dune safety as a coastal defence (TAW, 1986). In these guidelines a method is presented to calculate the strength of a dune during a storm surge. This method is based upon a normlized coastal profile after a stromsurge and an equilibrium of sand in this profile (Van de Graaff, 1986).
Also, new techniqes have been developed for coastal maintenance. Artificial beach nourishment has become important, partly a by-product of the dredging industry. Prices in the Netherlands are between USD1 to USD4 per cubic meter placed on the beach (Rijkswaterstaat, 1987). These low prices made it financially possible to switch from dune improvement at the landward side to improvement by beach nourishment.
A third important factor is the new regard of dune areas. Dune areas now have a much higher value because they are used as a source of drinking water (by infiltrating river water), they have a very important recreational value (camping grounds, daytime recreation) and they are critical areas from an ecological point of view (rare plants and animals, breeding grounds, etc.).
In the past years the nourishment projects were executed on a ad-hoc basis by the national government and not by the sea-defence authorities. This is caused by the fact that it are generally big projects and that sea-defence was very often not the

main reason for the nourishment plan.
Since sea-level rise must be anticipated (now 20 cm/century, in future it will be more because of the greenhouse-effect) it is to be expected that the erosion problems along the coast will increase.

7.3.1.6 *Finance*
The waterboard raise their own taxes. From this income they finance the costs of construction and maintenance of dikes and dunes. The coast of combating erosion has to be covered from the National Budget. However there is no structural budget for erosion control measures. Beach nourishments in the past have alwys been financed on and and-hoc basis. In order to be able to have a long term policy in this field, it is obvious that money for beach nourishment has to be available also in future. The only way to do that is having a special item on the National Budget for coastal defence. The coastal defence study should bring about the information on the amount of money involved with structural erosion control measures.

7.3.1.7 *Strategy*
Because in the past all attention was paid to improvement of the sea defence system, and not on erosion control, no general policy was developed regarding coastal maintenance. If the dunes were wide enough, there was no problem. Coastal erosion was only a problem, when the safety of low-lying polders behind the dunes was endangered. Because nowadays the dune areas itself are regarded as vary voluable, erosion is always a problem. From various groups from society there became a strong pressure to stop the erosion. Some 10 years ago it became therefore necessary to nourish a few beaches on places were there was no need for beach nourishment from a strictly 'sea-defence point of view'.
In other words, nourishment had to be done on places were the dunes were wide, and were there was no danger for inundation of polders. The legal framework in the Netherlands formally could not give a basis for these nourishments. Neither there was a component in the National Budget for financing such non-safety related nourishments. Finding money for these nourishments was therefore always a problem. It is clear that in the near future more mourishments will be required. For a long term policy regarding coastal maintenance, it is therefore necessary to have an fixed amount of money in the National Budget.
The real problem is how to get such a fixed amount of money on the National Budget. Otherwise it is not possible to do the job. Therefore the responsible department (Rijkswaterstaat) followed the a step-bystep strategy, using signals from society, leading to a strong public opinion favoring coastal maintenance on a national scale. The following steps were taken:

1. For a case were there was a strong pressure from the public, a detailed policy-analisis was made, to see what amount of money would be realistic to spend on beach maintenance project in relation to the lost values if erosion would continue at that place.

2. From that analysis followed that beach nourishment was a socially well acceptable solution for the erosion problem. Also tuhe costs proved to be not excessive.

3. An analysis was made of the costs of beach nourishment projects in the past. Ample publicity was given to the results. Special leaflets were made and distributed to policy makers in society.

4. The next step was that the Minister of Public Works asked for a policy analysis on coastal management. The study for this policy analysis was performed by Rijkswaterstaat, assisted by specialized institutes (like Delft Hydraulics) and universities. An inventory of all available knowledge regarding the Dutch coastline was made, forecast were made on the expected sea level rise and on the development of the coastline for the next century. Maintenance methods and costs were analyzed. This study took approx. one year and resulted in a set of 20 technical reports. On the basis of the technical reports, a discussion memorandum was produced in which 4 different management strategies were presented. These strategies were presented to parliament and other groups for discussion. See also the letter from the Minister to Parliament in the Annex to this chapter. Public hearings were held.

5. After the discussion the minister makes a choice from the alternatives and present the decision to parliament for approval. This has been done in May 1990. During the preparation of the text for this course, the final memorandum was not yet available. However, it is very likely that the chosen alternative is Full Erosion Control, with a few amendments. For example in areas were there is a wide sand bank in front of the dunes, this sand-bank will not be mainained. When approval (and the budget) is granted by parliament, the national coastal management policy is effective.

7.3.2 *Problem of the coast*

7.3.2.1 Causes of coastal erosion

The dune coast is a flexible sea defence against the North Sea. Characteristic is the continuous movement of sand in the coastal zone. There is an exchange of sand between the subaerial and subaqueous part of the coastal profile. Currents and waves move the sand from and to the shore in cross-shore and longshore direction. This process may cause a loss of sand from the sea-defence zone to adjacent coastal sections or to neighboring inlets.
Because of these processes there is a continuous movement of the borderline between land and water. Erosion and accretion alternate both in space and in time. Erosion nearly always causes problems.
There are two types of coastal erosion.

- A fast, sudden erosion of the dune front during storm surges, causing a considerable loss of sand to deeper water;

- A slowly, chronic erosion, which is not so striking, caused by sea level rise and morphological phenomena. Due to chronic erosion sand disappears from the coastal defence zone. An increase in sea level ise may cause an increasing chronic erosion. In that case also the coastal profile will adapt to the new waterlevel by moving in a landward direction.

The difference between the two types of erosion has been discussed more in detail by Van de Graaff in his contribution to the Short Course.
In the Dutch situation the fast erosion during a storm surge is a problem for the Waterboard, the chronic erosion is a problem for the National Government. Figure 4 shows the present situation of shoreline development. About half of the coast length is eroding. The coast in the Delta area is most severely attacked. The northern part of the Holland coast and Texel exhibit large erosive areas as well. In Texel, in addition is much faster than at other places.

7.3.2.2 *Impact of sea level rise*
According to our present knowledge the impact of sea level rise on the coast is twofold:

- First it causes a relatively deep underwater shore. In order to compensate that, there is a need of sand on the inshore zone. If this sand is not available from the sea bottom in a large enough quantity, this will cause a loss of sand from the dunes to beach and active zone. The sea-dunes, the row of dunes just adjacent to the beach, becomes narrower and moves in a landward direction: The direct effect of sea level rise. This effect will occur along the entire shoreline.

- The second effect is an increase of some of the erosive processes; the indirect effect. There will be an increasing demand of sand from the coastal sections neighboring tidal inlets. Because of the sea level rise the basins behind the tidal inlets will become deeper. This creates a need for sand to fill up the basins to a new equilibrium level. This sediment will partly be withdrawn from the coast. Especially if an increasing sea level rise is combined with a change in wind climate a significant increase of the erosion of the coastal sections neighboring tidal inlets is expected.

An increase of sea level rise will result in an increase of erosive coast length. Besides the sections that are already eroding now will suffer an increased erosion. For the areas near tidal inlets we expect an increase of erosion with approx. 0.5 m/year; for the other coastal sections this will be in the order of 0.2 m/year.

7.3.2.3 *Impact of chronic erosion on safety*
At this moment the dune coast along the entire Dutch coastline fulfill the requirements of a safe coastal defence system. The method of assessment of safety of dunes as a sea defence is discussed in more detail by Koster in his part of the

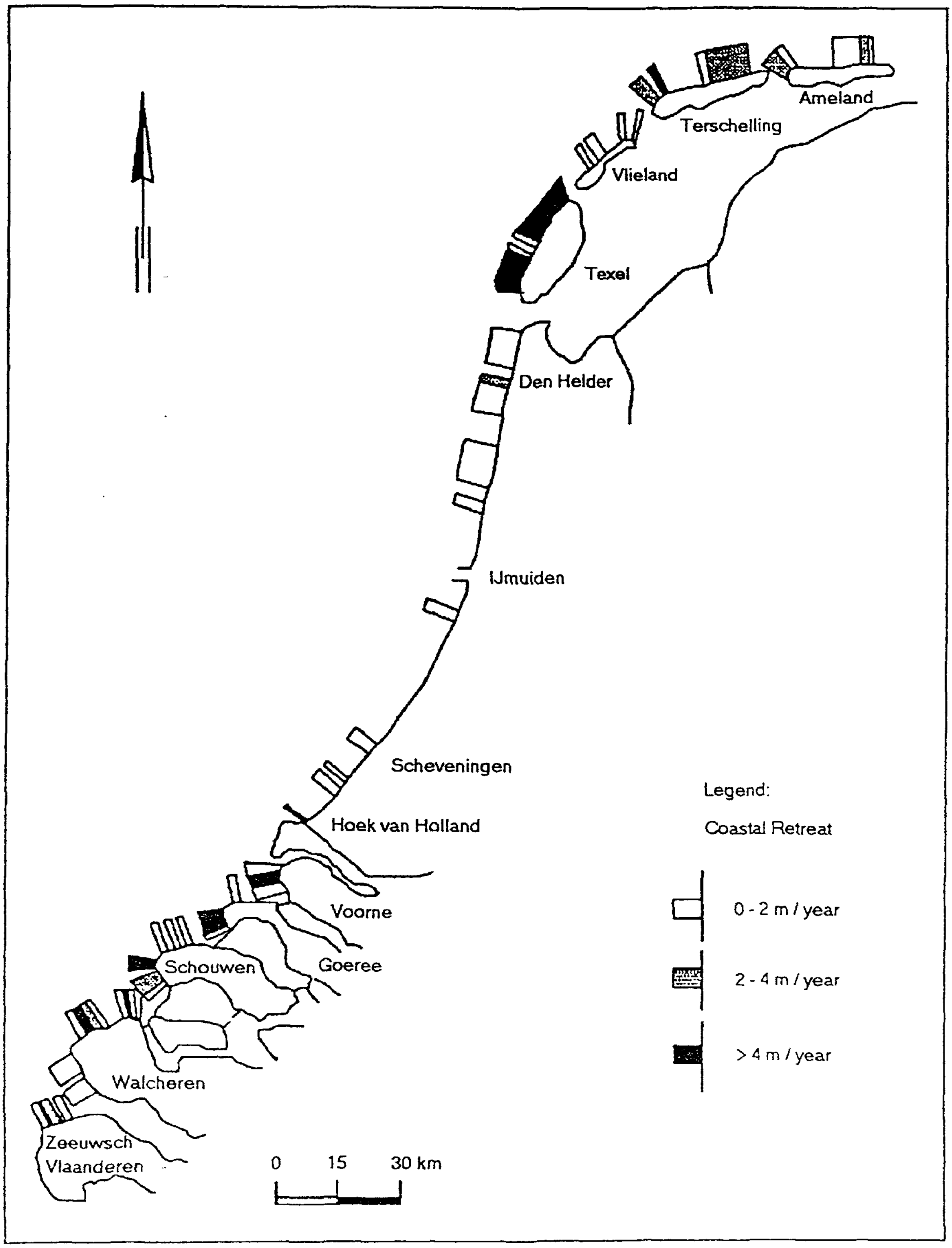


Figure 7.12. Coastal erosion in the Netherlands, in m/year.

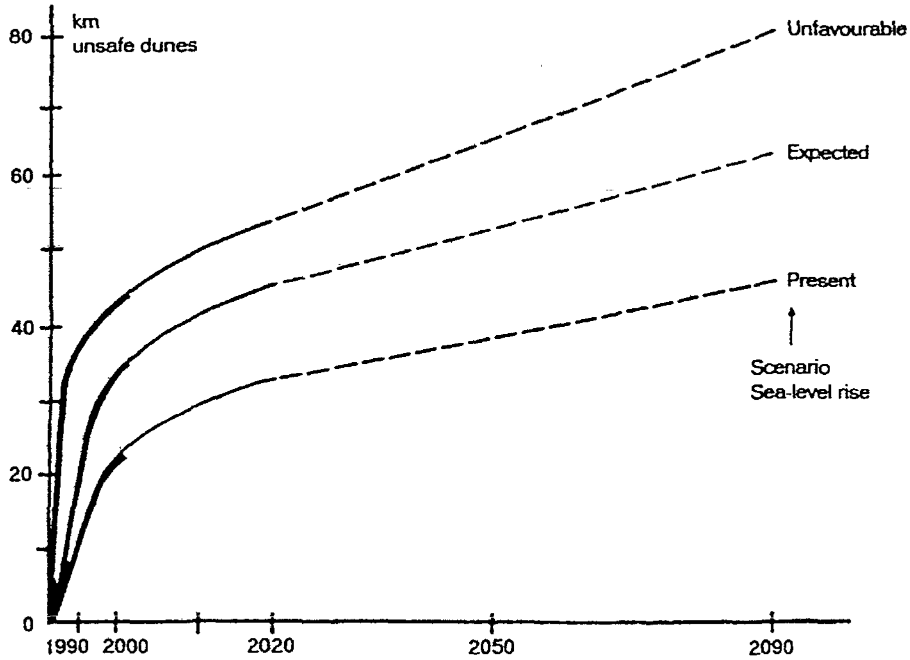


Figure 7.13. The expected length over which the dunecoast becomes unsafe in the next 100 years.

Short Course. Along the coast of Holland the dunes are able to withstand a one in 10.000 year storm.
However at some places there is hardly any spare width in the dunes where erosion can be awaited. The length of coast where safety can not longer be guaranteed is in the year 2000 about 20 km, rising to about 40 km in 2090 (Figure 7.13). For the unfavorable scenario of sea level rise an increase of about a factor 2 with respect to the present-day sea level rise must be anticipated. These effects are most severe in the Delta area.
In Figure 7.13, given on the vertical axis is the number of unsafe kilometers. P,E and U are various sealevel-rise strategies (P= present 20 cm/century; E= 60 cm/century; U= 80 cm/century + change in wind climate).

7.3.2.4 *Impact of chronic erosion on other functions in the dune area*
The dune area covers approx. 420 km^2. The effects of shoreline retreat on the dune area and functions accommodated here are also evaluated (Figure 7.14). In the year 2000 for the present-day value of sea level rise along about 40 km of coast very valuable nature area is lost. In 2090 this is increased to about 60 km, which is some 40 percent of coast length where these areas occur, in the unfavorable scenario an increase of about 50 percent with respect to the presentday se level rise may be expected.
The effects that have been reckoned with are the loss of land due to the landward

shift of the shoreline and the loss of area with a specific value (for instance wet dune valleys) that have been lost due to a landward shift of the back side of the front dune: since a minimum safety level must be maintained, a front dune with minimum dimensions must shift landward with the same pace as the shoreline retreat. Due to the landward shift of the (minimal) front dune a wet dune valley for instance can be replaces by bare sand. This can also happen to functions located behind the front dunes.

Locally and regionally the dune ecological system has a variety of qualities. Dunes with small lakes, (freshwater) marshland, dry areas, shrub, moor and woodlands are varied with 'normal' beachgrass or dry grasslands, strongly influenced by human activities.

Besides its natural values, the dune area is also of economic value as a production area of drinking water and for recreation. It is used as urban area and (at a few places) as industrial area. Most of these functions are just behind the sea-dunes. Locally one finds in the sea dunes a restaurant of some houses. Fortunately the sea-dunes are urbanized only on a few locations.

7.2.2.5 *Impact of shoreline retreat on values on the beach*

Although the beach is important for recreational use, we did not succeed in deriving a sufficient reliable parameter to express the value of the beach for this function. Therefore it was assumed that the beaches along the coast would not change as much that considerable affection to this function should be expected.

7.3.3 *Alternatives for coastal defence*

Alternatives for a coastal defence policy in any case must meet requirements concerning safety against inundation. Additionally demands concerning protection of other functions in the dunes can be made.

Objectives of coastal defence policy can be:

- To give way to natural development of the coastline where it can be admitted. Only the safety requirements are met.
- To prevent coastal retreat at certain places; safety requirements and demands for a selection of other functions in the dunes are met. In this case the problem is which other functions have to be selected. It can be housing, nature values, drinking water production, or a mixture.
- To prevent overall shoreline retreat; requirements concerning safety and all other functions are met.
- To strengthen coastal defence at weak places along the coast by constructing defence works into the sea; all requirements are met.

Starting from these four points mentioned here, four alternative policies for coastal defence and shoreline management are developed.

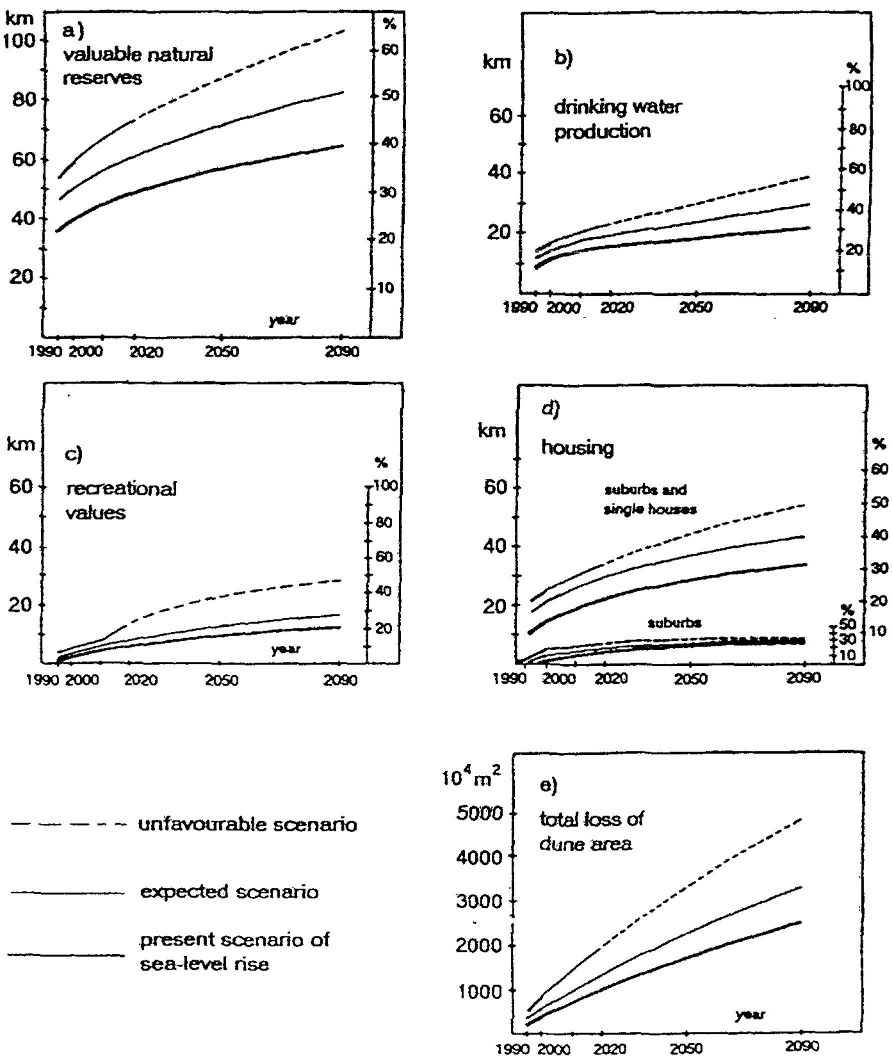


Figure 7.14. Loss of functions in the dune area when coastal erosion is not controlled. a= valuable natural reserves; b= drinking wate production; c= recreational values; d= housing; e= total loss of dune area. The three curves are the various sealevel scenarios (Fig.7.13).

In order to work out the strategies several measures have to be taken. In fact the following technical options can be used for implementing the four strategies:

- maintenance of existing constructions and dune profiles;
- nourishment of sand that disappears from the coastal profiles (erosion control);
  - at the front side of the front-dunes
  - at the back side of the front-dunes
- build new constructions for erosion control.

Four alternatives are presented to parliament and public: Withdrawal, Selective erosion control, Full erosion control, Seaward expansion.

Alternative 1: withdrawal (terugtrekken)
If nothing is done, the shoreline will erode. This is not acceptable at locations where the dunes have only marginal safety. Everywhere where the dunes are wider, erosion can be allowed until the minimal dune width is reached.
This alternative is the minimal alternative. The shoreline of the Netherlands will be determined by the natural processes. Only at those locations were the safety of the polders is in danger, action will be taken. Generally the action will be a beach nourishment, but also other solutions are possible (like the construction of a heavy sea-dike). After some years artificial headlands will be formed along the coast (the coast between the headlines continues to erode). The costs to defend these headlands will increase in due course.

If erosion continues, there is the possibility that villages in the dunes (not in the polder-area) have to be removed to a more inland location. This has happened often in the past centuries. Also damage will be caused to recreational areas, natural reserves and the drinking water production. The loss of land in the next decade will be in the order of 3.5 km^2 (800 acres) and 20 km of shoreline has to be defended by beach nourishment or other means.

In a few cases also the dunes has to be improved on the landward side. This improvement will cost approx. 100 ha (200 acres). In this alternative all sea dikes and groins will be maintained in the same way as it was done until today. A landward reonstruction of these structures has proven to be more expensive than maintenance on the present location. This alternative will cost 35 million guilders per year (16 million USD/year).

Alternative 2: selective erosion control (selectief handhaven)
The second alternative is control the erosion in a selective way. Here also safety is the primary aspect. But erosion is not only controlled in case of danger for inundation of polders, but also when important other functions are endangered. Because there are many functions in the dune area, some choices have to be made. What has to be protected, what is 'important' ? In this alternative the following

choices have been made:
- all villages in the dune area will be protected;
- natural reserves with an (international) high value will be protected;
- infiltration and production plants for drinking water production will be protected;
- investments for recreation will be protected (hotels, etc.)

The expected loss of land in the next decade will be approx. 100 ha (220 acres), 60 km of coastline has to be protected and the coast are 45 milion guilders per year (20 million USD/year). The details of this alternative have to be worked out on a regional level.

Alternative 3: full erosion control (handhaven)
The shoreline of 1990 will be maintained. Erosion will be fully compensated by beach nourishments. A small strip will be available for natural fluctuations of the beach. Nourishments will be performed on the beach, but probably also just in front of the beach, on the inshore zone. There will be no loss of land, 140 km of coastline has to be protected by nourishment. The costs are 60 million guilders per year (27 million USD/year).

Alternative 4: seaward expansion (zeewaarts)
This alternative is a more active one than the other alternatives. In this alternative the dunes which have a marginal safety are improved by making more beach in front of them. This will be done by the construction of very long groins or other constructions in the sea. This alternative has not yet worked out in such detail as the other one. The main purpose of this alternative is not the creation of extra land. Most of the constructed accretions are on locations were the do not have a high economic value. The purpose is improving the sea-defence.
Also 140 km of coastline requires protection in this alternative. The costs are approx. 80 million guilders per year (35 million USD/year).

7.3.4 *Method of analysis*

In Figure 7.15 the various issues linked up with coastal defence management are depicted. Natural development of the coast, resulting in a displacement of the shoreline, will affect utilization of the coast.
The purpose of the Dutch coastal policy is to control the process, given in the scheme and to build a legal and administrative framework for a good cooperation with all the parties concerned. All technical measures have an influence on the use of the coast.
Whether this influence is admitted or not is being prescribed by the alternatives of coastal defence. For instance, if affection of certain recreation activities is not permitted accroding to the alternative Selective erosion control, further retreat of the shoreline at a location where these activities are endangered is not permitted. In this case measures need to be taken to attain one's objective. This will result in an adaptation of the shoreline position. Costs of measures are computed.

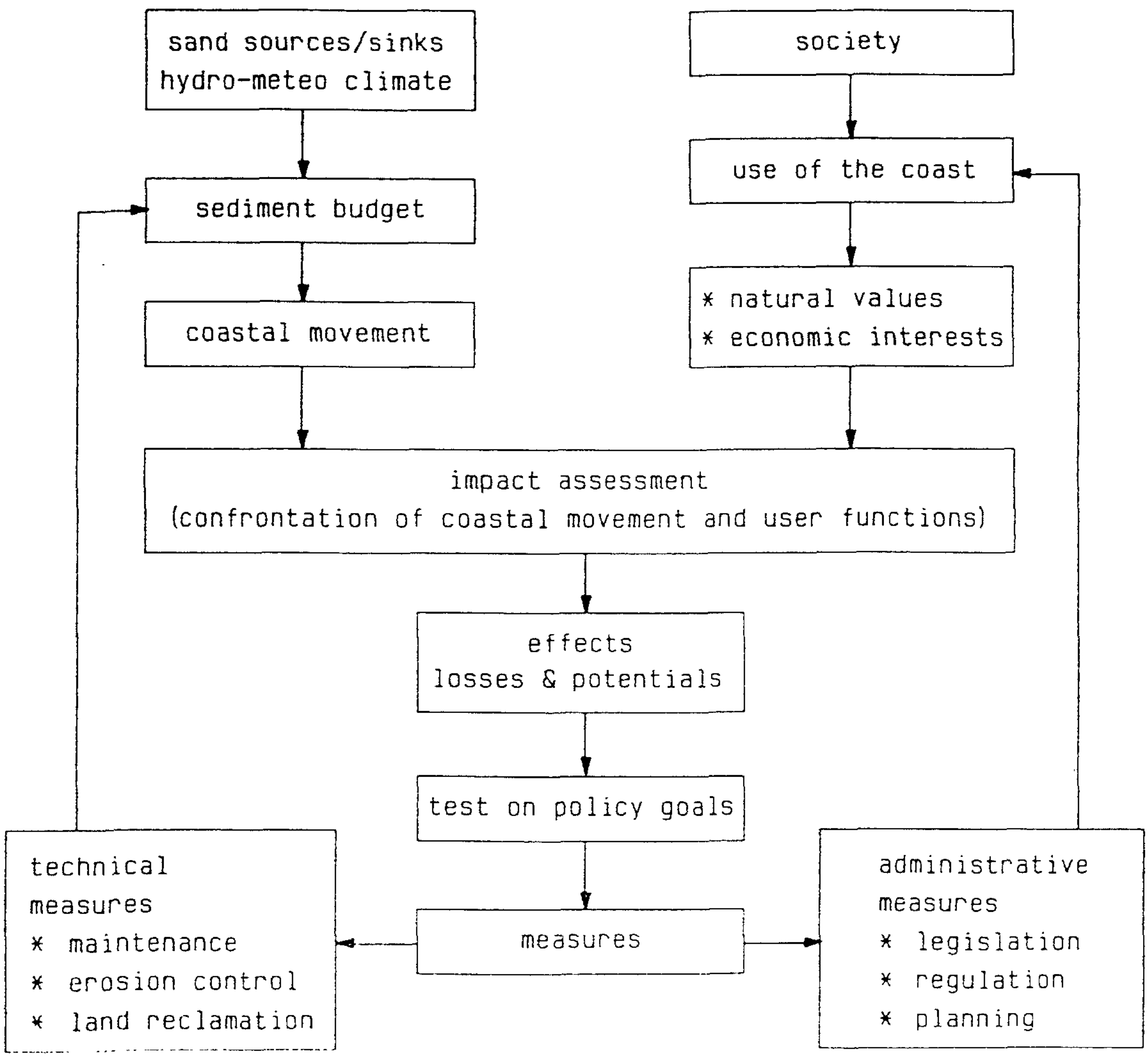


Figure 7.15. Coastal management scheme (coastal defence and coastal development planning.

This policy analysis approach was followed for evaluation of the effects of the alternatives. A computer model was developed for this analysis. here were several reasons leading to the decision of developing a model:

- the extent of the whole issue, both in space ant dime (350 grids in longshore direction, more than 10 grids in cross shore direction, 11 time steps) would ask for such a large number of actions, that failures would be easily made, when it was needed tot do it by hand;

- it was foreseen that, before deciding to chose for the alterna tives mentioned before, a large number of other alternatives should be investigated;

- since sensibility of the alternatives for variations in basic information is always very important, also in this field a number of evaluations needed to be anticipated.

In the presentation of this course attention will be paid to the steps that were followed in the evalution of the effects of the various alternativ es. Here the steps are shortly mentioned and some interesting issues will be noticed.

a. Prediction of shoreline behaviour for the next decades (up to 2090) for the various scenarios of sea level rise. Phenomenological analysis (shoreline, sand budget, long time series, trend analysis).Physical-mathematical and empirical model of coastal behaviour; different approaches for central part of the Dutch coast and the northern and the southern part. Explain model in some detail. Integration of phenomenological and model analysis to shoreline predictions.
b. Gathering of data concerning utilization of the dune area; choice of resolution; subdivision into a number of classes.
c. Identify the consequences of shoreline displacement for safety conditions of the coastal profile along the coast at each time interval; take measures when safety can no longer be guaranteed.
d. Identify the consequences of shoreline displacement for functions in the dune area; where will loss of valuable dune area occur ? Show the relation with coastal defence alternatives.
e. Compute the amount of sand needed for nourishments. Compute the costs for this sand and for maintenance activities. Dependence of costs for maintenance form type of coast and velocity of shoreline retreat.

7.3.5 *Effects of coastal defence alternatives*

The various alternatives have different implications for a series of criteria which are relevant for mutual comparison of the alternatives. These criteria are:

- length of the shoreline where measures need to be taken to guarantee safety;

- loss of dune area with economic cunctions of valuable nature area; although the effect of landward displacement of the shoreline will carry over into the

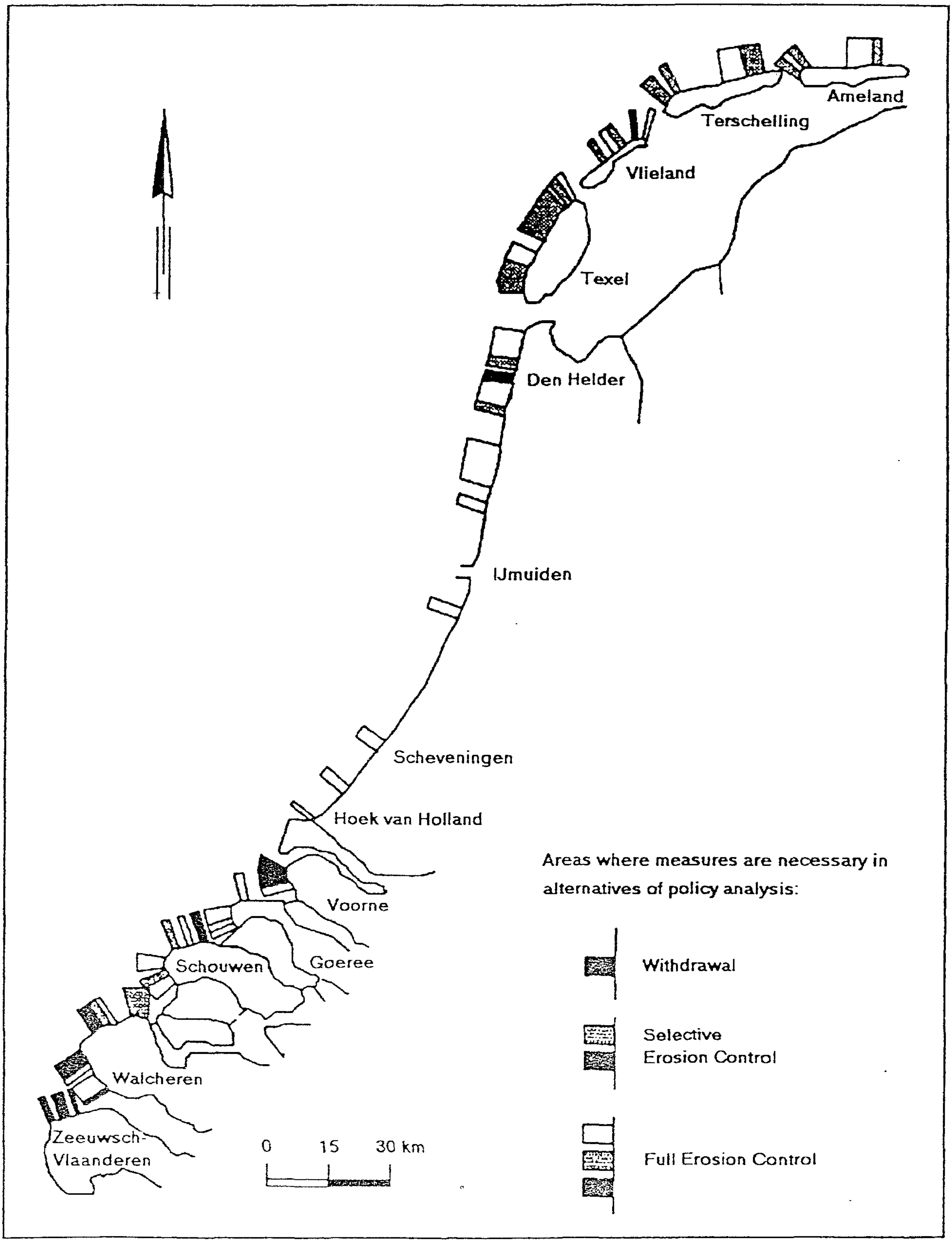


Figure 7.16. Areas where measures are necessary for the various alternatives (T= withdrawal; S= selective erosion control; H= full erosion control; Z= seaward expansion).

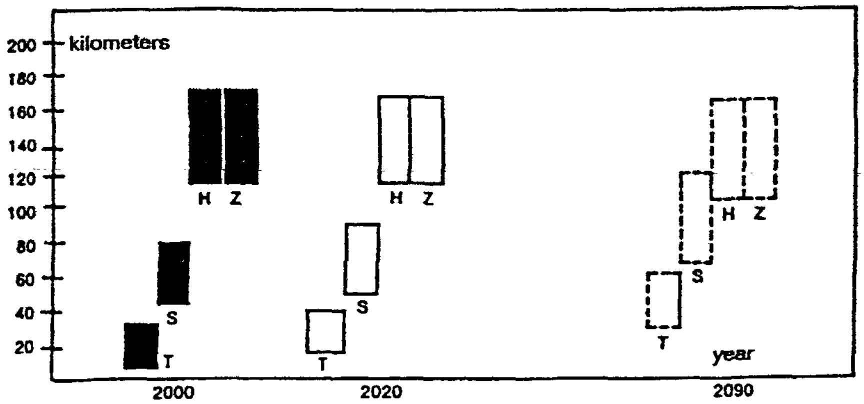


Figure 7.17. Length of the coast where coastal erosion has to be controlled.

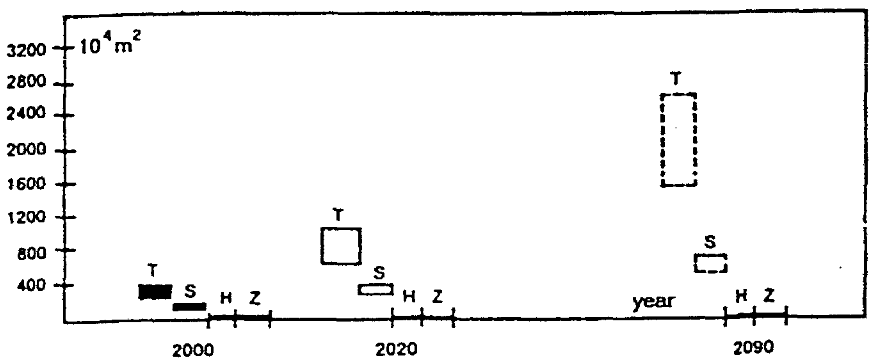


Figure 7.18: Total loss of important dune area

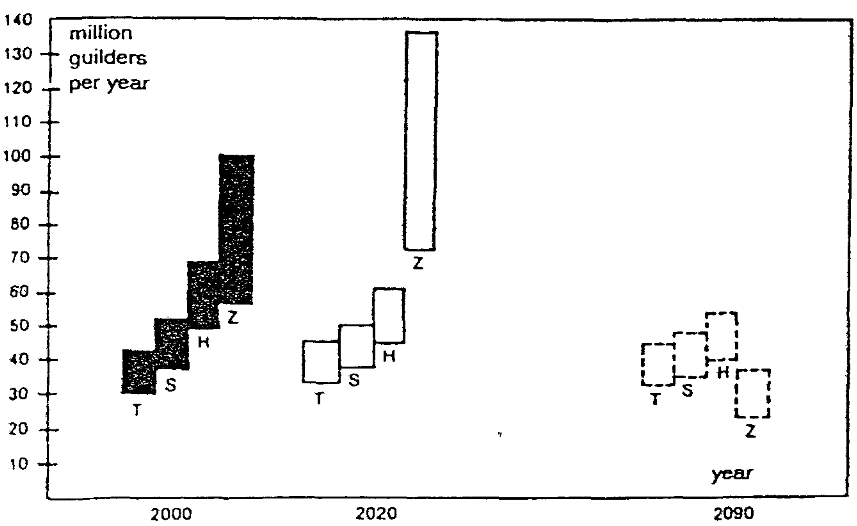


Figure 7.19: Costs of maintenance and erosion control for the total coast (in million guilders per year).

total width of the dune, only changes in the position of the front dune, were considered. Other effects, for instance the effect of changes in the ground water level in the dunes, are not described.

- costs for coastal defence measures.

The basic analysis was performed for the present-day sea level rise of 20 cm/century. The other scenario's discussed separately.
The distinction between the various alternatives with respect to measures needed for erosion control is illustrated with Figure 7.16, where places with measures which are necessary before the year 200 along the coast are indicated. For the alternative Withdrawal (T) the largest effort is concentrated at the Delta coast, where coastal defence is relatively weak. For alternative Selective erosion control (S) places with with measures are located in Wadden (to protect results (H) in a further increase of locations where measures need to be taken to actually all places that suffer shoreline retreat.
For the year 2000 for alternative T about 20 km of shoreline requires protection; for alternative H this is already about 140 kilometers. The bars indicate the variation between the results under slightly favorable conditions (average values for shoreline prediction, favorable assumptions for analysis procedures) and results under unfavorable conditions (both unfavorable shoreline predictions and assumptions for analysis). The length of coast that needs to be protected increases for alternative T as a function of time (Figure 7.17). They amount to about 18 million USD per year for alternative T, of which about 5 million USD for erosion control. Alternative H shows a rather constant level of length of coast that need to be protected as a function of time (Figure 7.17). The costs of this alternative amount to about 30 million USD per year in the year 2000 (Figure 7.18). Because all sand that disappears from the coast due to erosion must be replaced in this alternative it is more expensive than alternative T.
Giving berth to shoreline retreat (T) also esults in large losses of dune area: about 3.5 km^2 in 2000. On the contrary, full erosion control (H) does not lead to any loss of dune area (Figure 7.19).
The alternative Selective erosion control (S) leads to intermediate effects, both with respect to length of coast where measures need to be taken (60 km in 2000), as to costs (23 million USD per year in 2000) ant losses of area (1.5 km^2) in 2000). The alternative seaward expansion (Z) has not been worked out in such detail as the other alternatives; the estimates of the costs are therefore more tentative and amount to about 40 million USD per year (in 2000).
Acceleration of sea level rise from 20 to 60 cm/century results in an increase of costs for measures against erosion and loss of dune area of 25 percent. The extra costs and losses of dune area for a scenario of 85 cm/century, including changes in wind and wave climate, amount to 70-100 percent with respect to the case of 20 cm/century sea level rise for the year 2000 (see table). From this table it is clear that for coastal maintenance the rise of the sealevel itself is not a big (financial)

Table 7.3.A. Increase of several values in percent relative to the sealevel scenario of 20 cm/century.
(For example: If the sealevel rises with 60 cm/century, the cost of erosion control in 2090 is 40 percent more than the erosion control costs in 2090 with a sealevel rise of 20 cm/century).

Increase in % of Year (sealevel rise)	Amount of km to maintain	Cost of erosion control	Total cost of coastal maintenance	Loss of area
2000 (60 cm)				
T	45	25	10	20
S	35	20	10	25
H	40	25	15	-
2000 (85 cm + Wind)				
T	80	100	55	70
S	65	80	60	80
H	75	80	65	-
2020 (60 cm)				
T	30	25	10	20
S	35	25	15	30
H	60	30	20	-
2020 (85 cm + Wind)				
T	85	100	25	75
S	80	90	35	90
H	95	100	30	-
2090 (60 cm)				
T	60	40	10	25
S	40	45	10	25
H	40	30	10	-
2090 (85 cm + Wind)				
T	105	160	30	80
S	80	145	45	75
H	75	120	50	-

T - Withdrawal; S - Selective erosion control; H - Full erosion control.

problem. However, changes in the wind and wave climate (especially changes in the average wind direction), have considerable financial impacts.

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APPENDIX: Letter from the Minister of Public Affairs to parliament, announcing the new coastal policy:

To: The chairman of the Second Chamber
of the Parliament ('House of Commons')
la Binnenhof
2513 AA THE Hague
The Hague, May 9th, 1989
subject: Memorandum Sea defence after 1990
Mr. Chairman,
The protection of our country against the sea is of greatest importance. This is especially so for the more specific field of protection of the North Sea Coast. Now it becomes more and more clear that the rate of sea level rise will increase in the future years, it is to be expected that also the attack of the sea on our coastlines will increase. First of all the higher waterlevel causes a higher pressure on our sea defence along the coastline. Also follows from recent research that probably also the rate of erosion of the coastline will increase. This was one of the arguments of my conclusion that Water Authorities had to manage the sea defence structures along the coast, but that the care of the position of the coastline itself should be the responsibility of the National Government. I informed you on this subject in my letter of May 7th, 1985 (TK 18975, no.1). In the Commission Meeting of April 6th, 1987 Parliament supported this choice. The task of the National Government related to the care of the coastline will have a legal basis in the Law on the Sea defence and Flood Control (Wet op de Waterkering), on which the High Council of State will advise in short terms.

It is necessary that, before the Law on Sea defence takes effect, a long term policy has been developed for the management of the coastline. A strategic choice has to be made regarding the sea defence policy, also regarding the increased rate of sea level rise and the increased erosion rate caused by that fact. For that purpose in my department the memorandum 'Sea defence after 1990' (kustverdediging and 1990) has been prepared.
The preparation takes place in three phases:
a Production of a discussion-memorandum, in which an analysis is presented of the problems of sea defence and on which basis four policy altenarives are presented.
b Advise by the Advisory Council for Watermanagement and by the Technical Advisory Committee on Water defences. Also there will be discussions with the representatives of the provinces and the Waterboards.
c Making a choice by the government from the alternatives. This choice will be presented tot parliament in a policy-memorandum.
This memorandum will be send to zou at the end of this year.
Phase a is completed now. Today I have sand questions for advise to the Advisory Council for Watermanagement and to the Technical Advisory Committee on Wa-

ter defences. I have asked the Advisory Council to make arrangements for public hearings on this subject.

Policy alternatives

The policy alternatives presented in the discussion-memorandum are roughly worked out on a national level. This is done because after selecting one alternative by the government on a national level, furselecting detailing has to be done on a regional level, using local knowhow of provinces and water boards. This can be done by preparation of actual coastal plans by specialized regional (discussion) boards. The memorandum 'The sea defence along the Dutch coast' (parliamentary year 1976-1977, no. 14481) did already describe these boards and the Law on Sea defence will give them a legal basis.

The policy-alternatives give the boundaries within which the details have to be worked out. By making the alternatives attention has been focused on those parts of the coastline which are eroding. This has been done because the problems of eroding coastlines which possible threat to sfety or loss of important dune areas is more important than the problems of accreting coastal sections. For the same reason reclamation is also not placed into the discussion. Reclamation projects require - if demands on a good coastal management are fulfilledmainly a spatial planning process.

The alternatives can be characterized as follows:

I. WITHDRAWAL. Coastal erosion is principally accepted. Only in those coastal sections were erosion may cause inundation of low-lying polders behind the dunes, coastal erosion will be controlled.

II. SELECTIVE EROSION CONTROL. Besides locations were the polders are threatened, erosion is also controlled at those locations where considerable values in the dune area or on the beach are threatened by coastal erosion.

III. EROSION CONTROL. Everywhere the coastline will be maintained at its present location.

IV. SEAWARD. On some very eroding areas and relatively weak spots constructions will be build in the sea, which change the eroding trend of the coast into a more accreting trend. Everywhere else the coastline will be maintained at its present location.

Consolidation of the reached level of safety-the purpose of the Bill on the Sea defence-implies that withdrawal is the minimal alternative. The alternative Erosion Control is identical to erosion-stop-policy, sketched in my letter TK18975/3 of March 25th, 1988. The basic thoughts behind the first three alternatives is that coastal erosion has to be decreased or stopped. De basic thoughts behind the last alternatives is that coastal erosion locally is changes in coastal accretion. This requires specific constructions to be built in front of highly attacked coastal sections. These construction are also complementary to the other alternatives. These works have the only purpose to protect the coastline against erosion. They may have an interesting 'by-product', such as some reclaimed area, like the areas on both sided of the harbor moles of IJmuiden. The constructions can be build adjacent to reclamation works, which are mainly situated on stable or accreting coastlines; there coastal defence is not the primary objective.

present policy

In the following the various alternatives are compared with the present policy. The present policy consists of the compon ents:

1 By the construction of dune improvement works according to the Delta Law (Law which states that all sea defences has to be reinforced in order to guarantee a certain level of safety) always a certain sand-buffer has been formed to cope with the erosion for a number of years. Implicitly the alternative Withdrawal was used in those cases.

2 Additionally on ad-hoc basis a number of beach replenishment projects were executed on coastal sections with high values in the dune area or on the beach. As examples can be mentioned the nourishment works on Texel, near Westerschouwen and additional to the sea defence works near Cadzand.

Both components together form the 'bottleneck-policy', sketched in my letter TK 18975/3 of march 25th, 1988. The philosophy behind the present bottleneck-policy is equal to the alternative Selective Erosion Control. Executing this policy was limited by shortage of budgets, and had therefore a strongly ad-hoc character. Future policy should have a more structural basis.

no policy choice yet

All social effects of the alternatives have to be considered. A choice has to be made between more effort for coastal defence and losing less dunes with high values for society, or making less effort and losing more dunes. Safety against inundation of polders is guaranteed in any case. I have not yet made a choice. To prepare a choice a period of consultation and discussion is foreseen. It is my intention that the council of ministers makes a choice at the end of this year, which choice will be presented to parliament in a policy-memorandum.

Sincerely,
The Minister of Transport and Public Works,
N. Smit-Kroes

This ends our quotation of Louisse and Verhagen in Section 7.3.

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Appendix 2-1.
Recommendations on Groyne Parameters in Different Countries

Table 1. Length of Groins (Balsillie & Berg, 1972)

<table>
<tr><th>Investigator, Yr</th><th>Site</th><th>Type</th><th>Length/remarks</th></tr>
<tr><td>Owen, J.S., and Case G.O., 1908</td><td>England - General</td><td>Theory, exper.</td><td>Extend at least to low water line</td></tr>
<tr><td>Case, G.O., 1915</td><td>England, New York, New Jersey</td><td>Field - Experience</td><td>Long groins recommended - should extend from high to low water, and seaward of low water if possible.</td></tr>
<tr><td>Kressner, B., 1928</td><td>Germany</td><td>Movable bed, model study</td><td>Decreasing length downcoast: proposes principle of shortening groins downcoast at a small angle (4-6^o) to insure that downcoast beaches receive littoral drift.</td></tr>
<tr><td>Coen-Cagli, M.E., 1932</td><td>General</td><td>Theory - Experience</td><td>Pebble beaches: 40-50 meters seaward from shoreline should be sufficient length. Sandy beaches: extend to 2 or 3 meter depth; will interupt most of littoral drift.</td></tr>
<tr><td>Brown, E.I., 1939, 1940</td><td>General</td><td>Experience</td><td>Should extend at least to 6 foot depth below mean sea level. Under normal conditions, 80% of sand movement takes place at depths of less than 6 feet.</td></tr>
<tr><td>Duvivier, Jack, 1947</td><td>England</td><td>experience</td><td>Utilize shortest length possible to stabilize the beach. Has used short groins between sets of longer groins.</td></tr>
<tr><td>Frech, F.F., 1948</td><td>New Jersey</td><td>Field</td><td>Extend to 6 feet depth of water.</td></tr>
<tr><td>Nagai, Shositiro, 1956</td><td>Japan</td><td>Movable bed, model study</td><td>Optimum distance groins extend from shore line seaward 40% of distance from shoreline to breaking point of plunging breakers where wave steepness is ... = 0.01 0.02. Tests showed this gave greatest amount of deposition downdrift of groins, least updrift scour at groin sides and ends.</td></tr>
<tr><td>Horikawa, Kiyoshi, 1958; Horikawa, K., and Sonu, C., 1958</td><td>Japan</td><td>Movable bed, model study</td><td>Should extend seaward from shoreline 40-60% of distance from shoreline to breaking points of plunging waves.</td></tr>
<tr><td>Hiranandini, M.G., and Gole, C.V., 1961</td><td>Cochin, India</td><td>Field Study-prototype conditions, model study</td><td>200 foot minimum length preffered. Short groins defined asless than 150 feet.</td></tr>
<tr><td>Lee, C.E., 1961</td><td>Great Lakes</td><td>Field study</td><td>(64% 100ft., 21%: 100-150 ft., 15%: 150 ft.) of 841 groins examined, these percentages reported for Great Lakes; a predominance of short groins.</td></tr>
<tr><td>Rayner, A.C., and Rector, R.L.,1961</td><td>Great Lakes</td><td>Experience</td><td>Principal benefis of short groins (100' or less) on the Great Lakes is to retain a narrow protective beach from material eroded from bluffs, in order that further bluff erosion be retarded.</td></tr>
<tr><td>Kemp, P.H., 1962</td><td>England</td><td>Movable bed model study</td><td><table>
<tr><th rowspan="2">Groin Length</th><th colspan="3">Groin Orientation from Normal</th><th rowspan="4">Quanties of material collected in downdrift traps as% of total material after 3 wave cycles.</th></tr>
<tr><th>30° Updrift</th><th>Normal to Coast</th><th>20° Downdrift</th></tr>
<tr><td>Long</td><td>43</td><td>50</td><td>55</td></tr>
<tr><td>Short</td><td>67</td><td>71</td><td>67</td></tr>
</table>
High, impermeable groins used.</td></tr>
</table>

Table 1 (continued). Length of Groins (Balsillie and Berg 1972)

<table>
<tr><th>Investigator, Yr</th><th>Site</th><th>Type</th><th>Length/remarks</th></tr>
<tr><td>Bruun, Per, and Manohar, Madhav, 1963</td><td>North Sea</td><td>Field Experience</td><td>Most effective when extended out to depths of 12 to 18 feet of water.</td></tr>
<tr><td>Ishihara, Tojiro, and Sawaragi, Toro 1964</td><td>Japan</td><td>Fields study</td><td>Observed results showed that structures of T-groin design should be at least 60 meters in length.</td></tr>
<tr><td>Dunham, J.W., 1965</td><td>California</td><td>Field</td><td>Notes use of long groins to from artificial headlands to permanently hold sand, regardless of effects produced along adjoining segments of the shore.</td></tr>
<tr><td>Shore Protection, Planning and Design, Coastal Engineering Research Ctr., T.R. 4, 1966</td><td>General</td><td>Manual</td><td>Correct length is dependent upon prediction of ultimate stabilized beach profile. Original beach profile, conditions of littoral drift, refraction patterns, desired beach width are some of the factors utilized in methods for designing groins. Groin Type Depth to which extended below MLLW Amount of Littoral Drift Interrupted High 10' or more 100% High 4 to 10' 75% Low 10' or more 75% High 1 to 4' 50% Low <10' 50%
<table>
<tr><th>Groin type</th><th>Depth to with extended below MLLW</th><th>Amount of Littoral Drift Interrupted</th></tr>
<tr><td>High</td><td>10' or more</td><td>100 %</td></tr>
<tr><td>High</td><td>4 to 10'</td><td>75 %</td></tr>
<tr><td>Low</td><td>10' or more</td><td>75 %</td></tr>
<tr><td>High</td><td>1 to 4'</td><td>50 %</td></tr>
<tr><td>Low</td><td>< 10'</td><td>50 %</td></tr>
</table>
</td></tr>
<tr><td>Barcelo, J.P., 1970</td><td>Portugal</td><td>Model study, movable bed</td><td>Inclined groins must be extended for conditions of same spacing, e.g. where $\alpha_0 = 20^\circ$, inclined angle $= 70^\circ$, inclined groins must be 30% longer than corresponding normal groins.</td></tr>
<tr><td>Kolp, Otto, 1971</td><td>Fischland, Zempin, Nevendorf</td><td>Fields, Experiental</td><td>Generally recommends use of long groins, short groins fail to trap and retain sand efficiently; use of artificial fill in conjunction with long groins.</td></tr>
</table>

Table 2. Height of Groins (Balsillie and Berg 1972)

Investigator, Yr	Site	Type	Height/remarks
Owen, J.S., and Case. G.O, 1908	England, General	Theory Experience	Low, to allow some sand to pass over the structure.
Case, G.O., 1915	England, New York, New Jersey	Field	Groins should not exceed 2-3 ft. in height and should follow low profile of the beach. Low, long groins of adjustable type are recommended. High groins are too costly; stop all littoral drift and load to crosion downdrift of groins.
Dent, E.J., 1931	East Coast United States	Field	Low groins recommended - allow sand to drift over the structure and maintain leeward beach.
Coen-Cagli, M.E., 1932	General	Theory, Experience	1 meter height above high tide sufficient for outer portion of groins on a pebble beach. Beach Section: 50-60 cm above original beach level *(all: Recommendations for groins maintaining a sandy beach !!)*. Intermediate Horizontal Section: 50 cm above low tide level. Inclined End Section: end should be 50 cm. Lower than horizontal intermed. section.
Brown, E.I., 1939; 1940	General	Experience	Horizontal Beach Section: berm height. Sloping Intermediate Section: berm height to below low water line; slope of groin top slightly less than natural beach slope. Outer Section: gentle underwater slope seaward.
Evans, O.F., 1943	General, United States	Field	Prefers low groins as they reduce scour on downdrift side caused by wave overtopping. Recommends gradual decrease in height seaward; and should be at low water level or somewhat lower.
Duvivier, Jack, 1947; 1949	England	Experience	No higher than 3 ft. above beach level; as groin fills, it should be gradually heightened; higher groins cause scour.
Jones, J.H., 1948	United States	Movable bed, model study	Low, impermeable groins did not trap and retain as much material as high, impermeable structure.
Brater, B.F., 1953	Great Lakes	General Theory Field	1 ft. above high lake level has given good results.
Basic Coastal Model Hydraulic Research, London 1957	England	Movable bed, model study	High, impermeable groins closely spaced (1:1) arrested greatest amounts of drift, but caused sand to be croded from upper beach. Reduced drift to 1/8 of former value. Low, impermeable groins widely spaced (1:2) arrested 1/2 of littoral drift, but upper beach did not erode. Recommends usc. of low groins over high structures.
Hicker, D.F.,	General	Theory	Recommends usc of high groins (see text).

Table 2 (continued). Height of Groins (Balsillie & Berg 1972)

<table>
<tr><th>Investigator, Yr</th><th>Site</th><th>Type</th><th>Height/remarks</th></tr>
<tr><td>Savage, R.P., 1959</td><td>United States</td><td>Movable bed, model study</td><td>Low,short groins: trapped 12% of test sand.
High, short groins: trapped 25% of test sand.
High, long groins: trapped 60% of test sand.</td></tr>
<tr><td>Schift, J.B., 1959</td><td>Holland, General</td><td>Theory</td><td>Groins should remain as low as is compatible with their reducing effect on littoral drift.</td></tr>
<tr><td>Lee, C.E., 1961</td><td>Great Lakes</td><td>Field Study</td><td>Horizontal Shore Section: minimum height should be berm height of existing beach.
Intermediate Section: not steeper than existing beach profile; should approximate anticipated beach slope.
Seaward Section: governed by expected still water elevation at time of construction.
(specific criteria given for rubble-mound structures).</td></tr>
<tr><td>Kemp, P.H., 1962</td><td>England</td><td>Movable bed, model study</td><td><table>
<tr><th rowspan="2">Groin Type</th><th colspan="3">Orientation with Shore</th></tr>
<tr><th>30° Updrift</th><th>0°</th><th>20° Downdrift</th></tr>
<tr><td>low</td><td>51</td><td>60</td><td>87</td></tr>
<tr><td>high, long</td><td>43</td><td>50</td><td>55</td></tr>
<tr><td>high, short</td><td>67</td><td>71</td><td>67</td></tr>
</table>Quanties of material collected in downdrift traps as% of total material after 3 wave cycles, for imperme - able groins.</td></tr>
<tr><td>Bruun, Per, and Manohar, Madhav, 1963</td><td>General</td><td>Field, Experience</td><td>Minimum height should equal maximum water level plus height of normal wave uprush.</td></tr>
<tr><td>Shore Protection, Planning and Design, Coastal Engr. Research Ctr., TR4, 1966</td><td>General
General</td><td>Theory,
Experience</td><td>Groins is built in 3 sections, (a.) horizontal shore section (b.) intermediate sloped section, (c.) outer section.
Height dependent upon construction methods used, economics and beach profiles, wave uprush, and littoral drift (see Table 1).</td></tr>
</table>

Table 3. Spacing of Groins in a System (Balsillie & Berg 1972)

Investigator, Yr	Site	Type	Spacing/remarks
Owen, J.S., and Case, G.O., 1915	England, General	Theory, Experience	1:1 - Recommended spacing.
Case, G.O., 1915	England, New York, New Jersey	Field, Experience	No greater spacing than distance from high to low water line.
Kressner, B., 1928	Germany	Movable bed, model study	1:2 - 1:3 - Tests show this to be optimum spacing requirement; smaller spacing unnecessary, wider unadvisable.
Coen-Cagli, M.E., 1932	General	Theory, Experience	1:1.5 to 1:2 - Initial spacing for systems on pebble beaches, then decrease until desired effect is produced. 1:1.5 - Usually spacing of groins on a sandy beach.
Steiner, C.T., 1936	Rockway Bch, New York	Field	1:1 - Will maintain a beach about 1/2 this length.
Brown, E.I., 1939; 1940	General	Experience Wave Tank Studies	1:1 – 1:3 – 1:1 and less is never economical. 1:3 is maximum limit for spacing. Suggests that after length is decided, draw a line through the end of the groin parallel with direction of the storm approach. Projection of this line on line of connecting landward edge of groins will determine proper spacing.
Dobble, C.H., 1946	England	Experience	1:1 to 1:1.5 - Considered as best spacing.
Frech, P.P., 1948	General		1:1.5 - Generally accepted spacing ratio.
Brater, E.P., 1955	Great Lakes	Field, General	1:1 - Where wave action is severed and beach material is time 1:2 - Where wave action is less severe, and beach material is coarse sand or gravel.
Nagai, Shositiro, 1956	Japan	Movable bed, model study	1:3 - Provided optimum spacing for conditions during testing
'Basic Coastal Model' Hydraulic Research, London, 1957	England	Movable bed, model study	1:1 - High, impermeable arrested 7/8 of drift, but upper beach eroded. 1:2 - Low, impermeable arrested 1/2 of littoral drift, but upper did not erode.

Table 3 (continued). Spacing of Groins in a System (Balsillie & Berg 1972)

Investigator, Yr	Site	Type	Spacing/remarks
Hiranandini, M.G., and Gole, C.V, 1961	Cochin, India	Field study prototype conditions model study	1:3 - Should not exceed this ratio. 1:2 - Recommended spacing at Cochin.
Lee, C.E., 1961	Great Lakes	Field study	Governed by: 1.) Angle beach normally makes with shoreline 2.) Minimum width of beach required on the down-drift side of groins.
Bruun, Per, and Manohar, Madhav, 1963, Bruun, Per, 1955	General	Field, Experience	1:1.5 – 1:4 - Generally the range of ratios used. As site of material and amount of littoral drift increases, so should spacing. As steepness of beach profile and steepness of waves increases, spacing distance should decrease. 1:1 – 1:1.5 - If ratio is less than this groins in most cases will not work well.
Ishihara, Tojiro, and Sawaragi, Toru, 1964	Japan	Field	1:1.5–1:2 - Utilized this spacing in actual situation
Wiegel, R.L., 1964		Lab. Tests	1:2 – 1:3 - Most desirable distance between groins; the greater the relative groin length, the smaller should be the distance between groins (after Horikawa and Sonu, 1958). 1:4 - Desirable for conditions of waves of variable direction for groins normal to shoreline (after Hoyle and King. 1955).
Shore Protection Planning and Design, Coastal Engr. Research ctr., TR4 1966	General	Manual	1:2 or 1:3 - Suggested as rule of thumb method. Distance from berm crest to seaward end. Other considerations offered in text. A groin system too closely spaced diverts, material off-shore rather than create a wide beach.

Table 3 (continued). Spacing of Groins in a System (Balsillie & Berg 1972)

Investigator, Yr	Site	Type	Spacing/remarks
Barcelo, J.P., 1968	Portugal	Model study	1:2.5 – for 20° Where is the angle of wave incidence with the shoreline 1:3:5 - for - 10° 1:4 - for - 5°
Price, W.A., and Tomlinson, K.W., 1968	England	Movable bed, model study	1:1.5 - 1:2 - These ratios caused considerable deposition during tests.
Kolp, Otto, 1971	Fischland, Zempin, Nevendorf	Field, perimental	1:1 - A change in this ratio in terms of greater groin length will cause a reduction in rips at groin flanks.

Table 4. Permeability and adjustability of Groins (Balsillie & Berg 1972)

Investigator, Yr	Site	Type	Adjust.	Remarks
Case, G.O., 1915	England, New York, New Yersey	Field, Experience	Adjustable	Proposes use of adjustable groins so that they can be built up as accretion progresses.
Brown, E.I., 1939 1940	General	Experience	Impermeable	Groins should be sand-tight
Evans, D.P., 1943	General United States	Theory	Permeable	Appear to be sucessful where there are strong prevailing wind and shore currents, or where there is variable beach drifting and weak, variable currents.
Johnson, J.W., 1948	United States	Movable bed, model study	Impermeable	Tests showed that impermeable groins were more effective in causing accretion (single structures used in tests).
Shay, E.A., and	United States	Movable bed,	Permeable	Should

Table 4 (continued). Permeability and adjustability of Groins (Balsillie & Berg 1972)

Investigator, Yr	Site	Type	Adjust.	Remarks
Johnson, J.W., 1951		model study		be used in systems only - caused deposition of from 11 to 26% sand under 2 corresponding sets of wave conditions
			Impermeable	Much more effective than permeable groins for trapping littoral drift.
Brater, B.P.,1953	Great Lakes	Field, General	Impermeable	Permeable groins work only where wave action is mild or beach material is coarse.
Mason, M.A., 1953	Great Lakes	Field	Impermeable	Permeable structures appear to be poorly suited to Great Lakes area. The requirement of impermeability is absolute
'Basic Coastal Model', Hydr. Research, London, 1957	England	Movable bed, model study	Permeable	Had only a small influence on littoral drift and caused some loss of sand from upper beach. Ratio of voids to solids 1:1.
Bruun, Per, Gerritsen, F., and Morgan, W.H., 1957	Florida	Field, Experience	Permeable	Permeable groins are usually accompanied by lee side scour.
			Impermeable	Recommend use of low, impermeable, nonadjustable or impermeable adjustable groins on Florida shorelines.
Wicker, C.F., 1958	General	General theory	Impermeable	Recommendation for all situations.

Table 4 (continued). Permeability and adjustability of Groins (Balsillie & Berg 1972)

Investigator, Yr	Site	Type	Adjust.	Remarks
Lee, C.E., 1961	Great Lakes	Field study	Permeable	13% of 841 groins examined, these reported for the Great Lakes. Impermeable 87 %
Hoyle, J.W., and King, G.T., 1962	England	General	Impermeable	Do not recommend use of permeable groins.
Bruun, Per, and Monohar, Madhav, 1963	General	Field, Experience	Adjustable	Recommend use of adjustable groins to regulate amount of drift supplied to down-drift beaches.
Shore Protection, Planning, and Design, Coastal Engr. Research Ctr., TR4, 1966	General	Manual	Adjustable	Useful whore attempt is being made to widen beach with a minimum of damage to down-drift area.
			Permeable	Present state of knowledge does not lend conclusions as to effectiveness.
Price, W.A., and Tomlinson, K.W., 1963	England	Movable bed, model study	Impermeable	Permeable groins had little effect on long-shore drift
Kolp, Otto, 1971	Fischland, Zempin, Neuendorf	Field, Experimental	Permeable	Found that 37% open space reduced longshore flow 50%; piling was used in groin construction.

Table 5. Orientation of Groins with the Shoreline (Balsillie & Berg 1972).

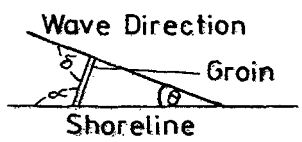
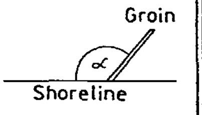
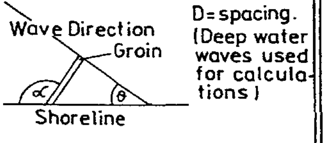
<table>
<tr><th>Investigator, Yr</th><th>Site</th><th>Type</th><th>Orient./remarks</th></tr>
<tr><td>Case, G.O., 1915</td><td>England, New York, New Jersey</td><td>Field, Experience</td><td>Recommends use of normal groins only, Oblique groins are more costly, are liable to be damaged by waves because of greater surface area exposed to waves, and cause more scour than groins constructed normal to the coast.</td></tr>
<tr><td>Duvivier, Jack, 1947; 1949</td><td>England</td><td>Experience</td><td>Normal to shboreline:
use where direction of drift is variable.
10° from normal, pointing away from drift: use where there is prevailing drift, and where beach material is shingle.
20° from normal, pointing away from drift: use where prevailing drift occurs, and where beach is sandy.</td></tr>
<tr><td>Nagai, Shositiro, 1956</td><td>Japan</td><td>Movable bed, model study</td><td><table><tr><th>θ</th><th>δ</th><th>α</th></tr><tr><td>30°</td><td>100°</td><td>110°</td></tr><tr><td>40°</td><td>110°</td><td>100°</td></tr></table>
These angles satisfied test conditions; in general should not be less than 95° nor greater than 120°, otherwise scour is the result.</td></tr>
<tr><td>Shimano, T., Hom-ma, M., Horikawa, K.,</td><td>Japan</td><td>Movable bed, model study</td><td>Should consider building groins upcoast (toward updrift direction).</td></tr>
</table>

Table 5 (continued). Orientation of Groins with the Shoreline (Balsillie & Berg 1972).

<table>
<tr><th>Investigator, Yr</th><th>Site</th><th>Type</th><th>Orient./remarks</th></tr>
<tr><td>Horikawa, Kiyoshim, and Sonu, C., 1958</td><td>Japan</td><td>Movable bed, model study</td><td>Best orientation
depends upon wave steepness.
For steep storm waves, 105°
For relatively flat waves. 90°
(Taken from Wiegel, 1964)

</td></tr>
<tr><td>Nagai, Shoshichiro, and Kubo, Hirokazo, 1958</td><td>Japan</td><td>Fixed basin, model study</td><td><table><tr><th>θ</th><th>α</th><th>D</th></tr><tr><td>30°</td><td>110°</td><td>1:3</td></tr><tr><td>45°</td><td>90, 110°</td><td>1:3</td></tr><tr><td>60, 90°</td><td>90°</td><td>1:3 - 1:4</td></tr></table>

Below are results of fixed basin test conducted to compare with movable bed study by Nagai, 1956.</td></tr>
<tr><td>Kiranandini, M.G., and Gole, C.V., 1961</td><td>Cochin, India</td><td>Field study - prototype conditions model study</td><td>Inclined
groins did not commend themselves for adoption on this coast.</td></tr>
<tr><td>Lee, C.B., 1961</td><td>Great Lakes</td><td>Field study</td><td>Inclined groins are ineffective or too costly for the effects produced.</td></tr>
<tr><td>Kemp, P.H., 1962</td><td>England</td><td>Movable bed, model study</td><td><table><tr><th>Orientation</th><th>Low Groin</th><th>High,Long</th><th>High,Short</th></tr><tr><td>30°updrift</td><td>51</td><td>43</td><td>67</td></tr><tr><td>Normal to Coast</td><td>60</td><td>50</td><td>71</td></tr><tr><td>20° downdrift</td><td>87</td><td>55</td><td>67</td></tr></table>30° updrift
51 - 43 - 67
Normal to Coast
60 - 50 - 71
20° downdrift
87 - 55 - 67
Table gives quanitites of material collected in down-drift traps as of total material after 3 wave cycles, for impermeable groins.
Gives results showing that by appropriate choice of groin type and aligament, it should be possible to orient the shoreline in such a way that the effects of storm attack are minimized.</td></tr>
</table>

Table 5 (continued). Orientation of Groins with the Shoreline (Balsillie & Berg 1972).

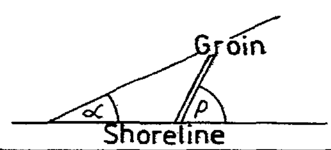
Investigator, Yr	Site	Type	Orient./remarks
Shore Protection, Manning and Design, Coastal Engr. Research Ctr., TR4, 1966	General	Manual	Maximum economy is achieved with a straight groin normal to the shoreline. In cases where shoreline alignment may change after groin construction it may be desirable to build groins at an angle initially, so that they will be normal to expected adjustment of shore.
Barcelo, J.P., 1970	Portugal	Model study	P=90° α=20° = angle of groin with shoreline. p=70° for α = 10° = obliquity of waves. Where variable wave conditions persist, groins constructed normal to the shoreline are preferred. 

Table 6. Groyne length (Balsillie & Berg 1972).

Investigator, Yr	Site	Groyne length
Anon 1960	Prototype, shingle E.coast England	400ft long with secondary groynes, in between, of length 250ft
Bakker and Joustra 1970	Prototype, Netherlands, sand	200m long
Balsillie and Berg 1972	General recommendations from literature survey	Lengths vary; most prefer long groynes
Barcelo 1968	Physical model	Long- i.e. beyond depth of material movement
Bruun 1952	Prototype, sand on clay, Denmark	Four lengths of groyne are described: 220m, 170ft, 700-1300 ft and 500-700ft. All have resulted in reduced erosion rates.
Bruun 1972	General prototype experience	Dutch groynes generally about 200 m length
Bruun and Manohar 1963	Prototype, general experience	USA - generally to 6ft below mean water level. N. Sea - generally to 12-18 ft below mean water level
Castanho et al 1974	Prototype, sand	Angola - length 100m (5m below low water), groynes satisfactory. Mozambique - length 50m (about mean water), groynes not long enough
Shore Protection Manual 1966	General	Length depends on ultimate beach profile and % interruption of drift
Curren and Chatham 1977	Physical model (for a specific project), sand beach represented	i) Length - 270 ft (to - 6ft contour) failed bacause of rip currents ii) Length - 520 ft was satisfactory if groynes were rubble mound but not if they were sheet pile
Duvivier 1953	Prototype, E coast, England	Scheme 1 - length 300 ft satisfactory expanse of sand maintained. Schme 2 - length 250-400 ft satisfactory in holding both sand and shingle. Scheme 3 - lengths up to 200 ft. Good results with both sand and shingle
Duvivier 1961	Prototype, S coast, England	Alternate groynes continued to MLWS
Everts 1979	Prototype, sand, New Jersey	Lengths from 100-300m, working satisfactorily
Fryde 1968	Prototype, E coast England	Groynes up to 125 m long were unsatisfactory because the lack of drift led to erosion. Groynes were shortened and beach replenished

Table 6 (continued). Groyne length (Balsillie & Berg 1972).

Investigator, Yr	Site	Groyne length
Hall 1963	General recommendations (USA)	Groynes usually extend to 6ft depth contour
Hiranandini and Gale 1960	Prototype, sand, India	Length 200ft (50ft beyond low water mark) - working satisfactorily
Horton 1950	General recommendations (USA)	Groynes usually extend to 6ft depth contour
Hoyle and King 1962	general recommendations (UK)	groynes should extend to mean low water
Hydraulics Research Station 1958 1959	Physical model	Length 180ft (beyond low water)
Hydraulics Research Station 1962	Prototype, shingle, SE England	Length 130ft. Satisfactory results
Ishihara and Sawaragi 1964	Prototype, sand, Japan	Length should be at least 60m
Lee 1953	Prototype, Great Lakes, USA	Length 250-300ft. Satisfactory results
Machemehl 1977	Prototype, N. Carolina, USA	Lengths varied from 150-325ft
Manohar 1967	General recommendations	Definite rules are not possible
Nagai 1956	Physical model	40% of distance to the line of plunging breakers
Nagai and Kubo 1958	Physical model	About 0.5 of the distance to breaker line for 1/20 slope
Omholt 1974	Prototype, New York, USA	50% of the groynes surveyed were less than 100ft; 90% less than 200ft

Table 6 (continued). Groyne length (Balsillie & Berg 1972).

Investigator, Yr	Site	Groyne length
Petersen 1963	General recommendations Germany	Up to 300 m long groynes are used
Pachois and Vollmer 1961	General recommendations	Length fixed by offshore depth conditions but usually extends to 6ft depth contour
Price and Tomlinson 1968	Physical model	Groynes extended to low water
Price and Tomlinson 1970	Physical model	Groynes length was 1.35 $\times$ distance between high and low water
Price, Tomlison and Willis 1972	Prototype, sand, S coast England	Length 90 m (below MLWS) - performed satisfactorily
Sellner 1962	Prototype, general recommendations	Lengths vary from less than 100 ft to several hundred feet
Shay and Johnson 1951	Physical model	Groyne extended to 2 x deep water wave height below still water. Total length was 33 1/3 xH.
Thorn and Simmons 1971	General recommendations	Shingle - groyne should extend just beyond theshingle toe. Sand - length depends on required levelsand the littoral drift
Vesper 1961	Prototype, sand, Connecticut USA	Average length 310ft (mean lowwater) - groynes performing satisfactorily
Wells 1952	Prototype, sand/shingle, Lake Erie	Length about 100ft
Zhdanov 1964	Model/Prototype, shingle, Russia	$L = b_{min} + S \tan \gamma + a$ where b_{min} = minimum proposed beach width above mean water level S = groyne spacing gamma = angle of wave attack for worst case a = minimum distance between shoreline and groyne root to ensure beach stability

Table 7. Groyne Height (Tomlison 1980).

Investigator, Yr	Site	Groyne Height used
Balsillie and Berg 1972	General recommendations from literature survey	Heights vary: adjustable heigth is preferred
Barcelo 1968	Physical model	High groynes were tested
Barcelo 1968	Physical model	High groynes were tested
Bruun and Manohar 1963	Prototype, general experience	Horizontal shore sectionat height of maximum high water and maximum wave run-up. Sloping central section paralled to expected beach. Horizontal outer section of littoral drift
Castanho et al 1974	Prototype	High, 2m
Shore Protection Manual 1966	General	Horizontal shore section equal to desired berm height. Sloping intermediate section depending on elevation of outer end. Outer section, horizontal, height determined by construction and safety considerations
Current and Chatham 1977	Physical model (for a specific project)	i) up to 12ft high ii) sheet piles 2ft high, rubble-mound up to 19 ft high
Everts 1979	Prototype, New Jersey	Shoreward end 1.5m above MHWS to 2.4m below MHWS
Hall 1963	General recommendations (USA)	Height depends on the required function of the groyne
Hiranandini and Gale 1960	Prototype, India	High enough to prevent material passing over the top
Hoyle and King 1962	General recommendations (UK)	High
Hydraulics Research Station 1957 1959	Physical model	Groynes 1.5ft high better at maintaining beach levels than those 3ft high

Table 7 (continued). Groyne Height (Tomlison 1980).

Investigator, Yr	Site	Groyne Height used
Ishihara and Sawaragi 1964	Prototype, Japan	Offshore section, paralled to shore, should be below mean sea level
Machemehl 1977	Prototype, N Carolina USA	Heights varied from 3.4 to 6.8ft
Manohar 1967	General recommendations	Definite rules are not possible. Adjustable height is recommended
Minikin 1954	General recommendations	Height should be very low
Muir Wood 1969	General recommendations	On sand beaches the height should belimited to 0.5-1.0m
Nagai 1956	Physical model	Groyne height of the order of wave height
Nagai and Kubo 1958	Physical model	Groyne height of the order of wave height
Peterson 1963	General recommendations Germany	Varies from very high to cater for proposed beach, to low above existing beach levels
Price and Tomlinson 1968	Physical model	Low (less than 0.5 x wave height)
Price and Tomlinson 1970	Physical model	Low (about 0.5x wave height)
Russell R C H 1960	General recommendations	Sand - height should be just above the beach. Shingle - height not so critical, depends on desired degree of interuption of drift
Sellner 1962	Prototype, general recommendations	Horizontal at top of beach, sloping parallel to shore down to low water
Shay and Johnson 1951	Physical model	Height of 4/3 x deep water wave height above still water level
Vesper 1961	Prototype, Connecticut, USA	Height approx 1.5ft at top of berm and approx 4ft at seaward end
Zhdanov 1964	Model/Prototype, Russia	Height about 0.5m parallel to beach (above mean sea level) and 0.3m (below mean sea level)

Table 8. Groyne Spacing (Tomlison 1980).

Investigator, Yr	Site	Groyne Spacing used
Anon 1960	Prototype, E coast England	Main groynes 1000ft apart (S/L - 2.5)
Bakker and Joustra 1970	Prototype, Netherlands	Spacing about 200m (SL - 1)
Balsillie and Berg 1972	General recommendations from literature survey	Spacing/length ratio is generally between 1 and 4
Barcelo 1968	Physical model	Angle of wave attack S/L 20^o – 2.5 10^o – 3.5 5^o– 4.0
Bruun 1952	Prototype, sand on clay, Denmark	Four spacings of groynes were used: 377m (S/L = 1.7), 270ft (S/L = 1.8 to 1.0), 125ft (S/L = 2.5 to 1.8)
Bruun 1972	Prototype, general	S/L from 1 to 4 recommended
Bruun and Manohar 1963	Prototype, general experience	S/L varies from 1.5 to 4
Castanho et al 1974	Prototype, sand	Angola - spacing 300m (S/L = 3) Mozambique - spacing 200m (S/L - 2.9)
Shore Protection Manual 1966	General	Spacing is a function of the groyne length and expected shoreline alignment. As a general rule S/L is between 2 and 3
Curren and Chatham 1977	Physical model (for a specific project)	i) Varies from 790-1310ft (S/L from 2.9 to 4.9) ii) Varies from 620-11400 ft (S/L from 1.9 to 1.2)
Duvivier 1952	Prototype, E coast England	Scheme 1:300ft (S/L = 1) Scheme 2:250-900ft (S/L less than 1 up to 3) Scheme 3:200-400ft (S/L less than 1 to over 2)
Everts 1979	Prototype, New Jersey	i) 100m (S/L - 1) ii) 100m and 300m (S/L = 1 to 2)
Fryde 1968	Prototype, E coast England	Spacing of 100m and 305m ((S/L from 1.3 to 2.4)
Gaughan and Komar 1976	Physical model	S/L of 5 to 6.7 recommended to eliminate edge waves
Hiranandini and Gale 1960	Prototypr, sand, India	Spacing 660ft (S/L = 3.3)

Table 8 (continued). Groyne Spacing (Tomlison 1980).

Investigator, Yr	Site	Groyne Spacing used
Horton 1950	General recommendations USA	S/L from 1 to 3 recommended
Hydraulics Research Station 1957 1959	Physical model	S/L of 1 and 2 used
Hydraulics Research Station 1962	Prototype, SE England	Spacing approx 160ft (S/L - 1.2)
Ishihara and Sawaragi 1964	Prototype, Japan	S/L from 1.5 to 2.0
Lee 1953	Prototype, Great Lakes USA	Spacings from 350-390ft (S/L = 1.3 to 1.4)
Manohar 1967	General recommendations	S/L from 1.5 to 4. Lower values are permissible on shingle shore, higher values if littoral drift is very high
Minikin 1954	General recommendations	For shore groynes S/L should be 1. For long groynes S/L should be 1.5
Nagai 1956	Physical model	Angle of wave attack - Groyne orientation 35–55°-< 30°-110°-90°
Petersen 1963	General recommendations Germany	S/L varies from 1 to 3
Pretious and Vollmr 1961	General recommendations	S/L from 1 to 3 recommended
Price and Tomlinson 1968	Physical model	S/L of 1.5 or 2 found to be most effective at reducing drift (S/L = 1 also tested)
Price and Tomlinson 1972	Physical model	Spacing of 1.5 x the horizontal distance from high to low water (S/L = 1.1)
Price, Tomlinson and Willis 1972	Prototype, S coast of England	Spacing of 190m (S/L = 2.1)
Sellner 1962	Prototype, general recommendations	Generally S/L should be from 1 to 3
Shay and Johnson 1951	Physical model	Spacing = deepwater wave length
Vasper 1961	Prototype, Connecticut, USA	S/L - 1.75
Wells 1952	Prototype, Lake Erie	S/L = 1
Zhdanov 1964	Model/Prototype, Russia	S/L varies from 1.3 to 1.5

Table 9. Groyne Permeability and Construction (Tomlison 1980).

Investigator, Yr	Site	Perm./construction
Anon 1960	Prototype, E Coast England	i) Concrete block and II) reinf. concrete piles and planks
Barcelo 1968	Physical model	Groynes impermeable (simulating rock fill prototype)
Bruun 1952	Prototype, Denmark	Two different constructions described: timber centre with rubble sides; concrete blocks
Cestanho et al 1974	Prototype	Angola - rubble mound Mozambique - impermeable
Duvivier 1953	Prototype, E Coast England	Scheme 1 - timber Scheme 2 - mass concrete Scheme 3 - timber
Everts 1979	Prototype, New Jersey	Stone/timbre impermeable or stone impermeable
Fryde 1968	Prototype, E Coast England	Groynes were of mass concrete piles and timber planks
Hall 1963	General recommendations	Permeable groynes not recommended
Hiranandini and Gale 1960	Prototype, India	Rubble mound
Horton 1950	General recommendations (USA)	Permeable groynes not recommended

Table 9 (continued). Groyne Permeability and Construction (Tomlison 1980).

Investigator, Yr	Site	Perm./construction
Hoyle and King 1962	General recommendations (UK)	Impermeable groynes
Hydraulies Research Station 1957 1958 1959	Physical model	Timber permeable groynes were not as satisfactory as impermeable groynes
Hydraulics Research Station 1962	Prototype, SE England	Timber impermeable
Ishihara and Sawaragi 1964	Prototype, Japan	Permeable concrete blocks
Lee 1953	Prototype, Great Lakes, USA	Steel sheet piles
Machemehl 1977	Prototype, N. Carolina USA	Sand filled nylon bags - impermeable
Price and Tomlinson 1968	Physical model	Permeable groynes had little effect
Price and Tomlinson 1970	Physical model	Impermeable groynes
Price, Tomlinson and Willis 11972	Prototype S England	Permeable - Makepeace Wood type
Shay and Johnson 1951	Physical model	Permeable had slight effect if used in a system at close spacing. Impermeable tested also
Vesper 1961	Prototype, Connecticut, USA	Stone
Wells 1952	Prototype, Great Lakes, USA	Many different types of construction eg. steel sheet piles, timber, stone
Zhdanov 1964	Model/Prototype, Russia	Impermeable

Table 10. Groyne Inclination (Tomlison 1980).

Investigator, Yr	Site	Groyne inclination
Barcelo 1968	Physical model	NB All groynes were at 90° to the coast except those listed in this table. For incident waves at low obliquities (5°, 10°) groynes normal to the coastline were best. For incident waves at 20° groynes inclined at 110° performed better
Duvivier 1953	Prototype E Coast England	Scheme 1 - groynes angled 20° downdrift
Hydraulics Research Station 1957	Physical model	Angles of up to 18° away from the normal proved to be little different except that those orientated up drift were scoured at the roots
Minikin 1954	General recommendations	Can be inclined = 10° from the normal. It is usual for groynes to be inclined downdrift it at all
Nagai 1956	Physical model	For angles of wave attack of 35-55° tha groyne orientation should be 110° otherwise groynes should be normal to the coast
Nagai and Kubo 1958	Physical model	Groyne orientation of 110° found to be satisfactory as well as 90°

Table 11. Groyne Shape (Tomlison 1980).

Investigator, Yr	Site	Groyne Shape
Barcelo 1968	Physical model	NB Groyne shape in the references is assumed to be straight. T groynes were not more effecient than straight groynes but they did prevent erosion at the roots
Bruun 1972	Prototype, general	T or L groynes may be advantageous
Shore Protection Manual 1966	General	Straight groynes are most economical
Ishihara and Sawaragi 1964	Prototype, Japan	T groynes recommended
Manohar 1967	General recommendations	T groynes are useful where there is severe wave action

Appendix 2-2. Bruun's Description of the Performance of Coastal Structures

Given below are excerpts from Bruun & Nayak (1980) representing Bruun's perception of coastal protection and the advantageous and adverse features of different defence measures. Perusal of the tables may turn out useful upon comparison with the contents of the main body of our document.

Table 3.5 Coastal protection in relation to source of materials and conditions of beach profiles for beneficial versus adverse effects.

Coastal situation : actual conditions of source of material and beach profiles	Seawalls and revetments
A. Plenty of source material: Overnourished profiles.	Might be necessary to avoid attack under extreme high-water and storm conditions. Should be built in such a way that they contribute to as little erosion of beach as possible. Vertical walls built too close to the shoreline increase erosion. Energy absorbing walls, revetments or rubble mounds are usually preferable.
B. Source material available: Sufficiently nourished profiles. Balance between material eroded and deposited.	Same as above.
C. Only limited source.	
C(a). Erosion only up to limited depth in the sea.	C(a1). Source of material : Can stop erosion provided they are stable at the depth at which erosion takes place. Even if groins are built, a seawall might also be necessary to avoid attack at toe of dune or cliff under extreme high water and storm conditions. It is very important that they be built in such a way that they contribute to erosion as little as possible. Sloping walls, revetments and rubble mounds work better than the other types in avoiding beach erosion due to low reflection of wave energy.
	C(a2). No source of material. Same as above.
C(b). Erosion up to deep water in the sea.	C(b1). Source of material : Unless material is supplied artificially to balance erosion, the groins built will work only for a limited period before collapsing. It is very important that they contribute to erosion as little as possible. Sloping walls, revetments and rubble mounds contribute by themselves to little beach erosion as compared to other types because of low reflection.
	C(b2). No source of material : Unless material for total balance of erosion is supplied, groins built, will work only for a limited time before collapsing. It is very important that they are built in such a way that they contribute to erosion as little as possible. This would mean low reflection. Sloping walls, revetments and rubble mounds by themselves contribute to little beach erosion as compared to the other types.

Table 3·6 Details of the performance of seawalls (ref. 3).

			Comments :
1.	What is wanted :	Storm tide and/or extreme protection of shore and beach. Protection of specific valuable areas (industry, buildings, highways etc).	Energy–adsorbing wall or revetment on dyke or dune. Any type of substantial wall with as little adverse effects as possible.
2.	Layout and geometry:	As streamlined as possible. It is best to leave and maintain a beach in front of the wall.	Erosion may be stopped at the wall but artificial nourishment may be needed to maintain beach in front of the wall.
		Influence on adjoining shores.	Leeside erosion may result if erosion continues leaving wall as protruding headland or if wall is built too far seaward and is not streamlined in horizontal geometry. Transfer of sand or other nourishment of downdrift shore may be needed.
3.	Combination with other coastal protective measures:	Groins.	To break longshore current and possibly build up beach in front of wall.
		Artificial nourishment	To maintain beach in front of wall and/or to check downdrift erosion.
4.	Design:	Energy – absorbing (sloping and / or mound type).	Considerate to beach stability due to friction and low reflection.
		Non-energy-absorbing (vertical sheet pile or slab).	May create local erosion due to less friction against currents and more reflection.

Table 3·7 Details of the performance of groins (ref. 3).

			Comments :
1.	Degree of efficiency wanted:	Just beach stabilization.	Short groins mainly covering the beach.
		Also widening of beach.	Longer groins, possibly extending beyond bar or breaker zone.
2.	Layout and geometry:	Streamlined in horizontal geometry. No sharp turns or corners.	Reaction of shore protected: Stable or widening and then stable.
			Influence on adjoining shores: Usually beneficial or neutral updrift but adverse downdrift.
3.	Combinations with other coastal protective measures:	Seawalls.	To cope with extreme conditions including storm surges.
		Artificial nourishment.	To fill groins and widen beach initially and maintain width. To eliminate adverse effects on downdrift beaches.
4.	Design: Length in agreement with point 1. Height to match beach profile wanted to the practical extent possible.	Impermeable: Energy absorbing.	Less reflection, less loss of sand.
		Non energy absorbing.	More reflection, more loss of sand.
		Adjustable elevation.	May be operated to match fluctuations of beach
		Fixed elevation.	Can not be operated to match fluctuations of beach.
	Length / spacing ratio from 1 : 1 to 1 : 4 depending upon quantity of drift and beach material. Most common ratio is 1 : 2.	Permeable : May be adjustable or fixed.	"To blow and have flour in your mouth at the same time." May provide beneficial results where currents are the main agents in transport of materials, that means in rivers and estuaries.

Table 3·8 Details of the performance of offshore breakwaters (ref. 3).

			Comments :
1.	What is wanted:	Protection or protection and beach.	If breakwater is built on littoral drift shore both are usually obtained.
2.	Layout and geometry:	Parallel to shore or largely following depth contours.	Tombolo formation will result on shore to be protected. Severe downdrift erosion may result due to littoral barrier effect.
3.	Combination with other coastal protective measures:	Groins.	This combination is unlikely unless groins are used to check downdrift erosion, thereby transferring problem further downdrift.
		Seawalls.	May be built to protect against extreme storms and tides to check downdrift erosion.
		Artificial nourishment.	May be used to create beach more rapidly if natural supply of material is limited or to check downdrift erosion.
4.	Design:	Energy absorbing structures preferable. See Table 3·6. Combination with natural reefs often advantageous.	

Table 3·9 Details of the performance of artificial nourishment (ref. 3).

1.	What is wanted:	Protection and beach.
2.	Layout and geometry :	Follow natural shoreline closely on straight or streamlined shores. Fill in pockets on headland shores and artificial pockets.
3.	Combinations with other coastal protective measures:	Groins: to create or maintain beach and to eliminate leeside erosion. Seawalls: to protect wall and/or create or maintain beach in front of wall and to eliminate leeside erosion.
		Offshore breakwaters: to create and maintain protective beach.
4.	Design :	Nourishment from land or offshore sources. Offshore equipment under development. Various methods tested in actual operation. Sand shall be suitable for nourishment. Main requirement is that sand should be as coarse or coarser than the natural beach material and of no less specific gravity. Bypassing arrangements by fixed or movable plants including weirs and floating plants. Movable arrangements preferable.

Seawalls can be classified into three groups, namely, sloping walls of revetment type, vertical gravity walls and piled structures or simple mound structures to control emergency situations. Figs. 3·7 to 3·14 show the cross sections of the various types of seawalls or revetments designed under a variety of conditions.

Rock mounds and revetments: Figs 3·7, 3·8 and 3·9 present the general design parameters as explained below. Hydraulic and wave mechanics design criteria are given in Appendix A and B whereas the soil stability characteristics are presented in Appendix D. With these details it is possible to determine the armour weights and the top elevation of the wall based on the information on waves and tides and predict the foundation stability. Additional information on revetment design is given in refs. 3, 6, 7, 14 and 43.

Gravity walls: Gravity walls, as shown in Fig 3·10, are useful at places where wave action is limited, *e.g.* in estuaries and protected bays. At the same time they may function as berthing places for small vessels. Their main drawback is that they, due to their vertical faces, have a high reflection coefficient. This stirs up the beach and bottom sediments in front of the structure (refs. 6, 14 and 43). A protective apron, therefore, in

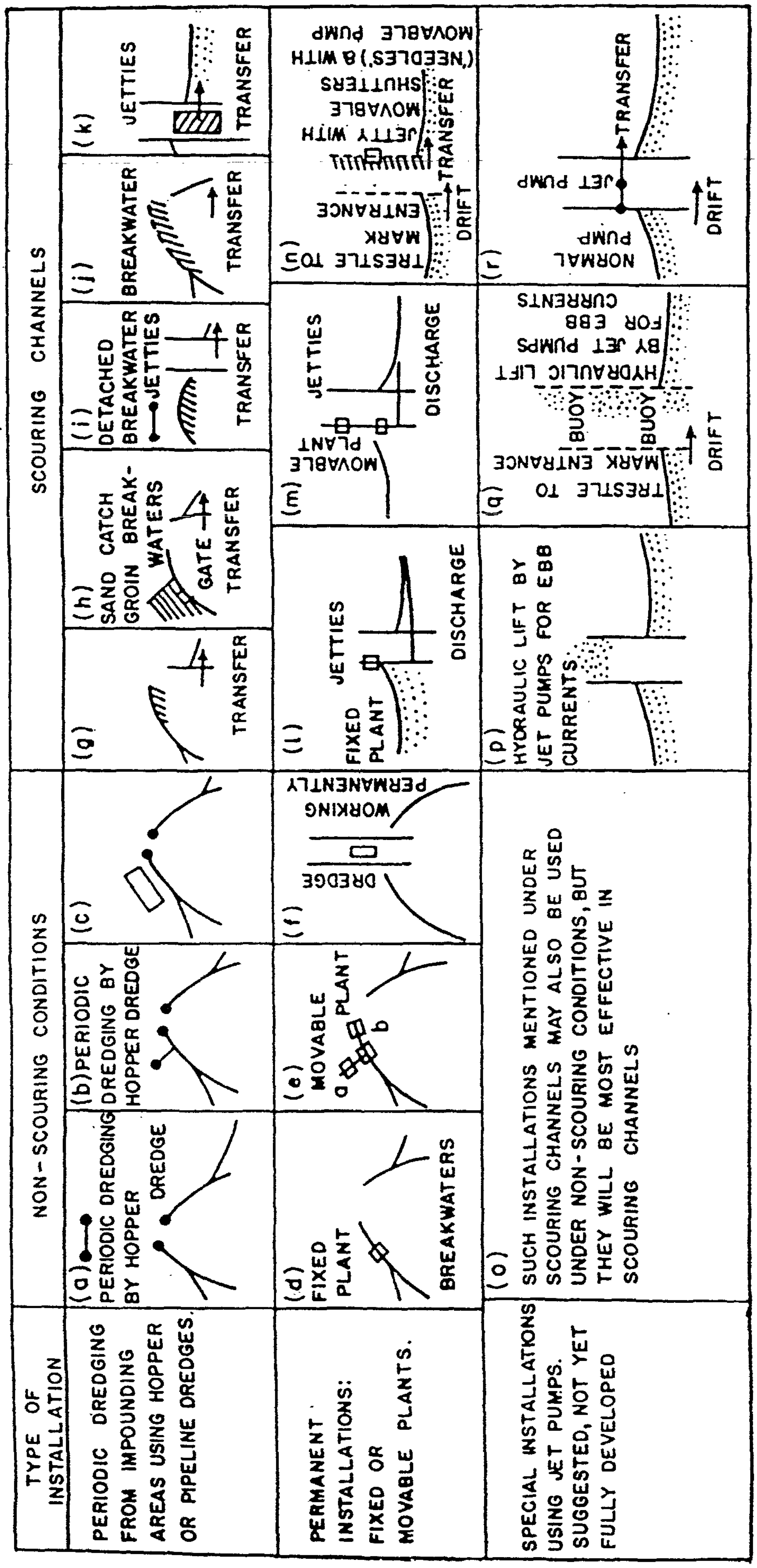


Fig. 3·21 By-passing plants and arrangements (ref. 4).

Appendix 4-1

CONSTRUCTION STANDARDS AND REGULATIONS

LOADS AND ACTIONS OF WAVES, ICE AND SHIPS ON THE HYDROTECHNICAL STRUCTURES

S N i P 2.06.04. - 82*

OFFICIAL ISSUE

(This Standard is a supplemented and updated version of the Standard SNiP 2.06.04 - 82).

STATE COMMITTEE OF THE SOVIET UNION ON THE CONSTRUCTION AFFAIRES

MOSCOW 1986

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(волновые, ледовые и от судов)
СНиП 2.06.04-82*

From the translator:

The paragraphs, figures and tables denoted by (*) are the supplemented material, introduced to the Standards SNiP 2.06.04 - 82.

The figures in the translation are xero-copies from the originals; the text explanations were translated only. Some notations used in Standards are different from these in the Western literature. However, in order to keep the faithfulness to the original, they were not changed.

Standards were translated by prof. dr. S.R. Massel, Institute of Hydro-engineering, Gdansk, Poland.

Gdansk, July 1989.

Contents

APPENDIX 5-1
Dutch Experience on Design of Dikes and Revetments

by

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Coastal Engineering Practice '92

Long Beach, CA, USA, 1992

DUTCH EXPERIENCE ON DESIGN OF DIKES AND REVETMENTS

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Abstract

The increased demand for reliable design methods for protective structures has resulted in The Netherlands in preparing a set of design guidelines for design of dikes and revetments. The Dutch principles of functional and technical design of these structures are discussed. Design methods concerning the shape and the height of dikes, as well as stability criteria for slope protection and for protection against overtopping are included. Two cases of failure of revetments are also briefly discussed.

Introduction

The low-lying countries as the Netherlands are strongly dependent on good (safe) sea defences (sea-dikes and/or dunes). In the past the design of dikes and revetments was mostly based on rather vague experience than on the general valid calculation methods. The increased demand for reliable design methods for protective structures has resulted in increased research in this field and, as a result, in preparing a set of design guidelines for various coastal structures.

In this paper a brief review of general design philosophy, geometrical design of dikes and stability criteria of revetments is given. Lessons from practice will be illustrated by a recent case of failure of a revetment near the entrance to the port of Rotterdam and the failure of test embankments at the IJssel-Lake (previously "Southern Sea") location. For a treatment of these matters in greater depth the reader is referred to the original reports and publications (see references).

Because of a shortage of natural rock materials, the use of alternative materials (i.e. industrial waste products such as minestone, slags and silex) in civil engineering application is still growing in the Netherlands (Pilarczyk, 1987). Most of these materials are used as sublayers or bulk material. However, special attention

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should be paid to their specific chemical and mechanical behaviour and environmental aspects when using these materials. The failure case of Zuidwal revetment is an illustration of these aspects.

Design philosophy and methodology

Design philosophy

Dikes are constructed to protect upland (population and economical values) against erosion or inundation due to storm surges. The main purpose of a dike or seawall is to fix the land and sea boundary, and it is not intended to protect either the beach fronting it or adjoining, unprotected beaches. Dikes are one of various forms of coastal protection which may be used singularly or in combination with other methods. 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) or by high-rate erosion (think also of 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 surviving. 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 minimize the negative effects of the solution chosen (see further Kraus and Pilkey, 1988).

Absolute safety against storm surges is nearly impossible to realize. Therefore, it is much better to speak about the probability of failure of a certain defence system. The ultimate potential threat for the Dutch sea defences is derived from extreme storm surge levels with a very low probability of exceedance (1% per century for sea-dikes and dunes) and equated with the average resistance of the dike (or dune). Under these ultimate load conditions, probability of failure of the dike (seawall) should not exceed 10%.
The probabilistic approach to sea defences as developed in The Netherlands is briefly summarized by Hoekstra and Pilarczyk (1992) and is treated more extensively in TAW-report (1990).

Design methodology

When designing coastal 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 1. The designer should be aware of the possible constructional and maintenance constrains (Pilarczyk, 1990).

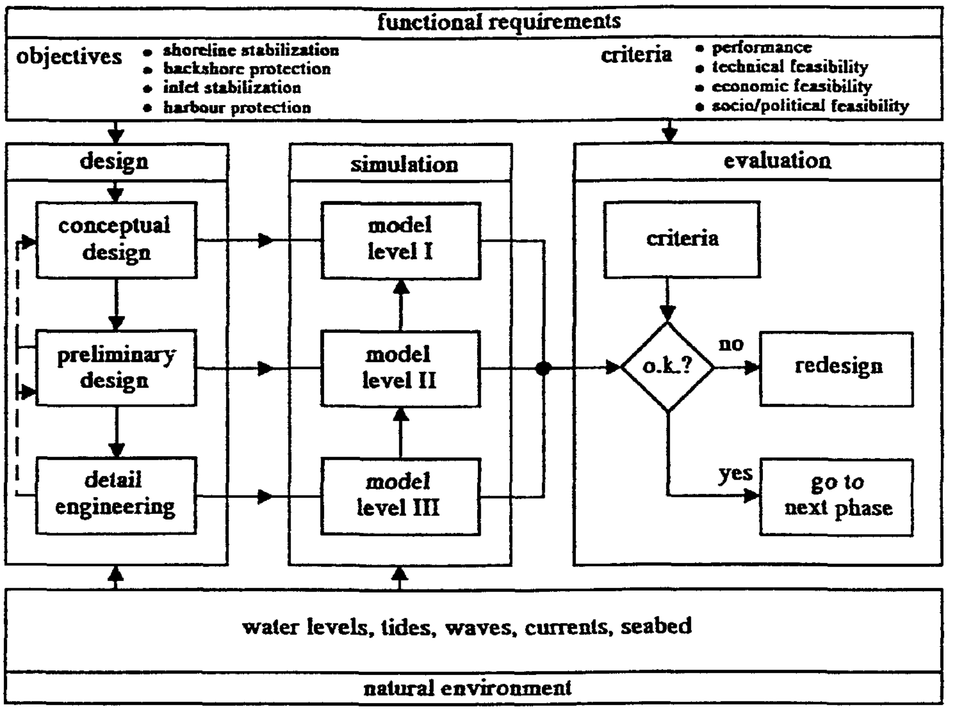


Figure 1 The design methodology

Based on the main functional objectives of the coastal structure a set of technical requirements has to be assessed. When designing a 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 minimized to an acceptable/responsible level,
6. legal restrictions.

Elaboration of these points 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 low-lands 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.

Geometrical design

Selection of the structural concept depends on the function, the local environmental conditions and the constructional constraints. The governing criteria are the technical and economic feasibility for the project under consideration. The function of the dike/seawall is mainly to protect the hinterland against the adverse effect of high-water and waves. If high-water protection is required, the structure should have a height well above the maximum level of wave-uprush during storm surges. This normally calls for high crest elevation (TAW, 1990). If, however, some, overtopping is allowed in view of the character of the hinterland, the design requirement is formulated in terms of the allowable amount of overtopping. Obviously crest-elevation can be reduced considerably in this case.

The shape of the cross-sectional profile of a dike/seawall is an influence on the distribution of wave forces, and thus, also influences the choice of material (type of protective units and their dimensions) suitable for slope protection (revetment), and the height of structure. The gradient of the slope must not be so steep that the whole slope of the revetment can lose stability (through sliding). This criterion gives, therefore, the maximum slope angle. More gentle (flatter) slopes lead to a reduced wave-force on the revetment and less wave run-up; wave energy is dissipated over a greater length. A similar effect can be obtained by applying a berm (trapezoidal profile). By using the wave run-up approach for calculation of the crest height of a trapezoidal profile of a dike/seawall for different slope gradients, the minimum volume of the embankment can be obtained (see Figure 3). However, this does not necessarily imply that minimum earth-volume coincides with minimum costs. An expensive part of the embankment comprises the revetment, and the slope area increases as the slope angle decreases. Careful attention is needed however, because the revetment costs are not always independent of the slope angle, e.g. for steep slopes heavy protection is needed while for mild slopes the (cheaper) alternative solutions (i.e., sand-mattesses, grass-mats etc.) can often provide sufficient protection.

Another point of economic optimization can be the available space for dike/seawall construction of improvement. All these factors should be taken into account optimizing the structure slope. The common Dutch practice is to apply a slope 1 on 3 on the inner slope and between 1 on 3 and 1 on 5 on the outer (seaward) slope. These mild slopes also minimize the scour at the toe of a dike (lower reflection). The minimum crest width is 2 m.

Present practice for obtaining a substantial reduction in wave run-up is to place the outer berm at (or close to) design water level. If the berm lies too much below that level, the highest storm flood waves would not break beneath the berm, and the run-up will be inadequately affected, thus providing relatively heavy wave loading on the upper slope. However, the optimum distribution of wave forces is obtained when the (relatively small) berm is about 0.5 to 1 wave-height below the design water level (see Figure 5). In such a case, the whole slope can be protected with the same (relatively small) units. When the berm is equal or somewhat higher than the design water level, the waves will always break on a lower slope. In this case, the lower slope needs much heavier protection (but along a shorter length) than the upper slope (very often a grass-mat is sufficient).

An important function of the berm can also be its use as an access road for maintenance or even for permanent use (promenade etc.).
The above discussion indicates that the proper choice of the shape is a very important step in the design of a dike/seawall. Mostly it is an iterative procedure.

It was decided in the Netherlands to base the design of all sea dikes fundamentally on a water level with a probability of exceedance of 10^{-4} per annum. In the Netherlands the wind set-up is mostly incorporated in the estimated storm-surge level. If this is not a case, the wind set-up should be calculated separately and added to the design water level. Besides the design flood level, several other elements also play a role in determining the design crest level of a dike (Figure 2).

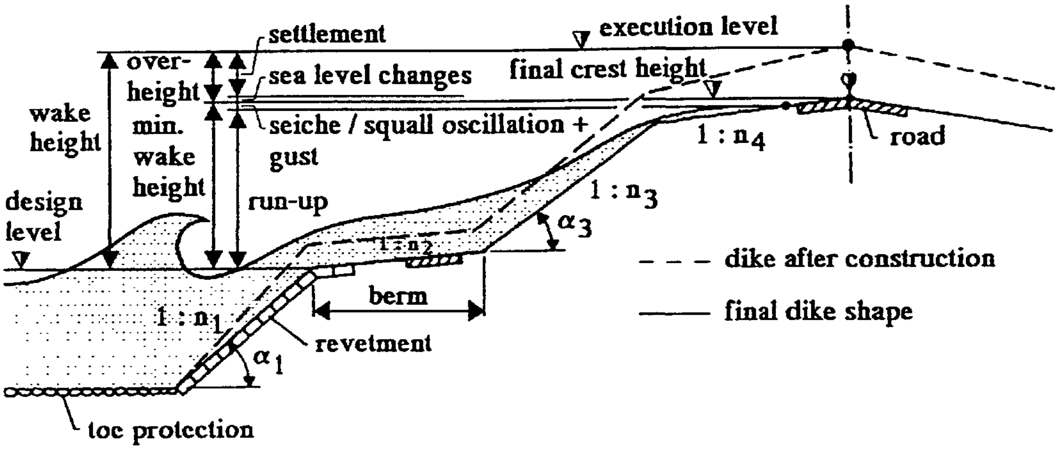


Figure 2 Determination of dike height

- Wave run-up (2% of exceedance is applied in the Netherlands) depending on wave height and period, angle of approach, roughness and permeability of the slope, and profile shape,
- An extra margin to the dike height to take into account seiches (oscillations) and gust bumps (single waves resulting from a sudden violent rush of wind); this margin in the Netherlands varies (depends on location) from 0 to 3 m for the seiches and 0 to 0.5 m for the gust bumps,
- A change in chart datum (NAP) or a rise in the mean sea level in the Netherlands: till now assumed roughly 0.25 m per century, in the future ~0.6 m per century (see Rijkswaterstaat, 1990),
- Settlement of the subsoil and the dike-body during its lifetime.

The combination of all the factors mentioned above defines the freeboard on the dike (called in Dutch as wake-height). The recommended minimum freeboard is 0.5 m.

The effective run-up (R), on an inclined slope can be defined as:

$R = R_n \gamma_R \gamma_B \gamma_\beta$

where:

R_n = run-up on smooth/plane slope (defined as a vertical height above SWL),

n = index of exceedance percentage, and

γ_R, γ_B, γ_β = reduction factors due to slope roughness, berm and oblique wave attack, respectively. The 2% run-up on smooth/plane slope is calculated from following equations:

$$\frac{R_{2\%}}{H_s} = (1.50 \div 1.75)\,\xi_p \quad \text{for} \quad \xi_p < \underline{2} \text{ to } 2.5 \ ,$$

and

$$\frac{R_{2\%}}{H_s} = (3.0 \div 3.5) \quad \text{for} \quad \xi_p > 2 \text{ to } \underline{2.5}$$

The lower numerical values (1.5 and 3.0) represents the average values while the higher values (1.75 and 3.5) represent the upper envelope of actual spreading in test results and, for safety reasons, are treated as representative ones for run-up calculations in the Netherlands.
The surf-similarity parameter (ξ_p) is defined by:

$$\xi_p = \frac{\tan\alpha}{(\sqrt{H_s/L_p})} = \frac{1.25}{(\sqrt{H_s})}\,T_p \tan\alpha$$

where:

H_s = significant wave height,

T_p = peak period, and

α = angle of slope

The reduction factor due to roughness (γ_R) is equal to 1 for asphalt and smooth concrete, about 0.9 for concrete blocks, pitched stone and grass, and about 0.50 - 0.6 for riprap.
The reduction factor due to the berm, accordingly to most recent tests on slopes 1 on 3 and 1 on 4, can be represented by

$$1^\circ \quad \left.\begin{array}{lll} \gamma_B = 1 - 0.06\ \xi_p\ B/H_s & \text{for} & \xi_p\ B/H_s \le 6 \\ \gamma_B = 0.6 \text{ to } 0.65 & \text{for} & \xi_p\ B/H_s > 6 \end{array}\right\}\ \xi_p < 86\ H_s/L_p$$

$$2^\circ \quad \left.\begin{array}{lll} \gamma_B = 1 - 5\ B/L_p & \text{for} & B/L_p \le 0{,}07 \\ \gamma_B = 0.6 \text{ to } 0.65 & \text{for} & B/L_p > 0{,}07 \end{array}\right\}\ \xi_p > 86\ H_s/L_p$$

In the Dutch practice, the upper level of protection is defined by $\le$ SWL + 0.5 R and $\ge H_s$ (sometimes $> 0.5\ H_s$).
Above this elevation the grassmat provides mostly a sufficient protection.

As it was already mentioned the height of a seawall is affected by functional requirements. In the case of a high-crested sea-wall (protection against inundation) the run-up and the necessary crest-height can be calculated accordingly to the common methods mentioned by Pilarczyk (1990) and in PIANC (1991/92). An example of possible variation of a crest-height as a function of a dike- or a seawall-shape is given in Figure 3.

results :	ctg $\alpha = 3$; $\gamma_b = 1$	ctg $\alpha = 4$; $\gamma_b = 1$	ctg $\alpha = 4$; $\gamma_b \cong 0.7$
storm surge MSL +	5.00	5.00	5.00
run-up	13.30	10.00	7.00
sea-level rise	0.25	0.25	0.25
seiches / oscillations	0.25	0.25	0.25
settlement	0.50	0.50	0.50
dike crest MSL +	19.30	16.00	13.00

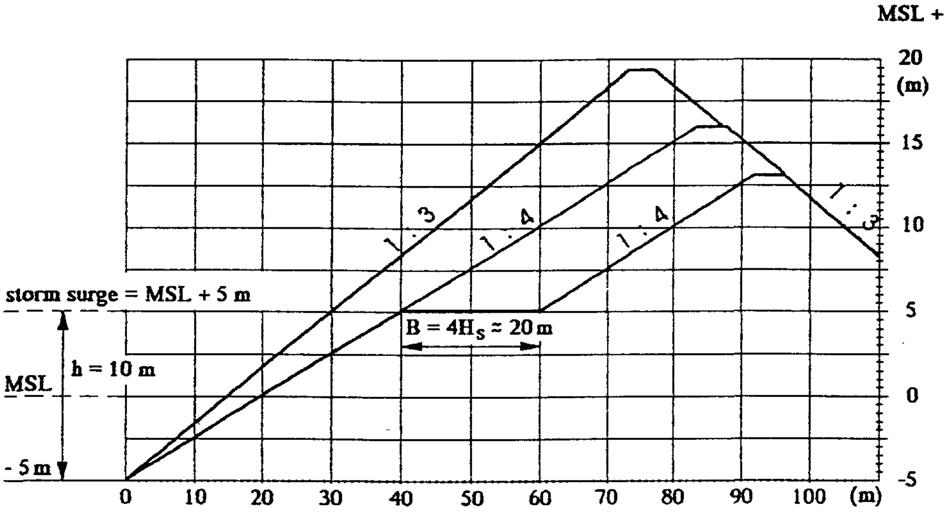


Figure 3 Example of dike calculation (alternatives); (H_s = 4.7 m, depth limited, T_p = 8.5 sec, berm reduction γ_B = 0.7 for ctgα = 4 and B = 4 H_s)

In the case of low-crest elevation (overtopping allowed) the design requirement is formulated in terms of the allowable amount of overtopping and the necessary protection of the splash-area (dimensions of units, length).

The method of calculation of overtopping discharge can be found in Pilarczyk (1990), SPM (1984), PIANC (1991/92), and other specialistic literature.

There are no generally valid recommendations for acceptable levels of overtopping for seawalls and/or dikes. In the standard Dutch practice a safe value 0.002 m^3/s for grassed crest and rear slope is recommended. Recent experience indicates that this value can be increased to 0.005 or even to 0.01 m^3/s for 'good' quality grassmat on clay-sublayer.

Information on a proper clay specification for a grassmat can be found in the guidelines (TAW, 1991).

Fukuda, et al. (1974), based on field observations, suggest the following figures on allowable overtopping related to inconvenience for persons or vehicles located 3 m behind the breakwater:
$q = 4.10^{-6}$ $m^3/m/s$: inconvenience for walking people
$q = 3.10^{-5}$ $m^3/m/s$: danger for people and traffic.
The design aspects on protection against overtopping are discussed in the next section.

Slope protection

General

In the past only local usage and experience have determined the selection of the type and dimensions of the coastal protection. Often designs were conservative and too costly or were inadequate. The technical feasibility and dimensioning of coastal structures can be actually determined on a more founded basis and supported by a better experience than in the past. Often, however, 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.

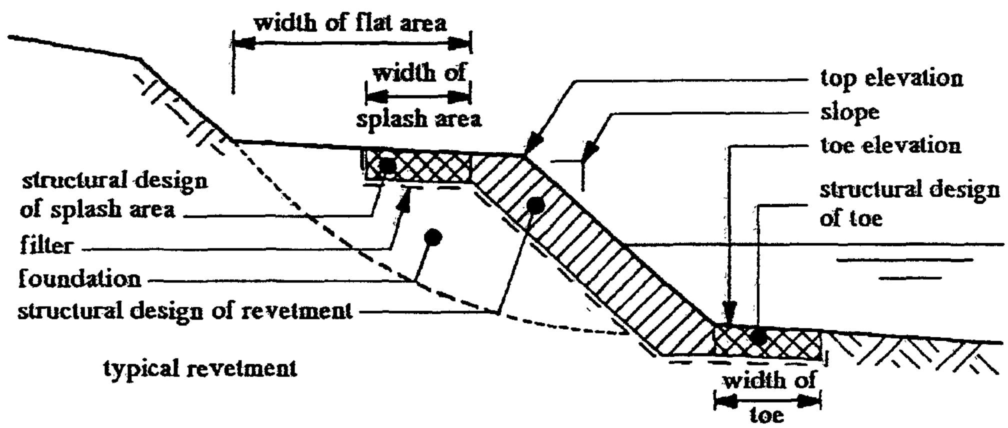


Figure 4 Design components of dike/seawall-revetment

Referring to the dikes/seawalls, a summary of the key elements that must be considered in the design (dimensioning) are illustrated in Figure 4.

The existing design rules for some (selected) structural elements (shape, height, cover-layer etc.) are briefly reviewed in the subsequent sections. For detail engineering (dimensioning) 'Coastal Protection' (Pilarczyk, 1990), the Manual on Rock (CUR/CIRIA 1991) and SPM (1984), can be of use.

Stability (or threshold conditions) for loose materials, from sand to rock, is investigated rather extensively, and the proper design criteria are availbale. However, the stability of randomly dumped quarried rock can often be substantially improved by taking

special measures (= composite systems, i.e., grouting, pitched stone, mattresses, etc.). On the other hand there are a number of protective systems related to artificial materials such as concrete (i.e., block-mats) and asphalt. Also grassmats serve as slope protection. For most of these systems it is possible to give some rough, indicative stability criteria which allow the engineer to make a comparison with a random placed rock, and thus, to make a proper choice of protection. The following systems will be considered: riprap, placed and/or pitched blocks/stones, grouted (bound) stones, bituminous systems, gabions/stone mattresses, fabric containers (bags, mats), and clay/grass-mats.
Because of the great variety of the possible composition of protective systems it is not possible to present a generally valid stability formulation for these systems. Therefore, only some principles and examples will be given here. For comparison of all these systems, the stability of dumped rock will serve as a reference.

Stability criteria for wave attack

The general empirical (approximate) formula derived by Pilarczyk (1990) and supported by large scale tests is:

$$\frac{H_s}{\Delta_m D} \leq \psi_u \phi \frac{\cos\alpha}{\xi_p^b} \; ; \; (\mathrm{ctg}\alpha \geq 2)$$

or

$$\Delta_m D = \psi_u^{-1} \phi^{-1} \cos\alpha^{-1} H_s \xi_p^b \; ; \; (\text{strength}) = (\text{load})$$

with:
ξ_p = breaker similarity index on a slope,

$$\xi_p = \tan\alpha \; (H_s/L_o)^{-0.5} = 1.25 \; T_p . H_s^{-0.5} . \tan\alpha$$

in which:
ψ_u = system-determined (empirical) stability upgrading factor (ψ_u = 1.0 for riprap as a reference and $\psi_u \geq 1$ for other revetment systems) [-],
ϕ = stability factor or stability function for incipient of motion, defined at $\xi_p = 1$ [-],
H_s = significant wave height [m],
T_p = peak wave period [s],
L_o = wave length [m]; $L_o = gT_p^2/2\pi$
D = specific size or thickness of protection unit [m],
α = slope angle [°],
Δ_m = relative density of a system-unit [-],
b = exponent related to the interaction process between waves and revetment type (roughness, porosity/permeability etc.), $0.5 \leq b \leq 1$. For rough and permeable revetments as riprap, b = 0.5.
For smooth and less permeable placed-block revetments it can be closed to b = 1. The value b = 2/3 can be treated as a common representative value for other systems (i.e. more open blocks and block-mats, mattresses of special design etc.).
D and Δ_m are defined for specific systems such as:

- rock ; $D = D_n = (M_{50}/\rho_s)^{1/3}$ and $\Delta_m = \Delta = (\rho_s - \rho_w)/\rho_w$
- blocks ; D = thickness of block and $\Delta_m = \Delta$
- mattresses; $D = d$ = average thickness of mattress and $\Delta_m = (1-n)\Delta$, where
  n = bulk-porosity of fill material and Δ = relative density of fill material. For common quarry stone $(1-n)\ \Delta \approx 1$.

For rock and $\xi_p > 3$, the sizes calculated at $\xi_p = 3$ can still be applied.
In the case of relatively impermeable core (i.e., sand or clay and limited number of waves ($N \approx 3000$)) the following indicative ϕ-values for rock can be determined:
$\phi = 2.25$ for incipient motion (motion 1 to 3 stones over the width of slope equal to D_n)
$\phi = 3.0$ as a first approximation for maximum tolerable damage for 2-layer system on granular filter (damage-depth less or equal to $2D_n$); $\phi = 3$ can also be applied for incipient motion of rock placed on permeable core (rockfill core or thick granular filter).
The ϕ-value equal to 2.25 will be used as a reference value for the stability comparison with other alternative systems. The difference with stability of rock due to the improving measures will be expressed by the upgrading of the factor ψ_u.
The important difference between the loose rock and the alternative systems concerns the behaviour of the systems after the initiated movement (damage). Due to the self-healing effect of the loose rock a certain displacement of rock units can be often accepted (up to $\phi \le 3$). In the case of alternative systems, i.e., block revetment, the initial damage (i.e. removing of one block) can easily lead to a progressive damage; there is no reserve-stability.
The comparison of stability of various systems (parameter ψ_u) and the necessary parameters for calculation purposes are contained in Table 1. Other structural requirements and design rules concerning revetments are given by Pilarczyk (1990).

Optimization of slope stability

The wave forces on a plane (continuous) slope are distributed rather unequally (the high wave-impact area near the water level, the intermediate uprush area and the low-attacked area beneath the point of breaking). The wave action on relatively fine materials indicates that nature tries to distribute the forces equally to provide equilibrium S-slopes. The same principle can be applied in designing the shape of seawalls and dikes leading to application of smaller protective units than in the case of a plane slope.
For practical reasons the 'optional' shape will be schematized to a trapezoidal profile. By selection of a proper position of a berm below the design water level and a proper width of a berm, the wave forces will be distributed in such a uniform way that the same material can be used along the whole profile. In this case the increase of stability (50% or more) can be realized by a berm with a width equal to 0.15 times wave length and situated (0.5 - 1.0) times wave height below design water level.

Based on the results of various studies, the indicative design guidelines have been prepared for riprap bermed slopes and toe-protection, as shown in Figure 5.

Criterion			Limits
$\frac{H_s}{\Delta_m D} = \psi_u \cdot \phi \frac{\cos\alpha}{\xi_p^b} = \psi_u \, 2.25 \frac{\cos\alpha}{\xi_p^b}$			$\phi_{(rock)} = 2.25$ $\mathrm{ctg}\,\alpha \geq 2$ $0.5 \leq b \leq 1.0$
System	ψ_u	Description	Sublayer
Ref.	1.0	Riprap (2 layers)	Granular
Rock	1.33	Riprap (tolerable damage)	Granular
Pitched	1.00	Poor quality (irregular-)stone	Granular
Stone	1.33	Good quality (regular-)stone	Granular
	1.50	Natural basalt	Granular
	1.50	Loose closed blocks; H_s < 1.5 m	Geotextile on sand
Blocks/	1.50	Loose (closed-)blocks	Granular
Block-	1.50	Blocks connected to geotextile	Granular
mats	2.00	Loose closed blocks	Geotextile on clay
	2.00	Cabled blocks/Open blocks (> 10%)	Granular
	≥2.50	Grouted (cabled-)blocks/Interlocked blocks adequately designed	Granular
Grout	1.50	Surface grouting (30% of voids)	Granular
	1.50	Pattern grouting (60% of voids)	Granular
Open Stone Asphalt	2.00	Open stone asphalt; $U_p \leq 6$ m/s	Geotexile on clay
	2.50	Open stone asphalt; H_s < 4 m	Sandasphalt
Gabions	2÷3.0	gabion/mattress as a unit, $H_s \leq 1.5$	Geotextile on sand or Geotextile on clay
	2÷2.5	stone-fill in a basket; $d_{min} = 1.8\ D_n$	
Fabric	1.00	$P_m \ll 1$ less permeable mattress	Sand or Clay and ev. Geotextile
Con-	1.50	$P_m \approx 1$ (P_m=ratio permeab.top/$_{sublayer}$	
tainers	2.00	$P_m \geq 2$ permeable mattress of special design	
Grass	–	Grass-mat on poor clay; U_p < 2 m/s	Clay (U_p= permiss. velocity)
		Grass-mat on proper clay; U_p < 3 m/s /s	

Table 1 Indicative categories for protective systems

For deep water wave conditions the reference for bermed (stepped) slopes is the stability of a straight slope. The increase in stability, called factor ψ_i, will be 1.0 if the bermed slope has the same straight slope (berm width, B = 0). The required stone size for the specific part of the bermed profile is equal to:

$$D_{n,B.S.} = \frac{1}{\psi_i} \times D_n \quad .$$

(bermed part) (straight slope)

For shallow-water wave conditions (wave-height depth limited, $H_s \approx 0.5$ h), the berm and down slope can be calculated as given under p.2° in Fig. 5. In this case the depth on a berm (d_B) is related to the depth (h) in front of the structure.
In the case of a higher position of the berm ($d_B/h \leq 0.4$) the down slope will be decisive for stability of the berm as a whole.
It is obvious that the guidelines presented in Fig. 5 can also be used, as a first approximation, for calculation of toe protection.
When applying this design concept, the stability of the chosen protective material should also be checked for conditions lower than design ones. In some cases the water-level may decrease quicker (i.e., due to tidal variation) than the waves, providing higher wave attack on lower parts of profile. The model investigation may provide the best solution in a particular case.

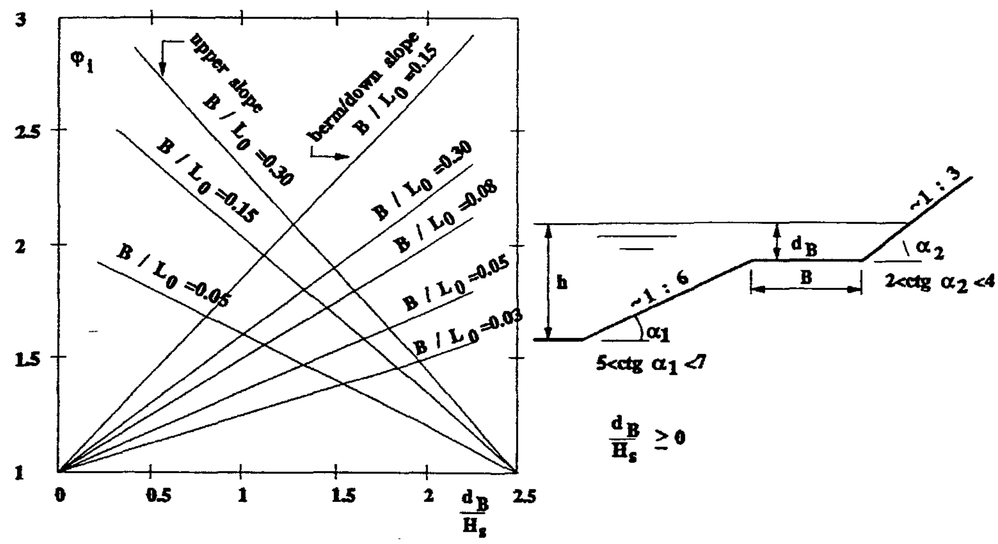
Scour and toe protection

Toe protection is supplemental armouring of the beach or bottom surface in front of a structure which prevents waves from scouring and undercutting it. Factors that affect the severity of toe scour include wave breaking (when near the toe), wave run-up and backwash, wave reflection, and grain size distribution of the beach or bottom materials.

Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Toe scour is a complex process. Specific (generally valid) guidance for scour prediction and toe design based on either prototype or model results have not been developed yet, but some general (indicative) guidelines for designing toe protection are given in SPM (1984).

The size of toe protection against waves can also be roughly estimated by using the common formulae on slope protection and introducing mild slopes (i.e. 1 on 10) and local wave height. The results in Figure 5 can also be used for this purpose.

Hales & Houston (1983) considered the stability of a rock blanket extending seaward from a permeable rubble slope on a 1:25 slope foreshore. They tested with regular waves to determine the conditions at which the rock in the scour blanket was just stable. To these conditions they fitted a mean trend given by:



1^o $H_s/h < 0.5$ $(h>2H_s)$

$$\underset{\text{berm/bermed slope}}{\left(\frac{H_s}{\Delta D_n}\right)_{B.S}} = \underset{\text{straight slope}}{\varphi_i \left(\frac{H_s}{\Delta D_n}\right)_{\alpha_i}} = \begin{cases} \left(\frac{H_s}{\Delta D_n}\right)_{\text{berm/ down slope}} = \varphi_{\text{berm/ down slope}} \times \left(\frac{\phi}{\sqrt{\xi(\alpha_1)}}\right) \\ \left(\frac{H_s}{\Delta D_n}\right)_{\text{upper slope}} = \varphi_{\text{upper slope}} \times \left(\frac{\phi}{\sqrt{\xi(\alpha_2)}}\right) \end{cases}$$

For $\frac{B}{L_0} \leq 0.15$ $\qquad \varphi_{\text{berm}} \cong 1.1 + 5\ \frac{B}{L_0}\ \frac{d_B}{H_s}$ $\qquad$ - for berm and down slope

Indication of optimal value

for whole profile $\left(\frac{H_s}{\Delta D_n} = \text{const.}\right)$

at $\frac{d_B}{H_s} \sim 1$ and $\frac{B}{L_0} \approx 0.15$

$$\begin{cases} \left(\frac{H_s}{\Delta D_n}\right)_{B.S} = 8.6\left(\frac{H_s}{L_0}\right)^{1/4} & \text{short waves } \frac{H_s}{L_0} \gtrsim 0.015 \\ \left(\frac{H_s}{\Delta D_n}\right)_{B.S} \approx 3 & \text{long waves } \frac{H_s}{L_0} < 0.015 \end{cases}$$

2^o $H_s/h \approx 0.5$ $\quad (H_s \approx 0.5h = \text{depth limited})$

$\frac{H_s}{\Delta D_n} = 10\ \frac{d_B}{h} - 1.5$ $\qquad$ for $0.4 < \frac{d_B}{h} < 1$

$\frac{H_s}{\Delta D_n} = \left(\frac{\phi}{\sqrt{\xi(\alpha_1)}}\right) > \frac{\phi}{1.5}$ (limit) $\qquad$ for $\frac{d_B}{h} \leq 0.4$

$\phi \approx 2.25$ for rip-rap

Figure 5 Stability of bermed slopes and toe-protection acc. to Pilarczyk.

$$\frac{H_b}{\Delta D_n} = (1.75 \div 2.85) \left(\frac{B_p}{L_s}\right)^{2/3} \approx 20\ (B_p/L_s)^{2/3} = 20 \left(\frac{B_p}{T\sqrt{gh_s}}\right)^{2/3}$$

(coefficient 17.5 represents a more conservative line)

where:
H_b is the breaker wave height ($\approx$ 0.78 h_s for regular waves),
L_s is the wave length in shallow water, given by $T(g\ h_s)^{1/2}$ in this instance,
B_p is the seaward extend of the toe protection,
h_s is the shallow water depth.
This formulation can be used as a first indication of decreasing of stone size (D_n) with the distance (B_p).

Note that the conservatism of the apron design (width and size of cover units) depends on the accuracy of the methods used to predict the waves and current action and to predict the maximum depth of scour. For specific projects a detailed study of scour of the natural bottom and at near by similar existing structures should be conducted at a planned site, and/or model studies should be considered before determining a final design.
In all cases, experience and sound engineering judgement play an important role in applying these design rules.

Protection against overtopping

If a structure (revetment) is overtopped, even by minor splash, the stability can be affected. Overtopping can: (a) erode the area above or behind the revetment, negating the structure's purpose; (b) remove soil supporting the top of the revetment, leading to the unraveling of the structure from the top down; and (c) increase the volume of water in the soil beneath the structure, contributing to drainage problems. The effects of overtopping can be limited by choosing a higher crest-level or by amouring the bank above or behind the revetment with a splash-apron. For a small amount of overtopping a grass-mat on clay can be adequate. The splash apron can be a filter blanket covered by a bedding layer and, if necessary to prevent scour by splash, riprap or pavement of concrete units or asphalt.

No definite method for designing against overtopping is known due to the lack of the proper method on estimating the hydraulic loading. Pilarczyk (1990) proposes the following, indicative way of design of the splash area (Figure 6):

$$\frac{H_s}{\Delta D_n} = \frac{2\cos\alpha_i}{\Phi_T\ \xi^{2b}\left(1 - \frac{R_c}{R_n}\right)}$$

where:
H_s = significant wave height,
ξ = breaker index; $\xi = \tan\alpha(H_s/L_o)^{-0.5}$,
α = slope angle,
α_i = angle of crest or inner-slope,
L_o = wave length,

b = coefficient equal to 0.5 for smooth slopes and 0.25 for riprap revetments,
R_c = crest-height above still water level,
R_n = wave run-up on plane slope,
D = thickness of protective unit ($D = D_n$ for rock), and
ϕ_T = total stability factor equal to 1.0 for rock, 0.5 for placed blocks and 0.4 for blockmats.

The width of protection in the splash-area, which is related to the energy decay, can be roughly assumed as equal to:

$$L_s = \frac{\psi}{5} T \sqrt{g(R_n - R_c)} \geq L_{min}$$

with a practical minimum (L_{min}) equal to total thickness of revetment (incl. sublayers) as used on the slope (= a minimum length of transition from the slope into the crest). ψ is an engineering-judgement factor related to the local conditions (importancy of structure), $\psi \geq 1$.

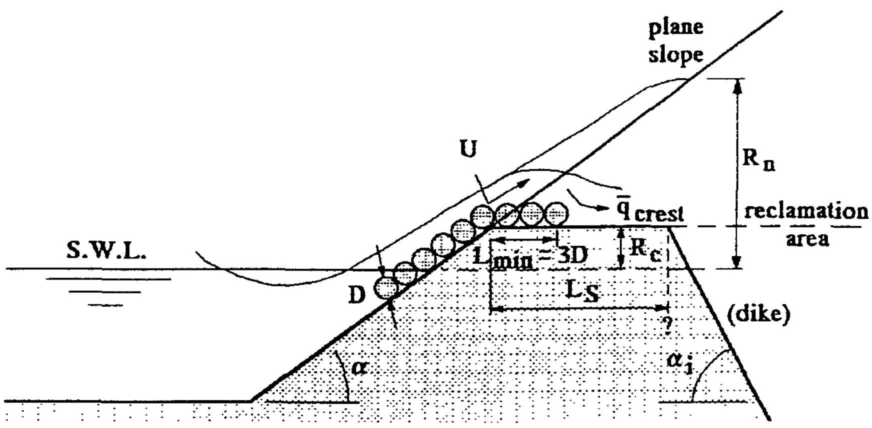


Figure 6 Definition of splash area

Stability of rockfill-protection of crest and rear slope of an overtopped or overflowed dam or dike can also be approached with the Knauss formula (Knauss, 1979). The advantage of this approach is that the overtopping-discharge, q, can be used directly as an input parameter for calculation. Knauss analysed steep shute flow hydraulics for the assessment of stone stability in overflow rockfill dams (impervious barrages with a rockfill spillway arrangement). This kind of flow seems to be rather similar to that during high overtopping.
His (simplified) stability relationship can be re-written in the following form:

$$q = 0.625 \sqrt{g} (\Delta D_n)^{1.5} (1.9 + 0.8 \Phi_p - 3\sin\alpha_i)$$

in which:
q = maximum admissible discharge ($m^3/s/m$)
g = gravitational acceleration (9.81 m/s^2)
D_n = equivalent stone diameter, $D_n = (M_{50}/\rho_s)^{1/3}$

Δ = relative density; $\Delta = (\rho_s - \rho_w)/\rho_w$
α_i = inner or crest slope angle
ϕ_p = stone arrangement packing-factor, ranging from 0.6 for natural dumped rock-fill to 1.1 for optimal manually placed rock; it seems to be reasonable to assume ϕ_p = 1.25 for placed blocks.

Examples of revetment failures

Zuidwal revetment (Rotterdam)

During the severe storm of February 28th - March 1st 1988, serious damage took place on the revetment (location Zuidwal) at the entrance to the port of Rotterdam. The measured wave conditions outside the entrance (deepwater) were: H_s = 5.5 m and T_p = 10 to 12 sec. The highest water level was NAP +2.2 m followed by the next high-water at NAP +1.3 m. The respective calculated wave heights at the toe of the structure at these high water-levels were H_s = 3.8 m and 3.5 m. The damage took place at the end of storm ($H_s \approx 3.0$ m).
The revetment consisted of columnar blocks (Basalton type) of 0.35 m height and density ρ_s = 2300 kg/m³. The toe of the revetment was located at NAP (normal average waterlevel) while the crest-berm was located at NAP +6.20 m. The average slope was 1 on 8 (locally 1 on 6, vertical to horizontal). Underneath the toplayer, a filter layer of silex (30-90 mm) with a thickness of 0.3 m was placed. The base soil consisted of a mix of a coarse sand and gravel. The interspaces in the revetment were grouted (blinded) by steel-slags (16-32 mm). The toe was constructed by a pile-row and a large stones (2000 to 6000 kg) on a filter-layer of rock (30-300 mm), and thickness of 0.5 m. The crest-berm was protected by rock (30-300 mm) on a slope 1 on 10. This crest-berm lies in front of a dune.
Accordingly to the visual inspection, the failure took place at the end of storm on March 1st. The revetment was completely removed for width of 200 m from the toe to the top (berm), some pieces of rock (2000 to 3000 kg) from the toe-structure were found on the slope up to NAP +5 m.
Check of stability: ψ_u = 2.5 (upgrading factor for basalton blocks), $\xi_p \approx 1$, $\cos\alpha \approx 1$, Δ = 1.25 (in sea-water).

$$\text{Required: } \frac{H_s}{\Delta D_n} = \psi_u . 2.25 \frac{\cos\alpha}{\xi_p^{2/3}} = 5.6$$

$$\text{Failure conditions: } \frac{H_s}{\Delta D_n} = \frac{(3.8 \text{ to } 3.0)}{1.25 \,.\, 0.35} = 8.7 \div 6.9 > 5.6 \text{ (required)}$$

It is obvious that the real stability was highly exceeded during the storm. Moreover, careful inspection and measurements on the undamaged sections adjacent to the damaged section indicated that due to the cementation (hydraulicity) of steel-slags (revetment was about 10 years old), the permeability of the toplayer decreased to 10^{-6} m/s for lower part of revetment and 10^{-4} m/s near the crest, in comparison with a permeability of 10^{-2} (to 10^{-3}) for a 'new' grouted revetment. The difference in permeability was at least a factor of 100, providing increase in uplift pressure by a factor of more than 4. This is probably the real explanation of this failure.

In a new design (for design wave H_s = 4.5 sec, frequency 1/100 per year), the thickness of blocks was choosen equal to 0.45 m with ρ_s = 2900 kg/m³, with surface grouting by a broken gravel 5-50 mm (no cementation process for this natural material), and unsorted mine-stone as a sublayer (low permeability). Check of stability:

$$\frac{H_s}{\Delta D} = \frac{4.5}{1.85 \cdot 0.45} = 5.4 < 5.6$$

N.B. $\Delta \approx 1.85$ (in sea-water conditions)

Test embankments IJssel-lake (Houtrib-dike)

The prototype tests on 4 types of blockmats and one on riprap (as a reference) along the Houtrib-dike in the IJssel-lake (near Lelystad) were carried out in view of future development plans in that area. The test sections were installed in September 1986. They were prone to wave attack from westerly directions; the fetch length being some 25 km. The lake normally has a more or less stagnant waterlevel. During storms, wind and wave set-up may occur. The waterdepth at the toe of the dike amounts some 4 metres. The dimensions of each of these types of blockmats were determined in such a way that it would be likely that the actual wave attack causing instability would come close to the design conditions in a period of 2 years ($H_s \approx 1.2$ m, T_p - 4 sec). The slope of the revetment was taken at 1 on 4 (vertical to horizontal) with a small berm at SWL -2 m. The width of each test section was 40 metres.

The following mat types were selected for these tests (from south to north).

- V.O.B.-mat: concrete blocks fixed to a geotextile by synthetic pins. The blocks are fixed to the geotextile blocks in a rectangular pattern. The connection between the blocks is secured by the geotextile. Standard sizes of the mat are 4 x 6 or 4 x 8 metres.
- Asam-mat: concrete step-blocks in "half-stone" bond, joined by 2 cables (stainless steel) per block and interlocked by cams. The shape of the blocks is a trapezium, in such a way that the top of each block is more or less horizontal after placing it on a slope. Standard size of a mat is 3 x 4 metres.
- Armourflex-mat: basically the same idea as the Asam-mat, although the shape of the blocks is different (no trapezium). The blocks applied in the tests were closed concrete blocks.
- Beto-mat: basicly the same principle as the Armourflexmat, the main differences being the shape of the blocks and the method of connection of the blocks with 2 steel cables per block.
  For the reference sections at both sides of the test sections, a mixture of 10-300 kg unit stone weight (riprap) has been taken (170 kg as average), with a density of 2700 kg/m³.

For the mat sections the following weights were taken:
- VOB-mat : total weight 340 kg/m²
- other mats: total weight 305 kg/m².

The specifications of protective units can be summarized as follows:

revetment type	D	Δ	ΔD	$\frac{H_s}{\Delta D}$ $H_s = 1.2$ m
rip-rap (D_n)	~ 0.40 m	1.7	0.68	1.76
V.O.B.	~ 0.15 m	1.3	0.195	6.15
ASAM	~ 0.17 m	1.3	0.22	5.45
Armorflex	~ 0.15 m	1.3	0.195	6.15
Beto	~ 0.17 m	1.3	0.22	5.45

The required stability factor ($H_s/\Delta D$) for the blockmats is rather high (~6) in comparison to a standard criterion of about 4. The reason of that was the availability of the standard block-mats.
Under all mats a geotextile has been applied to prevent washing out of fine materials through the gaps between the blocks (especially for the Asam, Armorflex and Beto types). Under the geotextile a layer (0.5 m thick) of minestone 10/250 mm has been applied, to be placed on top of the sandy core of the dike. At a part of the test sections the geotextile was placed directly on the sand at the upper part of the slope (above SWL).

Approximately 1 month after completion of the test sections (October 20-21, 1986), a severe storm occurred, causing damage to all block-mats. The estimated wave height during this storm exceeded the design conditions by some 30-40% (H_s ~ 1.7 m, T = 5 sec, max. wind set-up of waterlevel ~ 1 m). It is obvious that failure of the slope protection is very likely in this case. Some very restricted (limited) damage to the riprap sections was according to expectations. However, it should be mentioned that during the first day of the storm (20/10) when the max. wave height was about H_s ~ 1.2 m, no damage was observed during visual inspection. This supports the validity of present stability criteria.

Discussion of failure mechanisms:
The storm of 20/21th October 1986 caused such a high exceedance of the design loads that all mat types were seriously damaged. These circumstances make it impossible to refine the stability criteria of these mat types on the basis of the experiments. On the other hand, the high exceedance of the design loads resulted in a number of failure mechanisms, that occurred simultaneously.

The following mechanisms could be observed after the storm:
- <u>lifting up of the mats</u>, both of individual blocks and of groups of blocks.
  This phenomenon leads to pulling forces in the connections between blocks (cables or geotextile) and to deformation of the subsoil;
- <u>loosening of the connection</u> between 2 adjacent mat sections as a result of the increasing forces when (part of) the mat is lifted up;

- <u>sliding down of the mats</u> as soon as the connection between 2 adjacent mat sections has failed. Sliding may take place between the geotextile and the subsoil and also between the blocks and the geotextile (the latter for the types without block to geotextile fixation);
- <u>turning over of mat ends</u>, especially after failure of the connection between adjacent sections. According to the design rules, open mat ends without any anchoring or connection should be treated as a single block;
- <u>bobbing up of the mats</u> at the downward end of a mat section that has been lifted, resulting in an ongoing sliding down process;
- <u>breaking of steel cables</u>, causing a loss of the internal connection of a mat section; the blocks act as single stones then, with much lower stability level;
- <u>erosion of the toe material</u>, consisting of gravel 30-200 mm (1000 kg/m') on minestone. The toe level was some 2.5 m below the still water level during the storm, being too high to withstand the wave forces;
- <u>erosion of the sublayers</u>, starting from holes in the slope protection and from the eroded toe.
  This erosion was observed at the sections where the geotextile was placed directly on the sand and also at the sections where a 0.5 m thick layer of minestone was applied, although the extent of the erosion in the latter situation was less.

The most important lesson of the tests at the Houtrib-dike is that the weakest point of the blockmats is the joint between two adjacent mat sections.

<u>Conclusions</u>

1. The limitation of this paper does not allow to preparation of a fully (detailed) evaluation of the available Dutch experience on design of dikes and revetments. The background information can be found in the reports as mentioned in the references.
2. Dikes and/or seawalls are only one option for coastal defence and must be considered in conjunction with, or as an alternative to, beach management and other options.
3. The shape and slope gradient of a dike/seawall affects strongly the scouring process, run-up (height of a dike) and overtopping, and dimensions of slope protection units.
4. The proper design of toe protection (against scouring), slope protection (against wave forces), and protection against overtopping can be decisive for the total stability of a dike/seawall. Although definitive design methods are still lacking, the proposed design rules can be of use in particular cases.
5. Further prototype verification of developed dimensioning criteria is still needed. Careful evaluation of prototype failure-cases may provide useful information/data for verification purposes.
6. Waste materials may certainly be useful in hydraulic engineering, provided their specific weaknesses are recognized and effectively dealt with.

7. In all cases, experience and sound engineering judgement play an important role in applying these design rules, or else mathematical or physical testing can provide an optimum solution.

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App.5.1 (K.W.Pilarczyk) Dutch Experience on Design of Dikes and Revetments

<u>Keywords</u>

sea-dikes
seawalls
revetments
shore protection
riprap
blockmats
design criteria
revetment failures

APPENDIX 6-1
Natural vs. Man-Made Coastal Defence & Management

The Nature developed all types of shore protection long time before men analysed them and drew instructive conclusions. Consequently we should learn more from the Nature. This would mean that the right type of field data must be obtained. We must also admit that the Nature has been more imaginative and has often achieved more success than man. Perhaps what we observe in the Nature and collect as our discovery is mainly the success the Nature has achieved ; if the Nature had failed, nothing would have been left.

Table App.6-1.A
Natural and Man-Made Coastal Protection (Brunn, 1972)

Natural	Man-made
hline Shore rock	Sea wall
Rock reef	Submerged bulkhead or mound
Rock island	Offshore breakwater
Headlands	Large breakwater perpendicular to or at an angle with the shoreline
Rock perpendicular to shore	Groynes
Sea floor vegetation	Bottom mattresses
Dune	Dyke
Material transfer to shore by:	
Wind drift	Articial nourishment from
Rivers	land sources
Longshore littoral drift and sea bottom onshore drift	Artificial nourishment from offshore sources
Natural bypassing of drift at tidal inlets	Mechanical bypasing of drift at tidal inlets

An expansion of this table has been copied from Bruun (1972).
The idea of simulating and reproducing the Nature's patterns of coastal defence and management has been taken up by Khomitskiy (1983), who proposed a few practical schemes, cf. Appendix 7-2. It has also been advocated by Pilarczyk, on a number of formal and informal occasions.

B *Effectiveness of coastal defence measures*

Table App.6-1.B
Bruun's Expanded List of Natural and Man-Made Protection Structures

Natural Shore Protection	Man-made Equivalent	What It Does
Shore Rock	Sea Wall	Resists erosion by hardness of face
Rock Reef	Submerged bulkhead or breakwater	Forces larger waves to break on it, reduces wave access to shore, reduces wave attack in its shadow.
Rock Island	Offshore breakwater	Same as for rock reef.
Headland	Large breakwater perpendicular or at an angle to the shore.	Provides anchor for a stable beach or a bay form
Rock ledge or boulder ridge perpendicular to shore	*Groynes	Traps sand and encourages beach formation. Reduces sand demand for beach maintenance.
Low rock ledge or boulder formation parallel to shore	*Beach sill or armoured berm	Stabilizes bank toe. Reduces wave backrush effects and height of wave uprush.
Rock or boulder talus along the foot of shore cliff	*Revetment	Protects underlaying softer bank strata from wave action resists erosion.
Cobble or boulder beach cover	*Armoured beach, concrete block pavement or rock mattresses	Protects lower beach and offshore bed from erosion
Bay Form	Artificial bays, multiple bays formed between artificial headlands, long, high, widely-spaced groynes	Changes the ratio of beach material stripping and deposition tendencies by waves. Allows for development of more stable beach with lower littoral supply
Lake Botton Vegetation	Flexible mattresses	Reduces or prevents lake bottom erosion, dampens waves reaching shore
Lake Surface Vegetation	Floating Breakwaters	Dampens waves reaching the shore.
Dune	Dyke, artificial dune	In extreme storms, forms barrier and prevents inundation of low shoreline areas. Dunes provide reserve sand supply to fronting beaches.
Material transfer to shore by:		
Wind drift, rivers, shore erosion, longshore littoral drift, lake bottom transfer	*Artificial nourishment from land sources	Ensures continuity of supply of beach material, keeps the supply and loss of beach material in balance.
Natural by-passing of drift at bays, river mouth and headlands	Artificial nourishment from offshore sources, mechanical by-passing of drift at littoral barriers.	

*Protection methods most appropriate for use by individual landowner.