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GUIDE TO CONCRETE DYKE REVETMENTS

Report No SR 65
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Note:

This document is an English translation of the Dutch report "Leidraad cementbetonnen dijkbekledingen", Rapport 119, Stichting voor onderzoek, voorschriften en kwaliteitseisen op het gebied van beton (CUR-VB), September 1984.

The translation was performed by Mr C van Beesten, MSc, FICE, MIWES, Consulting Engineer, on behalf of Hydraulics Research Limited, and with the permission and co-operation of CUR-VB (Netherlands Committee for Research, Codes and Specifications for Concrete). The cost of the translation was funded in part by the Ministry of Agriculture, Fisheries and Food, under their Research Commission BA, Sea Defence Structures - Revetments and Breakwaters.

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NOTATION

b	=	thickness of filter layer
C	=	wave celerity
C_o	=	wave celerity in deep water
C_g	=	celerity of wave-groups
D	=	sieve opening diameter for granular filter
d	=	diameter ; depth of water ; thickness revetment blocks
E	=	energy density
$F_S(S)$	=	load probability function
$F_R(R)$	=	strength probability function
f	=	frequency
g	=	gravitational acceleration
H	=	wave height ; vertical water level variation
H_s	=	significant wave height
I_c	=	consistency index
I_p	=	plasticity index
k	=	filter layer permeability coefficient
k'	=	revetment permeability coefficient
L	=	wave length
L_o	=	wave length in deep water
n	=	ripening factor for clay
O	=	largest orifice in geotextile filter
R	=	structural strength
S	=	structural loading
T	=	wave period
$\bar{T}$	=	average wave period
X_n	=	base variables
Z	=	safety function

NOTATION (CONT'D)

α = revetment slope angle

γ = safety coefficient

Δ = relative density of revetment material

λ = leakage length

ξ = wave-breaking parameter

ρ_b = volumetric mass of the revetment

ρ_w = volumetric mass of water

$\Delta\Phi$ = maximum pressure under block revetment

GUIDE TO CONCRETE DYKE REVETMENTS

Foreword

On the initiative of the contact-group "Wet hydraulic engineering", which includes representatives of the Dutch cement-industry, the Government Public Works Department, Technical University at Delft and the Agricultural University of Wageningen, a decision was made to commence a study on the subject of concrete block revetment on bank slopes, in coordination with the "Technical Advisory Committee on Sea and Freshwater flood defences (TAW), and the Organisation for research, specifications and quality requirements for concrete (CUR-VB). A study-committee was formed for that purpose in 1981, which - in TAW connection - is set up by work-group 4 "Bank revetments" and - in CUR-VB connection - by the investigation-committee C45 "Concrete bank revetments". The Committee had the task of writing a guide concerning design, construction, management and maintenance of concrete block revetments.

The reasons for this study included:-

- there were no specific requirements formulated at that time;

- the possibilities for application (of blocks) are determined by the product systems placed on the market by the manufacturers; only experienced organisations and management in this field can pass judgement on the applicability of these systems;

- the existing gaps in the knowledge can lead to over- or under-dimensioning;

- concrete block revetments can in future become economically more attractive than asphalt revetments or natural stone (rock) and concrete could therefore be applied more frequently.

The results of the study have been presented here in the form of a guide. This guide is primarily directed to those technical staff immediately involved in design and management, and employed by land-drainage boards ("Water-Schappen" include land drainage and banks), consulting engineers, provincial public works departments and the Government public works department. The guide is not intended to be a scientific publication in which theoretical bases are thoroughly discussed. It has been attempted to present as much as possible of the background information without providing a solution for each possible problem. The latter is, in connection with different circumstances (geographical as well as other aspects), not only impossible but also undesirable, because it could result in an inflexibility towards other solutions.

The matters discussed in this guide need to be viewed as a summary of criteria which have to be satisfied in the design of a concrete block revetment. The accent here is more on the behaviour of a concrete block revetment under the influence of hydraulic loads and on guide lines for the execution of the construction, than on the considerations of concrete as a material.

In writing the guide, use has been made of recently carried out research at the (Delft) Hydraulics Laboratory and the (Delft) Soil Mechanics Laboratory. However, that research has not yet progressed to the extent that the strength of various types of revetment are known completely. This guide will, therefore, have to be modified in due course.

In order to keep the guide (comparatively) concise and readable, many points have been treated rather briefly. More background information can be obtained from the CUR-VB/COW report "Background to the guide for concrete-block bank revetments" obtainable from COW (Centre for Research into Sea defences). In that report the overall problem has been approached more theoretically and extensive references to literature have been included. It should, however, be stressed that this guide can be used as an independent unit, without needing the assistance of the other report. Both the guide and the background report were written by Ir G M Wolsink.

The committee, at the time of publication of the guide, comprised the following members:-

Professor ir A Glerum, Chairman	Delft University
ir G M Wolsink, Secretary	Delft University
C C Bakker	De Hoop B V
ir W Bandsma	Government Public Works Dept, Roadconstruction
ir H Burger	Government P W D Weirs & Sluices
ing M C P Cok	Gebr. Van Oord B V
ir E H Ebbens	Govt P W D, Centre for Research on Sea Defences
ing T J Leenknegt	Provincial P W D, Zeeland
ing M G M Pat	Delft University
ing L A Philipse	Waterboard (Drainage) Fryslân

SUMMARY*

This set of guidelines is intended for engineers and technicians directly associated with the design and management of dykes. It is not intended as a scientific work dealing exhaustively with theoretical fundamentals. It has been endeavoured as far as possible to give background information without offering a solution for every conceivable problem. For a treatment of these matters in greater depth the reader is referred to CUR-VB/COW report "Background to the guide to concrete dyke revetments" (32). With a view to application more specifically in the Netherlands, the treatment of the subjects is confined to the type of revetment composed of relatively small units

In the first part of this century a considerable amount of in-situ placing of concrete - then still a new material - was carried out, not always with favourable results. In due course the use of precast concrete units was increasingly adopted (Chapter 1). At first, manual methods were used for producing them and placing them on the slopes to be proctected. The units were made in a wide variety of shapes. Mechanization was subsequently introduced into the production process, and the handling and installation of the units were also to a great extent mechanized. In conjunction with this development the complexity and variety in the shapes of the units were reduced.

For the revetment, i.e. the protective covering, of a water-retaining structure requirements are formulated with reference to the purpose of the revetment, the technical features of constructing it, and possible special circumstances involved (Chapter 2). Various types of revetment are distinguished with reference to the properties of the units and of the base on which they are installed (Chapter 3). The material-technological properties of concrete in a maritime environment are only briefly considered, because comprehensive and readily accessible literature and codes of practice are available on the subject (Chapter 4).

Requirements are applied to the base layers of the revetment because these are important in maintaining its stability under wave action and in ensuring that the structure will continue to function permanently (Chapter 5). In this connection a distinction is drawn between permeable and impermeable bases. Research carried out at the (Delft) Hydraulic Engineering Laboratory has shown that an impermeable layer (clay) gives the revetment greater stability under wave attack than a permeable layer does. In order to derive lasting advantage from this greater stability, it is necessary to lay down requirements as to the material properties and the manner of use of the clay, while the circumstances of the job may impose restrictions on applicability.

It is stated what materials can suitably be used for a permeable layer and what requirements they must satisfy, more particularly with regard to penetration of material from the subgrade into the filter material.

Wave attack, which is a major factor governing the stability of the revetment, has a different frequency of occurrence at each level of the slope and varies in magnitude. This depends on the type of dyke concerned, eg, a sea dyke, and on many other factors. These matters are explained with reference to defined zones of loading (Chapter 6).

* From English summary incorporated in the Dutch report

The shape of the cross-sectional profile of the dyke is of influence on the type of concrete unit suitable for revetment construction (Chapter 7).

In the experience of many dyke managers, substantial damage is liable to occur at the transition from one type of revetment to another and in zones where the revetment ends. Although it is not practicable to give standard solutions, outright mistakes can be high-lighted. The toe construction, the upper boundary of the hard revetment and the transition to a diferent type of revetment are considered (Chapter 8).

The next two chapters are concerned with construction (Chapter 9) and with management and maintenance (Chapter 10).

The loads acting on the structure and its strength are then considered. First, the hydraulic boundary conditions such as wave characteristics, wave fields and wave deformations (including the breaking of waves) are dealt with (Chapter 11); next, special loads are reviewed (Chapter 12).

As an interim result of long-term research still in progress, some information concerning the stability of the installed revetment is given (Chapter 13). On the one hand, results of theoretical model studies and, on the other, results of recent research in a wave flume (1:2 linear scale) are reported. Because of the complexity of the subject there is as yet no simple-to-use mathematical model available for dealing with various kinds of revetment and subgrade. All the same, with the aid of the data yielded by empirical research it is possible to determine approximately the thickness of one of the given types of revetment.

Both the magnitude and the location of the load acting on the revetment, as well as the strength of the revetment as installed on the dyke slope, are subject to scatter. Particular values have a particular probability of being exceeded or of not being attained. Safety considerations are presented with reference to this (Chapter 14). It is indicated what approach might be possible with a view to obtaining a better understanding of the actual safety of a concrete revetment on a dyke. Linking up with the work of the Delta Commission, an order of magnitude for the safety level is given.

1 INTRODUCTION

The revetment of an embankment will be defined in the context of this guide as that part of the total covering layer that is loaded directly by the waves. Under the revetment is a sub-layer of clay, granular material or bitumenised sand which in the more modern embankments forms the protection for the sand body of the core. An intermediate layer sometimes exists between cover and sub-layer.

In order to meet the requirement for the cohesionless granular material to let no sand through, geotextile cloths or sheets can be added to the construction.

The purpose of the revetment of an embankment is, in conjunction with the sub-layer, to protect the body of the bank against erosion due to waves, currents and other more particular loads such as ice drifts.

The attention in this guide will be mainly directed to sea-walls (embankments along the sea); only occasional reference will be made to lake and river banks.

The first use of concrete in applications to strengthen and protect sea defences dates back to the beginning of this century. First use was probably to fix old and worn slopes of hand-placed natural stone or rock by filling holes with concrete. This produced a dense and closed surface, but also made a rigid monolith which left the whole construction no freedom of movement so that any loss of sub-layer material or soil could lead to breaks in the sub-soil. The original flexibility of the stone cover was thus lost.

The results obtained with concrete were, in general, initially not satisfactory, which could be ascribed to the rigidity of early applications but also to the inadequate knowledge of the appropriate concrete mixes to achieve a strong and dense concrete. Many disappointments occurred, especially when fresh concrete, placed in situ, came into contact with sea water (below and immediately above high water). The slopes thus treated began to look shabby after a short period of time.

Precast concrete blocks were in the past also often manufactured on site. Again as a result of insufficient knowledge of concrete as a material, careless production (unskilled labour, inadequate equipment, etc) the blocks were corroded and damaged ?
by wave impact, and such blocks would fall apart and return to loose aggregate state.

In choosing the shape of blocks a lot of attention was paid in the past to a dense and yet flexible link

abraded ?

between the elements of the revetment, in order to give the slope construction sufficient elasticity to permit some settlement of the foundation. Systems which did exist in the past as marketable units with all sorts of refined shapes to interlock have largely disappeared for economic reasons. The complicated shapes made mechanical manufacture often difficult if not impossible, whilst during the revetment construction the placing of such blocks was equally difficult. From the start of concrete block application until about 1965, the placing of blocks remained almost exclusively a hand operation. Each block was placed individually, by one or two men depending on the weight, by hand on the bank slope. The rate of production of such revetments, using the larger blocks, was not great. As the construction works increased in size, made possible by faster delivery of sand (for bank cores) and greater mechanisation to reduce labour costs, a search had to be made for a faster method of placing. It is the use of special blockclamps, with the ability to handle several blocks in one operation, which permitted greater production by placing them on previously prepared parts of the bank slope.

The manufacture of concrete slope revetments or individual blocks, in situ where they are needed for construction, and especially pouring concrete in the tidal zone, has to be discouraged according to experience. Only when production takes place in concrete block factories where the mixed concrete can be pressed, vibrated or shaken, can satisfactory results be expected from normal concrete. Casting concrete in situ in the tidal zone can exceptionally be tolerated as a result of more recent developments in the field of special "under-water-concrete".

In this type of concrete, special additives and/or preparation produce a greater resistance against segregation during pouring (eg colloidal concrete).

The choice (type and dimensions) of the concrete block revetments completed so far depends only on experience factors and on personal judgement or preference. Objective design criteria have not been available. This means that in situations where little or no experience has been gained, ie for extreme loading conditions (super storms) the question can be put whether the design is in fact technically and economically the correct one (not too light nor too extremely heavy).

This guide-line hopes to form a contribution towards a more reliable method of design for a concrete block revetment. However, due to the complexities of the subject matter, there are as yet no simple computer models available to calculate the revetment stability under wave attack. Research for this purpose at the Hydraulics Laboratory and Soil Mechanics Laboratory (at Delft, Netherlands) will perhaps in the longer term produce a practically useful way to do it. In the meantime it is possible to use with advantage the results of tests carried out in a wave-flume as presented in Chapter 13. For the larger bank protection projects it might be useful to test the bank slope with revetment, together with the corresponding boundary conditions, to the largest possible scale in a wave flume.

2 REVETMENT REQUIREMENTS TO BE MET

2.1 Functional requirements

In accordance with the purpose of the revetment on a sea wall, ie to form a protection for the body of the embankment, the following functional requirements need to be stated:-

(a) The revetment must be able to resist:

- the combination of wave and flow attacks;
- the forces exerted by drift-ice, ice frozen to the revetment, etc.
- excess pore water pressures exerted from the bank due to a raised phreatic (ground water) level.

(b) The underlying soil particles must be retained; the revetment, together with any filter - construction present, must prevent migration of those soil particles. The revetment must also protect the filter construction against wash-out.

(c) The revetment has to be durable, ie it has to resist erosion due to materials being carried in the flows over it (sand, gravel, etc) and against frost and chemical action.

(d) In order to retain its purpose fully, it is important that the revetment can mould itself to possible form changes of the slope (settlement and/or scour) without destroying the bond of the revetment surface. When the sub soil settles locally, or is eroded, or disturbed by animal burrows, and the revetment cannot adjust to the new contours, hollows are created under the revetment, so that the construction is seriously weakened, for instance under an external loading of breaking wave impact. A strong interlock between concrete elements, preventing settlement of the elements into hollows underneath, can be a disadvantage in this context (see also para 7.1).

On the other hand, such strong interlock between elements on a sound foundation will provide greater stability under wave attack than can be provided by blocks which interlink only by means of friction

Flexibility and stability requirements under wave attack are thus seen to be contradictory. In practice the best possible compromise solution would have to be adopted.

(e) The whole of the revetment and foundation soil must be stable against slipping.

2.2 Requirements for technical execution

In order to achieve the optimum construction costs, the following requirements could be framed:-

(a) The revetment has to be quick and easy to place, preferably by mechanical means. For the construction of sea defence works only, a limited period of the year (April to October) is available (outside the storm season). For the zone of daily tides the requirement for rapid placement is even more obvious.

(b) The revetment has to be such that setting out and measurement can be carried out easily, especially for non-straight bank alignments.

There are, however, systems of revetment with unit shapes which require very accurate placing, because the tolerances are virtually nil. A typical example is illustrated in Fig 1. This system does not allow deviations from a straight line. Curved work in such cases can only be executed with in-situ connecting pours of fresh concrete between short lengths of straight alignments. This does not look particularly attractive, whilst the concrete quality as well as the behaviour under loads and settlements can be different.

By leaving joints "open" it is possible with many systems, within certain limits, to place units around a longitudinal curve. When the curves are reasonably easy, such open joints in a revetment on a granular

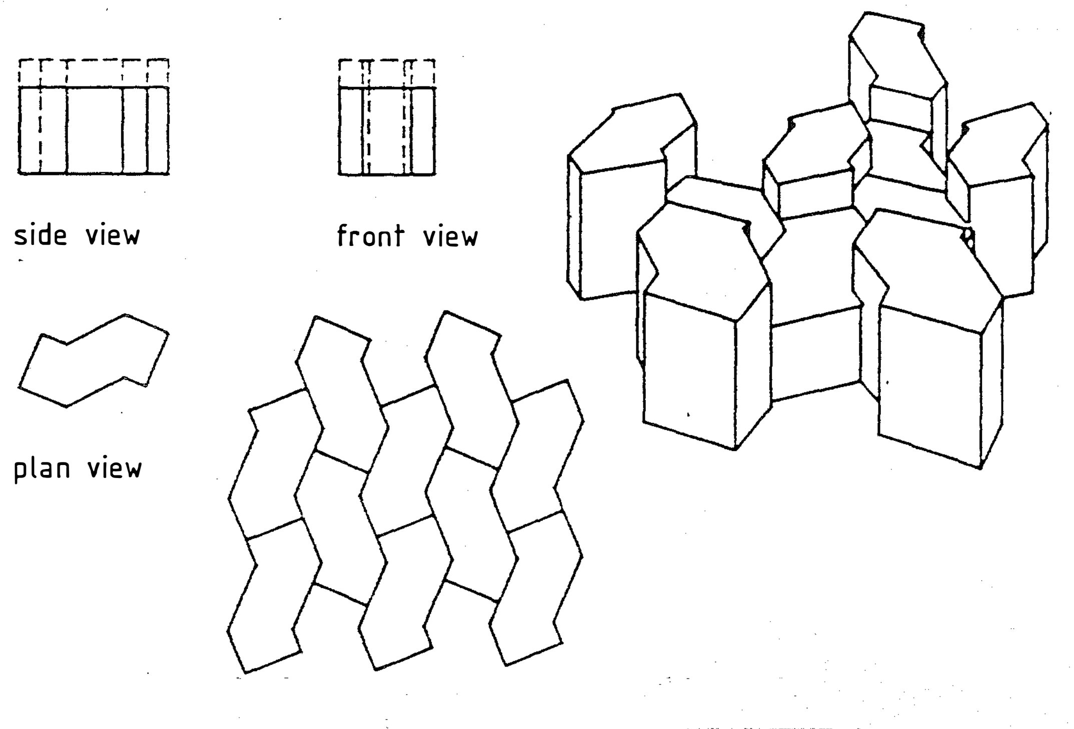
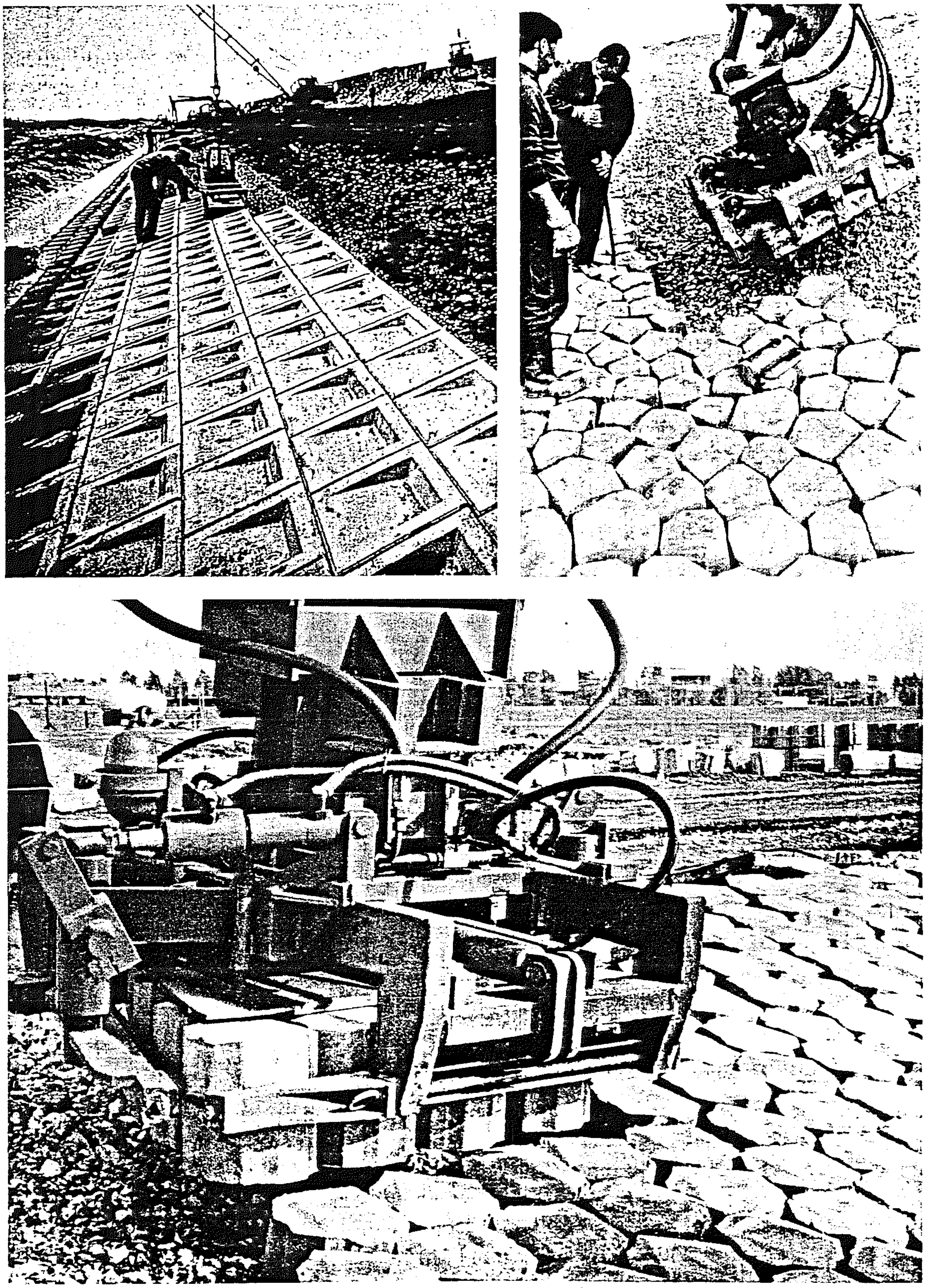


Fig. 1. Example of a system in which there is little freedom in dimensional co-ordination.



Mechanized placing of concrete units.

filter of sufficient coarseness need not give rise to problems. Clay as the under layer is less suitable in these situations.

Any "barrel" curvature, ie a curved slope in the vertical sense, needs to be small so that the movement and settlement of blocks is not obstructed, as that could reduce stability of the blocks.

2.3 Management and maintenance requirements

For a revetment to meet the requirement to provide a durable protection, the following demands need to be fulfilled:

(a) When unexpected damage occurs locally, it is important that the revetment can be repaired quickly and easily.

(b) The revetment must not be too easily damaged by vandals.

2.4 Special Requirements

Local circumstances could lead to one or more of the following requirements.

(a) Where an embankment is liable to be subjected to frequently occurring wave attack, it can be useful to reduce the wave-runup above the revetment by means of specially shaped blocks (with projections or voids).

(b) Temporary revetments should as much as possible be of block types which can be re-used elsewhere.

(c) Special requirements on the surface treatment of the blocks in order to fit better into areas of environmental importance.

(d) The revetment together with the foundation layer sometimes has to be watertight when embankments have to stand up to high water levels for longer periods.

3 REVETMENT TYPES

3.1 General

For the choice from various alternatives in a given situation, the judgement criteria to be formulated can follow the requirements outlined in Chapter 2. Because the various technical criteria are not all of equal importance, their significance will have to be weighted. Finally, the various alternatives will be checked against the judgement criteria and the weighting factors, thus producing a technical evaluation. For further considerations on this method of assessment, the reader is directed to the CUR-VB/COW report "Background to the guide on concrete block revetments" (Ref 32).

For the classification of concrete block revetments various aspects can be selected such as the following major division:

- According to block shape;

- According to porosity;

- According to the relationship between the porosity of the base layer and the intermediate layer if any;

- According to the combination of prefabricated concrete mats with artificial fibre carriers and/or cables.

The following could be named as Sub-divisions:

- Mechanical placing: yes or no;

- reinforced or mass concrete;

- prefabricated or cast-in-situ concrete;

- placement: only above water, or above and below water.

3.2 Main classification

3.2.1 Shape of concrete blocks

The elements can be sub-divided into:

- block and column shaped elements;

- plate or tile shaped elements;

- uninterrupted plate shape (continuous monolithic pour).

Block and column shaped elements

In the Netherlands these shapes are mainly used. The block shaped elements can be sub-divided in accordance with the given shape, the interlocking between blocks in relation to the possibility of comparative movement of one block to another, the locking action, the degree of wave run-up reduction provided, etc. Block shaped elements are usually made with gravel aggregate, but also with basalt as coarse aggregate to increase weight. For aesthetic or environmental reasons the blocks can be washed when freshly cast to show exposed aggregate, or sometimes provided with an additional basalt-aggregate surface layer for different aggregate exposure. The most common block element dimensions in plan are 0.50 m x 0.50 m or 0.25 m x 0.30 m, but 0.30 m x 0.30 m and 0.25 m x 0.25 m also occur. Matching thicknesses usually range between 0.15 m and 0.30 m. Blocks are commonly placed on the slope in a staggered vertical-joint pattern (see top left hand photograph, next page).

The column or polygon shaped elements can, depending on the producer, be regular or irregular in shape. These elements derive their strength as revetment material nowadays mainly by means of the joint filling material which has to take care of the interlock friction. The columns should therefore be made such that joint fillers can be applied.

In the past all manner of revetments with interlocking blocks were put on the market, with elements "hooking" together. Due to problems with dimensioning accuracy and automatic fabrication, this type is no longer in use*.

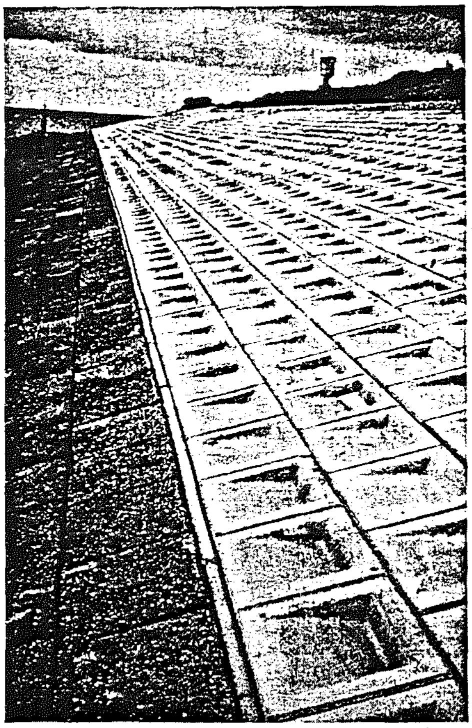
The height of column revetment blocks ranges in practice from 0.20 m to 0.50 m, and is also expressed in weight per m^2. Joint filler materials used are gravel, broken stone, rubble and non-hydraulic slag. Maximum dimensions of the individual joint filler granules should be in accordance with the average spaces between columns. A spread in grain sizes of the joint fillers produces better results against waves washing the joints clean.

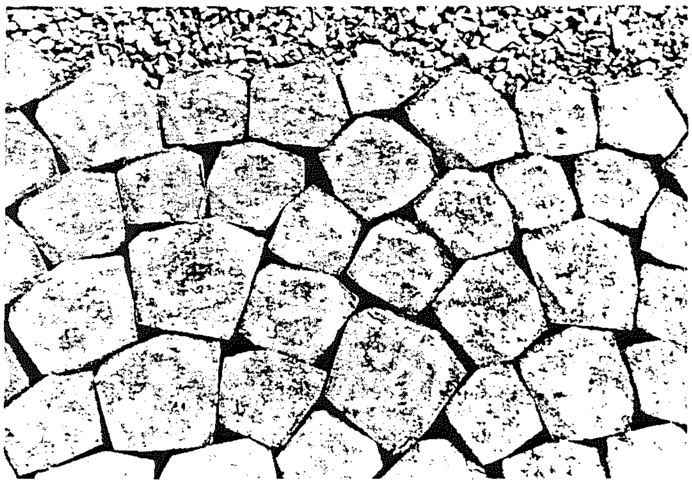
Plate shaped elements

Solid plate shaped elements are rarely if ever used in the Netherlands. However, slab shaped blocks with holes for vegetative growth are used on a large scale. In addition to the holes there are also shallow grooves in the upper surfaces, linking the holes. If

Translator's note:

* This comment presumably refers to the Netherlands.





Some types of revetment.

all holes and grooves are filled with clay** and all clay is thoroughly rooted with grass a strong revetment could result. The slabs are usually 0.40 m x 0.40 m or 0.40 m x 0.60 m in plan dimensions with thickness ranging from 0.09 to and including 0.15 m. This type of revetment can be placed by hand as well as mechanically, and is commonly placed well above the influence of the daily tides.

Continuous concrete slab

The continuous concrete slab is rarely if ever used in the Netherlands because of the danger of uneven settlement and soil erosion. It is still used on a large scale elsewhere.

3.2.2 Permeability of concrete revetment

The revetment can be:

- closed or nearly closed;

- open.

The permeability of tightly fitting blocks can be regarded as very small in comparison with the permeability of the under-layer. To make such revetments more pervious, the block shaped elements can be supplied with chamfered corners and/or notches in the sides of the blocks. That will in addition permit filling with joint material to provide additional interlocking action.

Permeability of column shaped elements with joint filler materials (granular) is large in comparison with the underlayer, and the revetment is classified as very open. If that type of revetment is penetrated with a hot poured bitumen, a completely closed revetment will result.

3.2.3 Relationship of concrete block revetment with the permeability of the foundation layer and the intermediate layer

How the hydraulic loadings act on the revetment cannot be seen in isolation from the mutual relationship of

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** Clay is used in the Dutch text and it may be Dutch practice, although it may not be the best soil for growing grass (note from translator).

the permeabilities of the foundation, the intermediate layer and the revetment (see also Chapter 13). The selection of the concrete block revetment is accordingly determined largely by that relationship.

The following differences can be noted (see figs 2 to 9):

- open revetment on permeable intermediate layer and impermeable foundation;

- open revetment on permeable intermediate layer and permeable foundation;

- open revetment without intermediate layer on permeable foundation;

- closely-placed revetment with or without permeable intermediate layer on permeable foundation;

- - ditto - on permeable intermediate layer with impermeable foundation;

- - ditto - without intermediate layer on impermeable foundation;

- closed revetment on impermeable intermediate layer with impermeable foundation;

- grassed revetment on impermeable foundation.

Figures 2 to 9 offer only some examples of possible types of construction, without attempting to be exhaustive; they should therefore not be interpreted as standard constructions.

Because blocks laid direct on an impermeable foundation (clay) are more stable under wave-attack, this type of construction can be preferred. It is, however, not always possible or desirable to apply that construction.

At the lower levels in the tidal zone the erosion resistance of clay and the construction operation for a placed type of revetment are uncertain, whilst in addition in cases of possible pore-water pressures in the body of the bank a permeable foundation layer is essential. The necessary requirements for clay are described in paragraphs 5.2 and 5.4.1.

Where in the Figs 2 to 9 reference is made to geotextile <u>in brackets</u>, it may also be omitted from the construction. In connection with site construction work the geotextile can provide a cover to make placement of the subsequent layer easier; the geotextile forms in that case a protection role - a separator against accidental mixing of materials.

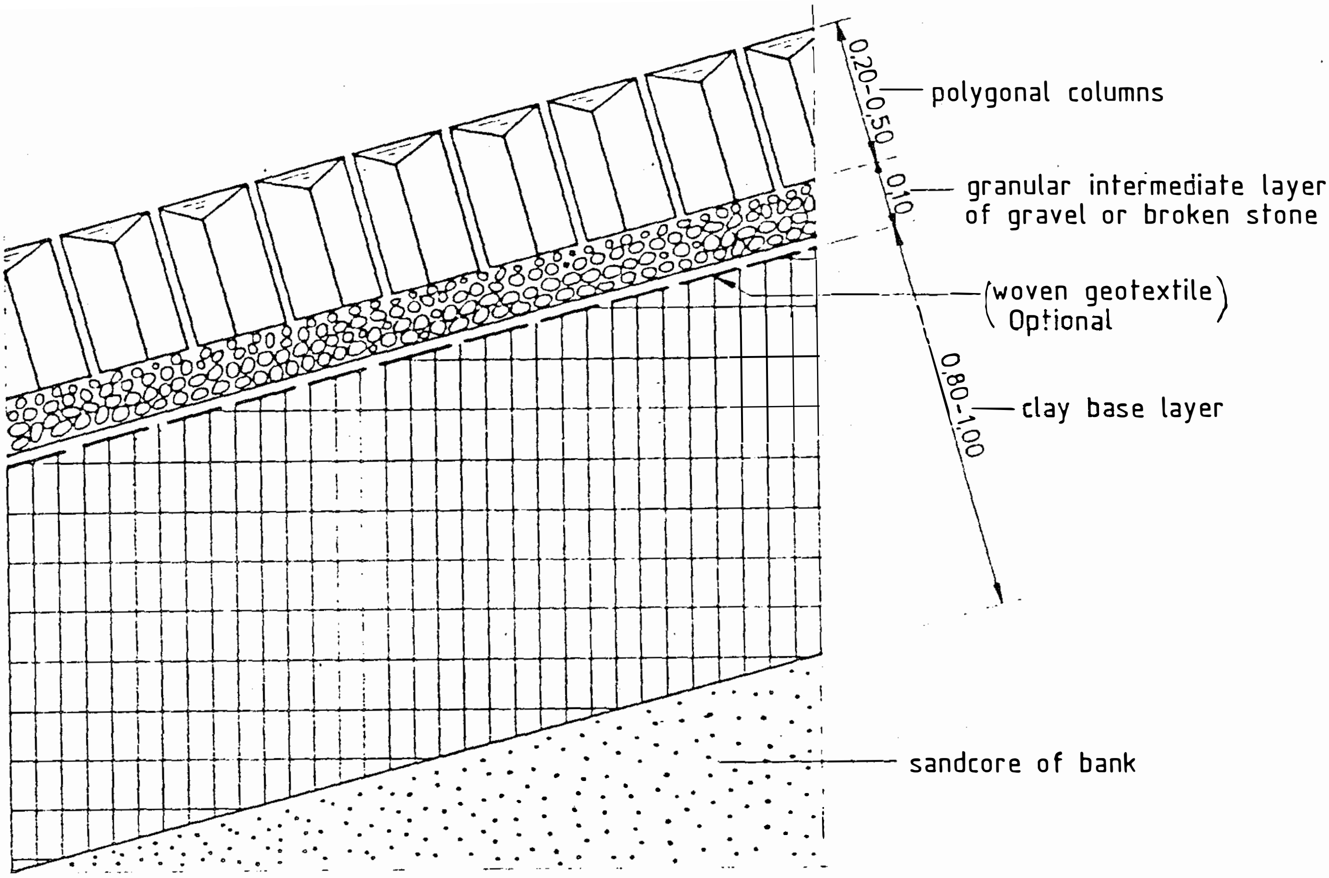

Fig. 2. Open revetment on permeable intermediate layer with impermeable under-layer.

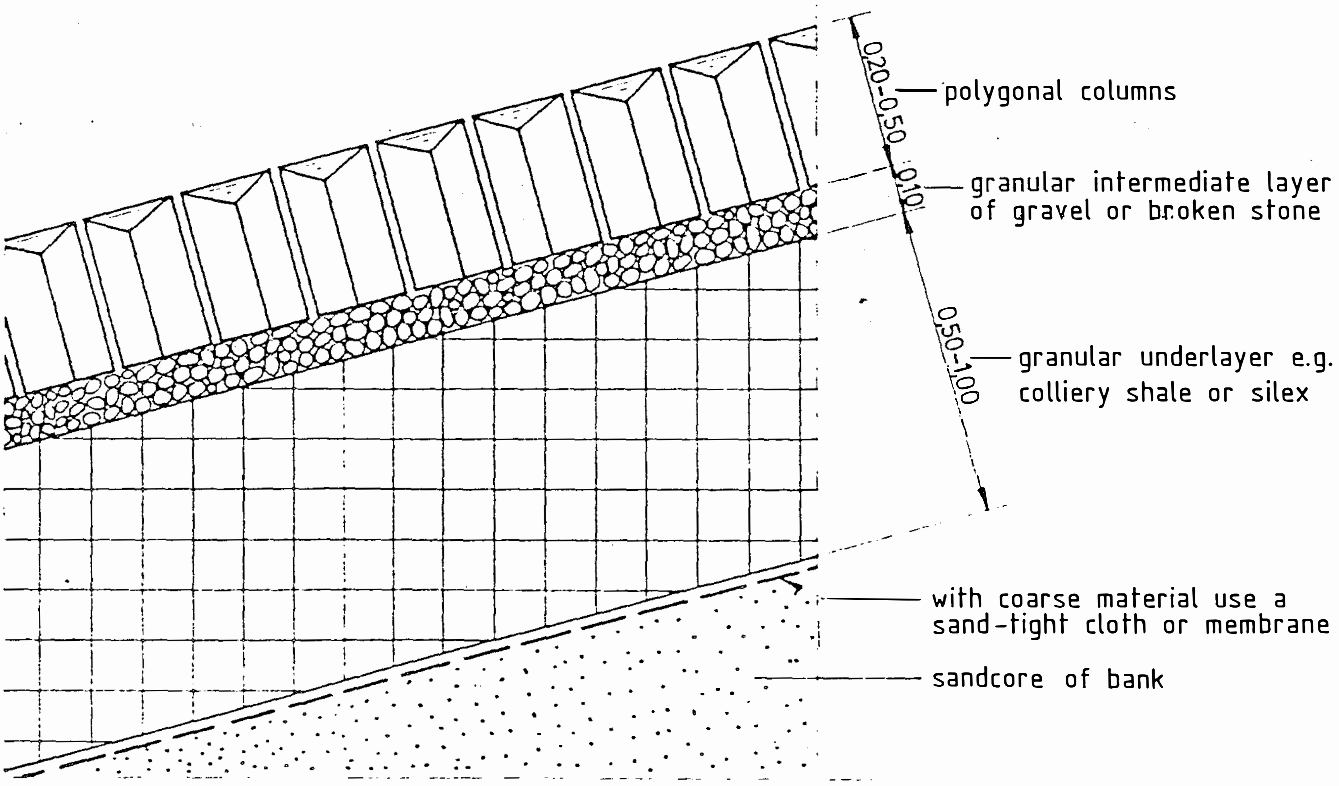

Fig. 3. Open revetment on permeable intermediate layer with permeable under-layer.

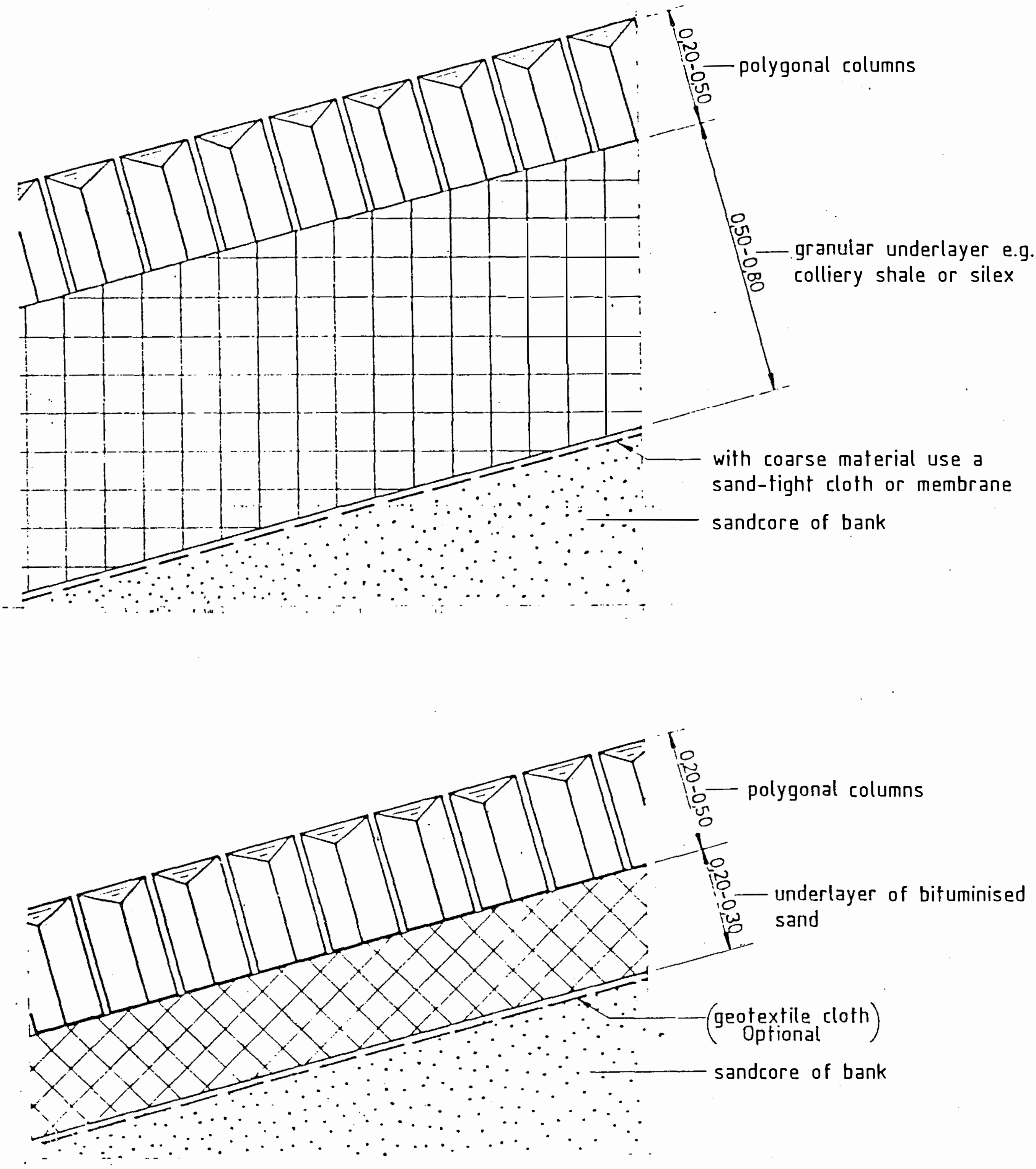

Fig. 4. Open revetment without intermediate layer on permeable under-layer.

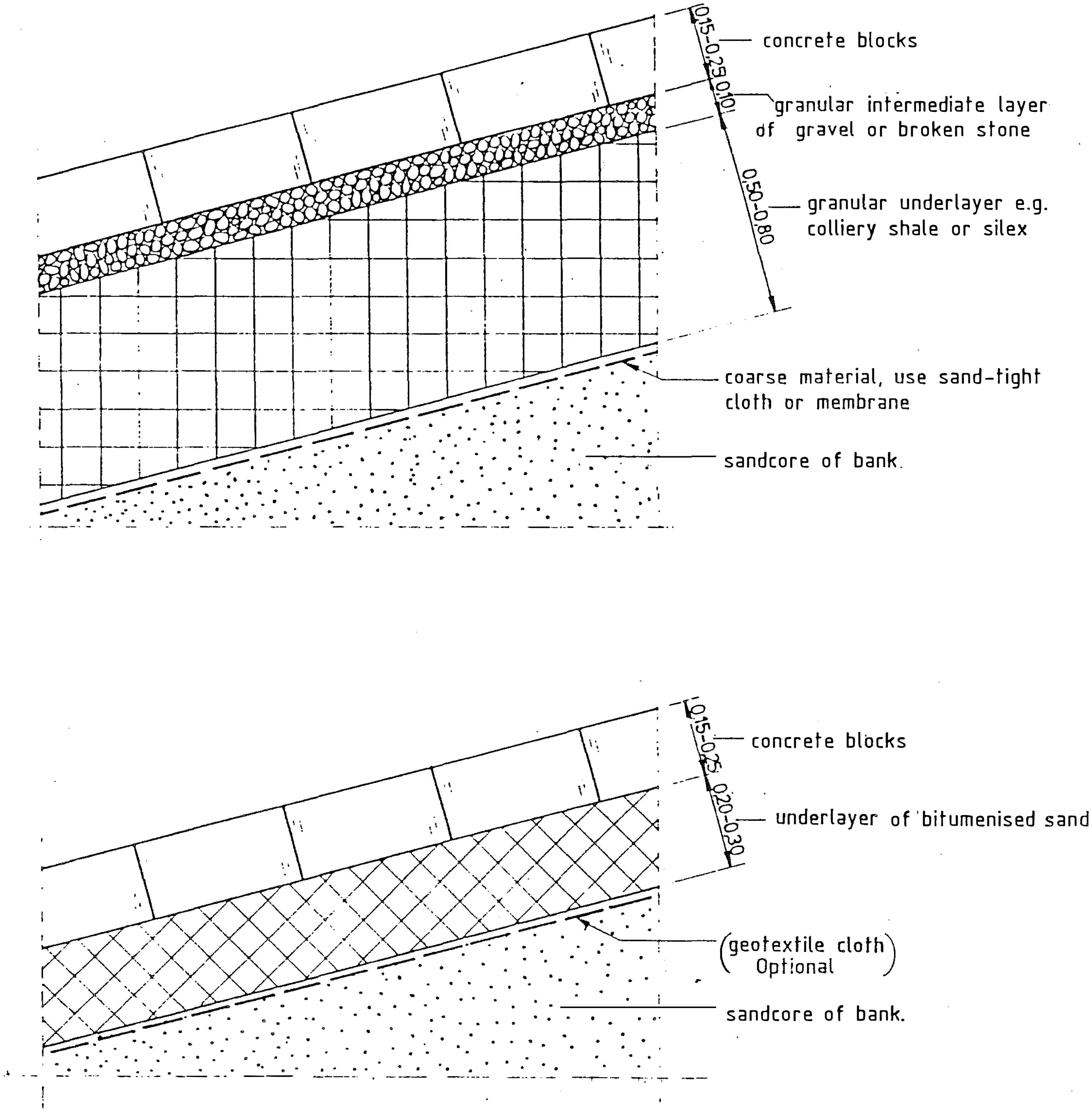

Fig. 5. Continuous revetment with or without permeable intermediate layer on permeable under-layer.

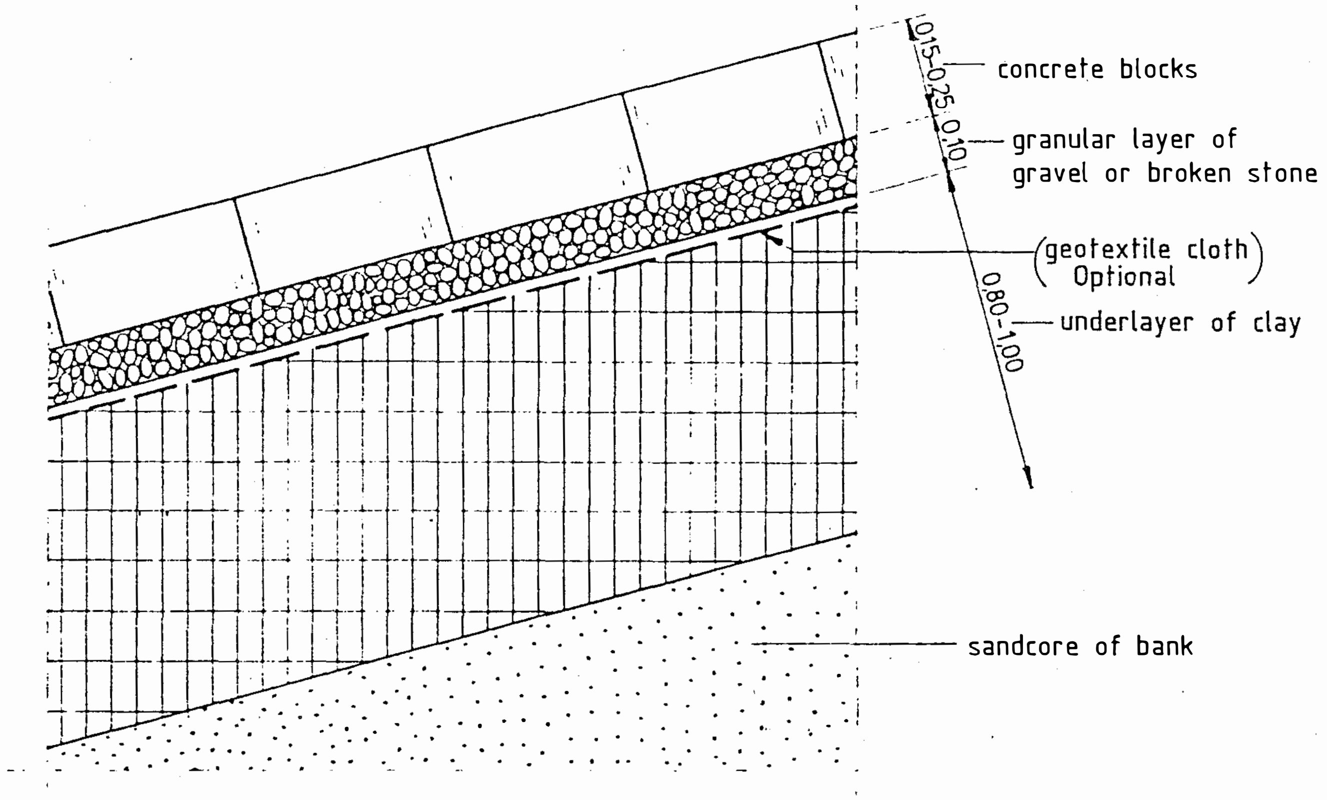

Fig. 6. Continuous revetment on permeable intermediate layer with impermeable under-layer.

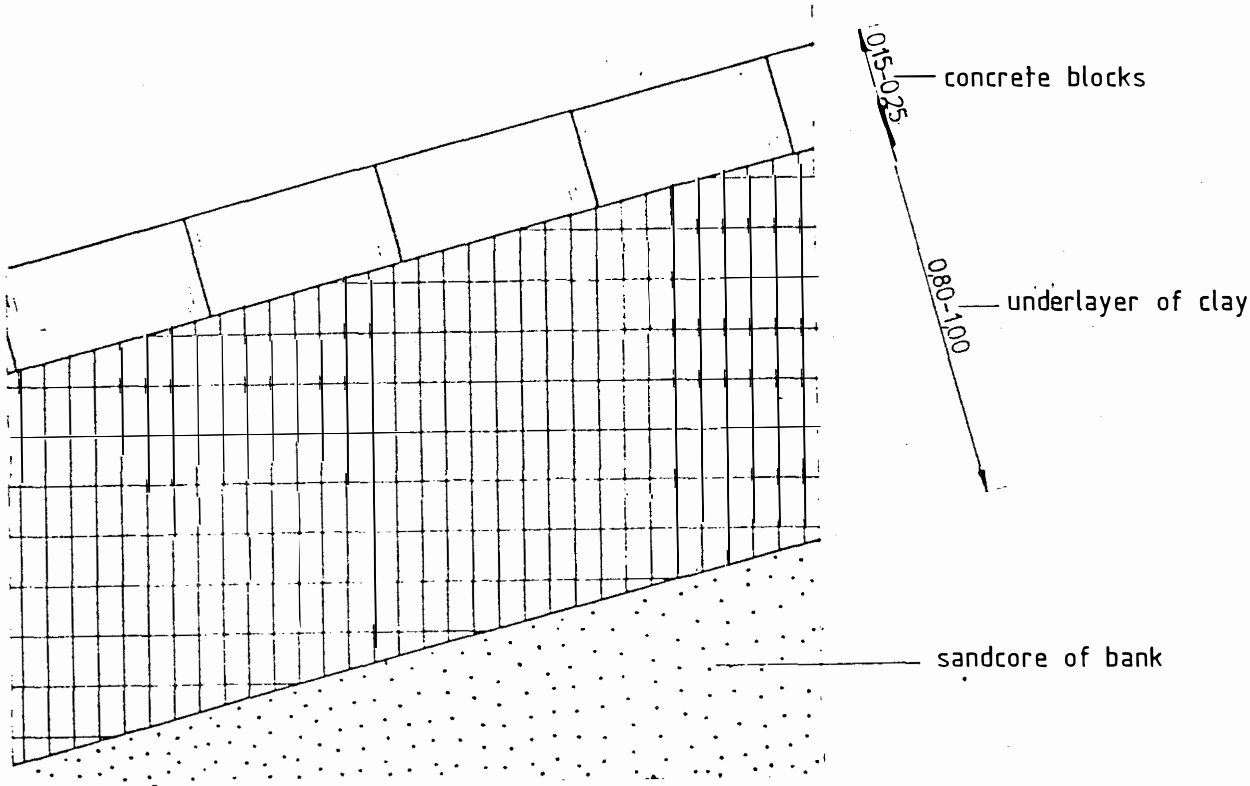

Fig. 7. Continuous revetment without intermediate layer on impermeable under-layer.

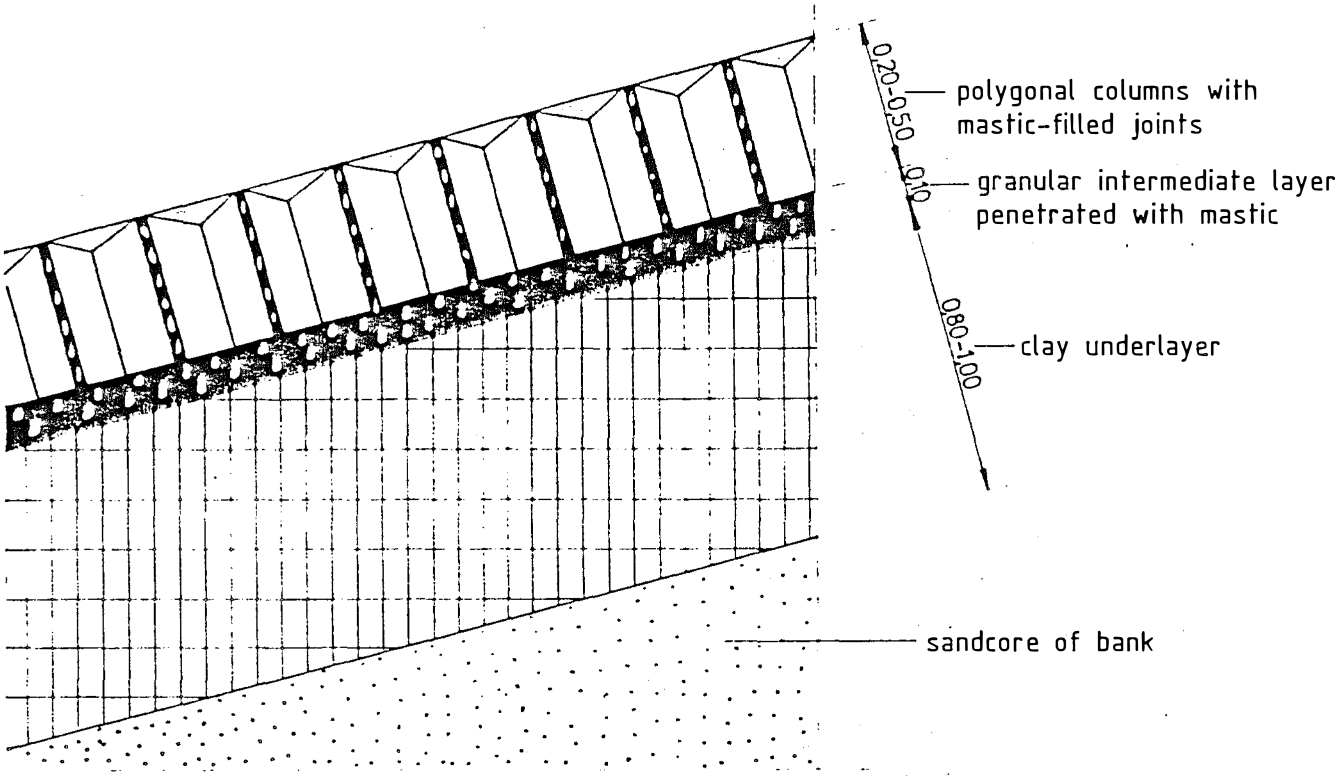

Fig. 8. Closed revetment on impermeable intermediate layer with impermeable under-layer.

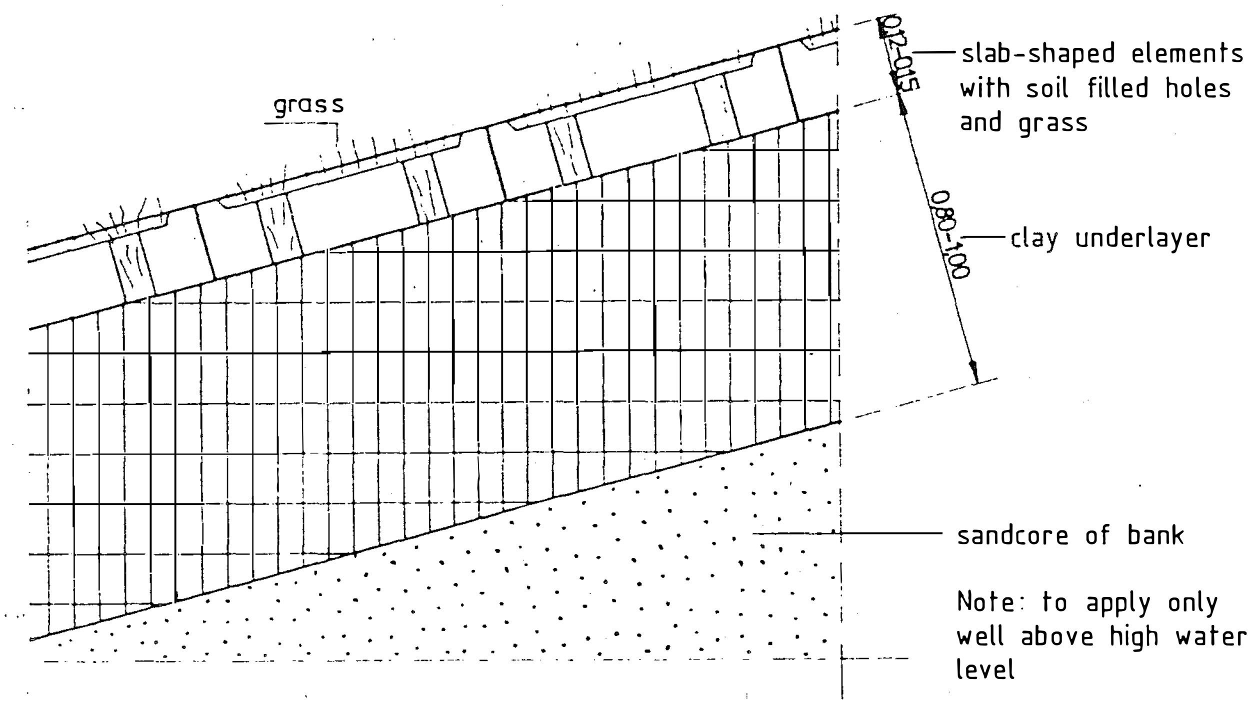

Fig. 9. Grassed revetment on impermeable under-layer.

3.2.4 Prefabricted mats of concrete blocks on geotextile carriers

A new development in the protection of river and lake banks comprises the application of factory-made interlocking concrete blocks on a carrier of geotextile fabric and sometimes provided with cables through the blocks.

Application so far is mostly on the banks of canals, rivers and lakes. These mats are permeable and, therefore, have to be counted as open revetment.

3.3 Sub-division

3.3.1 Mechanical construction: yes or no

There is increasing preference for the use of systems which permit mechanical construction methods. One result of this is the development of the prefabricated block mats described in 3.2.4 above.

Originally only the plate shaped elements could be placed mechanically. Now also column shaped elements can be placed mechanically, so that it can be said that in Holland most of the currently used revetments can be placed mechanically.

3.3.2 Reinforced or mass concrete

All normal types of revetment are manufactured in mass concrete. Concrete elements for special revetments construction are the only types to be made in reinforced concrete in the Netherlands.

3.3.3 Concrete placed in-situ or on site manufactured blocks

Prefabricated concrete blocks should have preference over in-situ placed concrete because the former ensures in general a better quality concrete. Even made-to-measure gap-fill blocks should preferably be prefabricated. In general the use of in-situ concrete even for filling joints and narrow gaps should be avoided as much as possible.

3.3.4 Construction method: only above water or both above and below water

Only the prefabricated block mats can be placed under water; all other block or column revetments are limited to above water application; however, this means that no lower edge fixing construction can be incorporated.

Another new development consists of colloidal concrete for pouring under water.

4 CONCRETE TECHNOLOGY

4.1 General

During its life in use as bank revetment, concrete quality can deteriorate. The causes for this can be mechanical, biological, physical or chemical.

Mechanical attack on concrete can be due to excessive loading, or in the case of revetment mainly through the scouring action of sand and water. Biological effects arise in the form of growth of algae, (water) plants and other organisms. These growths may adversely affect the possibility of walking over the revetment, and the aesthetics, but do not (in general) damage the concrete skin. Physical causes of damage are for example large temperature changes and frost. Chemical attack can occur through any contact with water through the chemicals in solution, or purified water.

Corrosion of steel if present in the concrete can be destructive and should be prevented, just as any other form of attack should be guarded against.

In the choice of the raw materials for the construction elements, important considerations are the workability of the concrete and the economics.

4.2 Characteristics of wet concrete-mixes and concrete

The degree of compaction of a concrete-mix depends on many factors, such as type and quantity of cement, particle shape and distribution of the aggregates and especially the amount of water with possible additives in the mix.

Water-separation or "bleeding" of a concrete mix can be counteracted by using less water, including additives in the mix and by increasing the "specific surface" of the aggregates (ie total surface of all particles).

Strength of a concrete element of a construction is most effectively tested by a destructive test. It is possible to use laboratory tests instead. Standard laboratory tests exist for the determination of erosion resistance, modulus of elasticity, shrinkage, creep and other characteristics of concrete materials. By choosing heavy aggregates the volume-mass of concrete can be increased, which has a favourable influence on the stability against wave-attack (see Chapter 13).

4.3 Requirements, standards and specifications

The manufacture and application of concrete is tied to a number of standards (see literature list: Dutch standards). Unless otherwise specified, the concrete as well as the products made of it, have to comply with the requirements prescribed in the standards.

Concrete used for bank revetments has to be at least of quality defined as B30 in the (Dutch) concrete standards*, in order to limit the damage potential of mechanical, physical or chemical attacks. When strength during transport is important, greater strengths may be required.

For better resistance against chemical attack, the cement has to have greater than the normal sulphate resistance and a low content of free lime. The type of cement which meets these requirements is blast-furnace cement, which therefore is desirable for revetment use.

The concrete aggregates used to have to comply with the pertinent product standards.

If reinforcement is used in the concrete, the cover required under the concrete standards for "aggressive environment" (sea bank revetment!) should be strictly adhered to.

Careful compaction and finishing treatment is of importance to achieve a dense water-tight surface. The concrete should be protected for a period of at least one week against drying out. Control of the composition of the concrete, mixing, finishing and strength is required.

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* Translator's note: Concrete quality B30 = 30 N/mm^2 after 28 days

Poor and good concrete side by side.

Severe erosion due to the scouring action of granular material and water.

4.4 Further Explanation

The foregoing requirements and standards have been formulated in the light of experience with the use of concrete in water construction works. Exaggerated requirements for the strength of concrete materials can result in unnecessarily high costs.

Building materials in an aggressive environment are under attack, sometimes slowly, occasionally quickly. The rate of progress of such attack depends on the material's resistance. In that connection the composition and pore-distribution is of course important. Demands are therefore made on the concrete's raw materials, fabrication, storage and quality. Depending on the type of cement the concrete can suffer more or less from the chemical attack of calcium and magnesium sulphates present in sea water.

A fortunate circumstance is that sulphate attack in sea water is much less than that shown by comparative tests in a laboratory with the same sulphate concentration. This is due to the presence of chlorides in sea water. Practice has shown that attack on concrete in sea water is mostly small, provided the concrete is dense and of good quality. In Holland the specifications for concrete in coastal areas nearly always insist on the use of blast-furnace cement. When the blast-furnace slag content is at least 65%, this cement is sulphate-resistant.

Frost forms another type of attack. In CUR-VB report 64 "Frost-resistant Concrete" (Ref 10) test results are given which show that blast-furnace cement provides better frost resistance than does Portland cement.

Finally there is the possibility of mechanical damage to concrete. This will be mainly caused by abrasion due to flow of water carrying sand and/or gravel, and bursting action when the salts in the water form crystals within the concrete.

Temperature differences can cause tensions in the surface layers, causing cracking.
The above discussion shows clearly that concrete constructions have a limited life, but that the life can be strongly influenced by the concrete quality. Standards provide clear guidance on the use of reinforcement.

Corrosion to steel reinforcement due to penetration of carbonic acid from the air and chloride from sea-water can be reduced with dense, good-quality concrete.

In addition the concrete cover has to be adequate, and crack-formation should be prevented as far as possible. It should, however, be noted that steel reinforcement in concrete for sea-wall revetment is rarely considered necessary.

5 REQUIREMENTS TO BE MET BY UNDER-LAYER

5.1 General

Each under layer or foundation together with a possible intermediate layer can be defined as the transition construction between the usual sand core of the embankment to the outside revetment cover. A foundation with an intermediate layer has to fulfill the following functions depending on the circumstances:-

- prevention of soil migration (washing out) from the bank core;

- forming a smooth slope face for ease of placing the revetment (particularly applicable to the intermediate layer);

- to form an extra safety margin in case of revetment damage. So that any consequent erosion hole does not immediately reach the sand core of the bank;

- forming a watertight layer on the permeable bank-core;

- forming a good drainage layer immediately under the revetment (this applies to the intermediate layer);

- forming a temporary protective cover against flow and wave attack during construction of the bank;

- to function as a temporary low bank to retain the hydraulic (sand) fill for the main bank-core, and to be retained afterwards to form a permanent part of the bank*.

Underlayers can be divided into two main groups ie permeable and watertight.

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* Translator's note: typical Dutch construction of a new embankment along tidal waters.

5.2 Watertight foundation layer

A watertight layer is generally produced by using clay. If, however, an intermediate layer of granular material is present, possibly with a geotextile cloth to prevent the granular material being trodden into the clay, then the underlayer in relation to the revetment is a permeable layer.

Concrete blocks laid directly onto the clay in the zone above the tidal levels need not present any problems, provided the clay is of good quality and the revetment is a reasonably tightly fitting design. However, blocks directly placed on clay in the daily tidal zone can present problems with construction as well as leading to erosion.

When the blocks fit snugly to the clay slope, it would be difficult to create high water pressures under the blocks. Therefore, the stability of such blocks under wave attack will be greater than for the same blocks on a granular (filter) layer. This does require an optimum fit of the blocks on the clay; the following points have to be considered:

- erosion of the clay under the revetment;

- hollows in the clay under the blocks;

- plastic and viscous characteristics of clay;

- shrinkage and swelling behaviour;

- clay workability, particularly with reference to compaction for construction.

The detailed requirements for clay to be used are dealt with in paragraph 5.4.1. Here the guide lines are limited to some information for the construction using clay as derived from reference 20. It should be realised that (constantly) changing work conditions make it difficult in practice to attain the optimum construction condition of the clay. In order to produce a good under (foundation) layer, it is recommended that an attempt should be made to keep to the working limits set out below, if at all possible.

The most important aspects for the placing of clay are:-

- bringing on site;

- compacting;

- producing a smooth slope and placing of blocks.

Bringing clay on site

The optimum workability conditions, expressed in the moisture content, lie generally between comparatively narrow limits. With some types of clay these limits can, therefore, be exceeded swiftly when the moisture content changes as a result of weather changes. In this context it is important that the plasticity index* Ip of the clay is not too low in order to eliminate the possibility of a swift change from the semi-solid to the semi-liquid state during a period of rain (Ip > 20%). A good criterion for the workability of clay is the consistency index* I_c(I_c > 0.8). This requirement provides the necessary difference between plastic and liquid limit.

Compaction of clay

During construction, efforts must be made to obtain the densest possible soil structure (compactions). In order to obtain optimum density on site, the ideal limitations for moisture content are:-

- at least equal or only a little below the optimum moisture content according to the Proctor test. (Rijkswaterstaat standards 1978);

- at maximum equal to or only a little higher than the plastic limit.

The density attained should be at least equal to about 95% Proctor density. Compaction has to produce a clay layer as homogenous as possible. Layer thickness for

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* Consistency limits are expressed in the moisture content (ie the mass of water divided by the mass of dry matter x 100%), and split into the following:-
  - liquid limit W_l:
    the moisture content at which a groove made in a soil sample is just filled again after the dish containing the sample has been dropped 25 times over a height of 10 mm onto a solid surface;
  - plastic limit W_p:
    the moisture content at which it is just no longer possible to roll out a ball of clay to a 3 mm thick thread without crumbling;
  - shrinkage limit W_s:
    the moisture content at which the sample when drying out no longer reduces in volume.

(footnote continued on page 17)

compaction should not be too great, if necessary it can be done in 2 layers of maximum 0.40 m thickness. During and after compaction it is necessary to carry out tests on density and consistency. There should be no large lumps of clay present which would not meet the homogeneous fill requirement.

Smoothing the slope and placing of blocks

Care should be taken during surface smoothing of the slope that the top 20 mm or so are not loosened too much, as that can lead, under the heavy loads to be expected after completion, to erosion or damage. It is equally unacceptable to fill deep vehicle tracks etc with loose, crumbly clay. Vehicular or even pedestrian traffic on the finished clay slope face should be avoided.

Furthermore it is important to prevent local contamination with porous materials, because contaminations could produce local spots where high pore pressures would occur so that blocks could be lifted or erosion take place.

It is recommended that the clay layer should be placed such that after compaction the whole surface is a couple of centimetres proud of the design profile. The surplus clay can then be trimmed off to produce a flat, smooth and dense surface, which should not show any cracks.

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liquid condition		
	liquid limit	
plastic condition		
	plastic limit	consistency limits
solid condition		
	shrinkage limit	

The derived indexes are:
plasticity index: $I_p = W_l - W_p$

Consistency index I_c $\dfrac{W_l - W}{I_p} = \dfrac{W_l - W}{W_l - W_p}$

in which W = water (moisture content) of the soil being worked.

In order to improve the contact between blocks and the clay surface, it is recommended to press the blocks down with a roller.

Work can continue when it rains until the clay is weakened to the point where placing and compaction requirements can no longer be met.

As the clay surface should not be allowed to dry out, the concrete blocks should in general be placed on the same day that the slope is trimmed to final profile.

No blocks should be placed during frosty weather, nor when the clay is still frozen.

Where a sand-core embankment has been constructed, with a clay layer for revetment foundation, the clay layer should, as with all other waterproof covers (eg asphalt-type revetment), not be too thin so that excess pore pressures in the sand-core cannot lift the layers. In this context the landward slope of the embankment should not be forgotten.

Another reason to maintain a sufficiently thick clay layer is the function described in paragraph 5.1, ie to form a safety zone when the revetment is damaged. In general (in the Netherlands) the clay thickness for sea walls is taken at 0.8 to 1.0 m.

Use is occasionally made of a paving of gravel or sand on the clay in the tidal zone in order to protect the clay and make block placing easier. The method does reduce the advantage of the more stable block position on clay instead of on granular filter. This method is not recommended.

When high level saltings in front of the embankment make it necessary to extend the clay underlayer deep down, a better method would consist of a temporary small bank to keep the lower slope of the embankment dry. When, however, the mud and ground levels in front of the embankment are low, the underlayer in the tidal zone will have to be made in erosion resistant material, eg colliery shale. In those situations the clay underlayer can only be provided above the daily tide zone.

5.3 Permeable under layers

One of the functions which a permeable underlayer has is to provide protection of the embankment core against wash-out and it, therefore, has to satisfy the normal filter design criteria against migration of sand, (see para 5.5).

Permeable underlayers can be divided into:-

- underlayers of granular materials, possibly combined with a geotextile cloth, or with a layer of clay underneath;

- underlayers of bonded granular materials.

5.3.1 Under layers of granular materials

In order to satisfy the sand-tightness requirements the granular filter has to be constructed correctly. This filter can comprise one layer with a nearly homogeneous particle distribution, or comprise more layers with the upper layers having increasing particle sizes. The bottom layer is designed to stop particle migration from the foundation layer, and each succeeding layer has to do the same for the particles in the layer below. For a graded filter the construction will have to be a responsible compromise between technical requirements and economic possibilites. It is in practice often easier and cheaper to use a geotextile cloth to achieve a sand-tight filter.

The revetment itself has to satisfy the filter principle concerning material-tightness. The dimensions of the particles on the upper side of the underlayer filter have to be large enough to prevent that material being eroded through the openings (joint and holes) in the revetment. However, the application of too coarse particles in the upper layers of the filters is also wrong, because it then becomes more difficult to get a smooth block-slope and the block placers will try to use large pieces of material to get the slope looking right. This could lead to uneven settlements and lack of a smooth surface.

Other demands to be satisfied by filters under a revetment concern the permeability. Excess pore water pressures are in themselves acceptable as long as they are compensated by extra weight, so that neither the filter nor the revetment can be lifted, nor that those pressures can lead to sub-soil weakening resulting in reduced shear-strength and possible soil-slips.

As stated before, where a revetment is constructed of separate unconnected elements, stability is favourably influenced when the blocks are placed directly on a less pervious material (see Chapter 13).

Various materials can be used for filter construction, eg various types of slag, crushed gravel, broken stone, silex, colliery shale, and brick-rubble. If slag is used, care should be taken to ensure that it does not contain water-soluble pollutants, which could be environmentally harmful.

The other function (see 5.1) of an underlayer to act as an in-built extra safety margin against damage done to the revetment can be realised by:

- placing a clay layer under the granular layer;

- make the underlayer sufficiently thick;

- use material with a high volumetric mass;

- make the stone dimensions in the underlayer sufficiently large.

When the base layer for any part of a revetment consists of colliery shale from a temporary bank, it is desirable to cover the shale with an intermediate layer of finer granular material, eg crushed stone or crushed gravel to a thickness of 0.05 to 0.10 m, for two reasons:-

- when blocks are laid directly on shale, the finer particles will wash out so that hollows and/or settlement can occur;

- the unequal sizes of shale elements make it difficult to get a smooth slope.

Placing a block revetment directly on a geotextile cloth which in turn lies directly on the sand core of the bank cannot be recommended. The geotextile forms no suitable second line of defence in case of damage to the revetment. Also the geotextile is easily damaged during the placing of blocks.

It is a possibility to combine a geotextile filter with a granular layer in one construction. This has the advantage that no difficult filter requirements need to be met by the granular material.

The functional requirement for stability against sliding, mentioned in Chaper 2, needs to be observed by taking into account the particle shape of the granules. In order to get sufficient stability (especially during construction) the particles need to be angular, depending also on the degree of slope. In general the angular shape will be satisfied, when broken materials are employed, hence when gravel is used it should be crushed gravel.

5.3.2 Under layers of bonded granular materials

For a bonded filter under the revetment both cement and bitumen can be used as the bonding agent. With a correct selection of the mixture both the cement and bitumen can be used to prepare an underlayer with a permeability which approaches that of cohesionless sand.

Cement-bonded underlayers have the disadvantage that they cannot follow uneven settlements and erosion holes without cracking. Bitumen-bonded underlayers are less sensitive to differential settlements due to the viscous characteristics of bitumen. This assumes, however, that settlements develop gradually. The percentage of bitumen used will materially influence the plasticity of the bonded layer.

When a bonded underlayer is used, the revetment can be placed directly on it.

This type of underlayer makes for a less stable block revetment than that provided by a good clay base.

5.4 Characteristics of materials

5.4.1 Clay

For the use of clay as a layer directly under a concrete block revetment, the erosion phenomenon is important in the context of block stability under wave attack.

The following parameters influence the erosion behaviour of clay:-

- particle dimensions, ie clay, silt and sand content;
- the clay-mineral;
- the organic matter content;
- consistency limits and the derived indices;
- manner of sedimentation and consolidation;
- the density, the (optimum) water content and the degree of compaction;
- permeability;
- shrinkage and swelling behaviour;
- cohesion and shear resistance;
- physical-chemical characteristics of clay, pore-water and eroding water;
- homogeneity;

Although it is possible to give indications to the degree of erosion sensitivity as a function of a number of these parameters separately, there is no possibility of expressing them in an erosion formula. The description of the influence of the various parameters can in general only be qualitative.

In order to determine resistance against erosion experimentally, various types of erosion tests are possible. A distinction can be made between those tests which determine erosion-resistance along standardised procedures, taking no direct account of the circumstances in practice, and those where the relationship with the practical situation does exist. For the choice of a suitable test method it is recommended that an experienced soil mechanics laboratory should be consulted. Although the erosion resistance of a specific clay cannot be easily calculated, cetain general limitations can be presented within which the stability against erosion is more certain (see also Ref 20):-

- minimum about 20% clay ($d < 0.002$ mm);
- maximum about 40 to 50% clay;
- maximum about 25% sand ($d > 0.063$ mm);
- maximum about 3% organic matter.

When clay is taken from saltings the degree of "ripeness" has to be taken into account. When a submerged soil emerges above water surface, it begins to lose water through evaporation; direct at first, afterwards through shrinkage cracks and vegetation. This drying - out process is termed "ripening". Through reduction in pore water pressures the effective stresses increase, thus making the clay more easy to work. If any saltings clay is to be used, it should be sufficiently ripe, and this process can be speeded up by placing it on a stock-pile and turning the pile several times.

Ripening-factor can be calculated from:-

$$n = \frac{W - p(100 - L - H)}{L + b\,H}$$

in which

b = ratio of water binding power of a specific mass of organic matter to that of the same mass of clay fraction (b = appr 3.0)

p = grams moisture bound by 1 gr non-colloidal matter (dry clay minus clay fraction and organic matter) (p = approx 0.3)

L = grams clay fraction per 100 gr dry matter

H = grams organic matter per 100 gr dry matter

W = grams of water per 100 gr dry matter

Apart from the water content, the percentage clay fraction and organic matter influence the rate at which the clay "ripens" with time. The concept of the "ripening-factor" (n-number) is used. It is defined as the water content in grammes which is bound by 1 gram of the clay fraction. A completely ripened clay has a "ripening-factor" n less than 0.5.

5.4.2 Colliery shale

Colliery shale is a waste product from coal mining (see also Ref 19). During the formation of coal under high pressure, sand and clay formed sandstone and clay shale. Colliery shale comprises mostly clay shale (this obviously applies to Holland). Depending on the degree of consolidation the clay shale can be divided into clay-stone and slate stone. Most of the clay shale comprises the softer clay-stone, which when exposed to the air generally disintegrates easily to smaller fragments. This process can seriously reduce the permeability with lapse of time. The rate and degree of such reduction depends on the degree of consolidation. The material does not reduce the particles smaller than the 2 mm sieve dimension. The reason for this is that the high pressure which changed clay to clay-stone removed the thin water-coating from the clay particles, and with that also removed the clay characteristics, such as plasticity, shrinkage and swelling potential. Damp colliery shale has no resistance against alternate frost and thaw cycles.

During the determination of the particle distribution account has to be taken of a certain amount of disintegration of the colliery shale as a result of transport, stock-piling and placing.

Consolidation possibilites of colliery shale are heavily dependent on the moisture content and the grain size. When the moisture content of unsorted colliery shale from stock-piles increases by a few percent through precipitation, there is a real possibility that the optimum moisture content will be exceeded, which may lead to a state at which it cannot be consolidated nor used for traffic. Water logged colliery shale is in principle more sensitive to settlement flows than sand. In the USA some colliery shale dams have collapsed by that process.

When during the operations with colliery shale the material is not too badly remoulded, the permeability will be comparable to the values for a very coarse sand (without silt), even after fragmentation of the shale. Through compaction and allowing traffic during wet weather, the resultant remoulding can cause serious deterioration in the permeability. If the compaction is carried out with a bulldozer, which does not cause such remoulding damage then permeability is not seriously affected. Compaction is essential in order to prevent differential settlement.

In the bank building industry colliery shale is commonly (Holland) delivered in two gradations, ie 0-70 mm and 10-125 mm. The volumetric mass of the particles is about 2,200 to 2,500 kg/m^3. The volumetric mass of the material dumped loose is 1,700 to 1,800 kg/m^3.

5.4.3 Silex

Silex, also known (Holland) as "firestone", is a very hard rock, consisting almost entirely of a micro crystalline form of SiO_2 The volumetric mass is about 2,600 kg/m^3. The volumetric mass of the loose-dumped material depends of course on the particle grading, and is about 1,600 kg/m^3; this can be increased to about 1,800 kg/m^3 by mechanical compaction.

Silex occurs in more or less regular layers (strata) in the lime stone deposits (marl) in Zuid Limburg (Holland). The by-product of cement manufacture using limestone is partly silex. Delivery of silex is in general in three size ranges, ie 0-25 mm, 0-90 mm and 25-70 mm.

Because of the production and process method used, the silex always contains some "tauw", which is pure lime-stone recognisable by the colour. It comes from the uppermost marl strata, which are so hard that it is not crushed in the crusher, but separated out with the silex.

5.4.4 Slag

As an under layer to and filler between revetment blocks various types of slag are used:-

(a) Litz-Donawitz (or LD), a slag produced as a by-product in the LD steel production process. The particles must have a volumetric mass of at least 3,100 kg/m^3.

LD slag can be contaminated by particles of lime, and encapsulated steel and iron. A disadvantage is that the hydraulic action of the lime can cause petrification.

(b) Phosphor-slag

This slag is a calcium silicate which is derived from phosphor ore by the addition of gravel (amongst other items). The volumetric mass of this slag is 2,800 kg/m^3. It is angular in shape and looks like natural rock.

Slag from lead ore which was often used in the past is no longer applied because of the danger of lead leaching out which makes it no longer acceptable in the environment.

5.4.5 Geotextile cloth or membrane

The strength of a woven material is in general expressed as the force exerted at failure per unit length in the direction at right-angles to the direction of the force. The measured strength also depends on the shape and dimensions of the test strip, the method of clamping and the loading. The reduction in the long duration strength in comparison to the short duration load can in some cases be 50% or more. In civil engineering construction the geotextile is generally not subjected to an imposed stress but more to imposed deformation. Geotextiles are well able to cope with such deformation due to the large permissible extensions, provided that the deformation does not occur over a very short distance.

Deterioration of the mechanical characteristics can be caused through chemical or photo-chemical attack on the material of the fibres or through mechanical damage to the construction (Ref 17).

Geotextile durability is in the first instance dependent on the molecular construction of the artificial fibres. Resistance of the fibre material against attack can be improved by the addition of protective substances to the basic matter. Carbon can be added to reduce the adverse effects of Ultra Violet (UV) radiation and also various anti-oxydants. Research has however shown that anti-oxydants in the long run leach out to a substantial degree. Iron ions will shorten the life of polypropylene. Acids can have negative effects on the action of anti-oxydants.

Woven polyamids (nylon) are not subject to the ageing process to any notable degree.

5.4.6 Sandasphalt, bitumenised sand

Sandasphalt comprises sand, filler and bitumen, whilst bitumenised sand contains only sand and bitumen. The bitumen content is 3% to 5% (mass/mass), which makes it open-structured as the bitumen only serves to bind the sand particles together.

The small bitumen quantity is just sufficient to cover the sand particles with a thin bitumen film of a few microns thickness. Moreover, the bitumen concentrates on the areas where the sand particles touch. This means that bitumenised sand, depending on the degree of compaction, the particle distribution and particle shape, has a large permeability which approaches (for the practically possible compaction) the permeability of cohesionless sand.

Durability is determined by the durable binding qualities of bitumen.

For a more extensive treatment of this subject reference should be made to Ref 33: "Guide to the use of asphalt in hydraulic construction".

5.5 Filter characteristics

5.5.1 General

For more information reference should be made to literature Ref No 15, 16, 17 and 18. The most important requirement to be fulfilled by a filter in slope protection works consists of its ability to protect the soil underneath against erosion by waves or currents. The filter should prevent migration of sand particles. In considering the sand-tightness of filter constructions two conditions must be distinguished:-

- sand-tightness irrespective of flow conditions, no matter how strong;
- sand-tightness under the condition that specified limits in the flow conditions are not exceeded.

With the sand-tightness independent of the flow conditions, the tightness depends on the fact that particles of the foundation material cannot penetrate into the filter material due to the particle sizes of the foundation soils being greater than the pore-dimensions in the filter.

With sand-tightness dependent on the flow conditions it is important to know which direction the flow has in relation to the filter orientation.

If the flow is at right-angles to the interface between filter and base material, the filter will always be sand-tight if the (flow) gradient is smaller than about 1*. The gradient force makes equilibrium

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* Translator's note: Dutch usage is: a gradient of 100 %. However English/American usage is as shown here (see Terzaghi and Peck "Soil Mechanics in Engineering Practice", 2nd Ed 1967, page 48 (article 11)).

with the weight of a column of sand (specific weight 2650 kg/m^3 and a void ratio of 0.4). The gradient of 1 is also known as the "critical" or "fluidisation" gradient.

Without cohesive forces between the particles, or other additional pressures due to surcharges, the equilibrium will be disturbed with gradients in excess of 1 and movement of sand particles is possible. With such steep gradients it is important to know if the flow changes direction. If the flow does not change direction, the soil particles can "arch" at the entrance to filter pores, which improves the sand-tightness. Under such circumstances a "natural" filter can be formed, so that for sand-tightness it would be sufficient to exclude only the largest particles from migration through the filter. During the formation of a natural filter all particle transport ceases because the remaining coarse fraction functions as a filter for the underlying layers. Under a cyclic loading a natural filter could be destroyed.

If the flow is parallel to the separation plane between filter and base-material, the items of major importance for sand-tightness are the gradient and through that the flow velocity within the filter. When a critical value of the gradient is exceeded the flow velocity in the filter could become so great that the base-material begins to move and the sand-tightness will be lost.

For granular filters the internal stability of the graded mixture is important. For geotextile filters attention should be given to clogging by silt particles, strength of the cloth (or membrane), elasticity and durability.

5.5.2 Types of filters

(a) Granular filters

Historically, filters have been used which consisted of granular materials, which can be coarse, fine, rounded, flat or angular, more or less well graded, with a large or small volumetric mass.

A filter of that type can comprise one layer with a nearly homogeneous particle gradation, or comprise several layers, with a gradually increasing particle size.

An advantage of the granular filter is its easy adaptability to the filter qualifications by virtue of the large degree of freedom in the composition of the granular mixtures. However, that advantage (and freedom) is limited by economic considerations.

(b) Geotextile filters

The most common forms of geotextiles are:

- mesh-fabric;
- ribbon-fabric;
- mats;
- cloths;
- membranes.

Mesh-fabric is woven using nearly cylindrical threads (monofilament). Characteristic of mesh fabric is the regular pattern of openings and the large percentage of openings per surface unit. The size of the openings is mainly governed by the filament thickness and the number of filaments per unit length (see note below).

Ribbon fabrics are woven with artificial fibre-ribbons lying flat and tightly together in the fabric. Characteristic of this fabric is the very small percentage of openings.

Mats are woven of split film filaments, made of strengthened film, fibrilated or not, and possibly twisted. The filaments are through this process turned into a fibrous structure. The size of the openings depends mainly on the thickness of the filaments and their spacing.

Cloths are woven with multifilament threads, twined or non-twined. The threads are packed closely together. Because the weave is thin, it remains pliable, strongly resembling a textile.

Membranes comprise long or short fibres, which possess cohesion with or without a binding agent. Characteristic of this material is that it looks untidy but very dense. Layer thickness can vary from several millimetres to fractions of mm.

Translator's note: Rankilor in "Membranes in Ground Engineering" (Published J Wiley) speaks of the "number of picks per cm".

Geotextile filters have the advantage of being very thin, specifically when compared to granular filters, but they are on the other hand very easily damaged.

(c) Composite filters

In this group a distinction can be made between granular filters with a binding agent, and granular filters wrapped in geotextiles.

Examples of the first group are the open-textured mixtures of sand and stony materials, bound with bitumen, such as sand asphalt, bitumenised sand, etc. The bitumen provides the binding agent, gives the granular filter a greater degree of stability and strength.

5.5.3 Sand-tightness requirements

Granular filters

(a) Independent from the flow conditions for nearly uniform filter and base-materials (very steep sieve-curves), sand-tightness will be satisfied when (see Fig 10).

$$D_{50f} < 4 \text{ to } 5\ D_{50b} \qquad (1)$$

where D_{50} is the diameter of the sieve - aperture through which 50% of the sample passes.
Index f = filter and index b = base material.

For filter and base-materials with non-uniform particles (relatively flat sieve-curve, see Fig 11), the criterion is

$$D_{15f} < 4 \text{ to } 5\ D_{85b} \qquad (2)$$

(b) Depending on the flow conditions, it is sometimes possible to relax the requirements set out under (a) above. It is, in those circumstances, recommended that the advice of a specialist research laboratory is obtained.

It is moreover desirable to aim for the sieve-curves of filter and base materials to be as near as possible parallel (Figs 10 and 11), because the validity of formulas (1) and (2) depends on that. Compaction of filter material has to be carried out in layers which are not too thick.

Geotextile filters

(a) Independent of the flow conditions sand-tightness is achieved when:

$O_{max} \leq D_{15b}$

in which O_{max} is the largest aperture in the geotextile filter. In practice O_{max} is usually given the dimension which is the equivalent of the average diameter of those sand particles belonging to the 2% (m/m) which passes through the filter: O_{98}.

(b) Dependent on the conditions of flow

When a 'natural' filter is formed through the action of cyclical loading, the following sand-tightness condition is valid:

$O_{max} < D_{85b}$

Under steady flow conditions, for mats, meshes, ribbon-fabric and cloths the formula is:

$O_{90} < D_{90b}$

For membranes the limits are more favourable:

$O_{90} < 1.8\ D_{90b}$

For the design of filters the first consideration must be to investigate the flow conditions. If these are strongly cyclic with gradients in the base material steeper than 1:1, the filter-rules of case (a) above should be used. For other conditions of flow the rules under (b) can be employed.

When flows are mainly parallel to the separation plane between filter and base-material, and the flow-gradient is not too large, even very open filters could be sufficiently sand-tight.

Revetment constructions for coastal defences are often subjected to strong cyclic flows and, therefore, need to satisfy the rules indicated in (a) above.

5.5.4 Other requirements

The permeability of a filter also has to satisfy the requirements related to the danger of up-lift of the revetment due to pore-water pressures within the bank, if the permeability is inadequate to prevent satisfactory relief of such pressures. On the other hand, a low-permeability filter has the advantage of

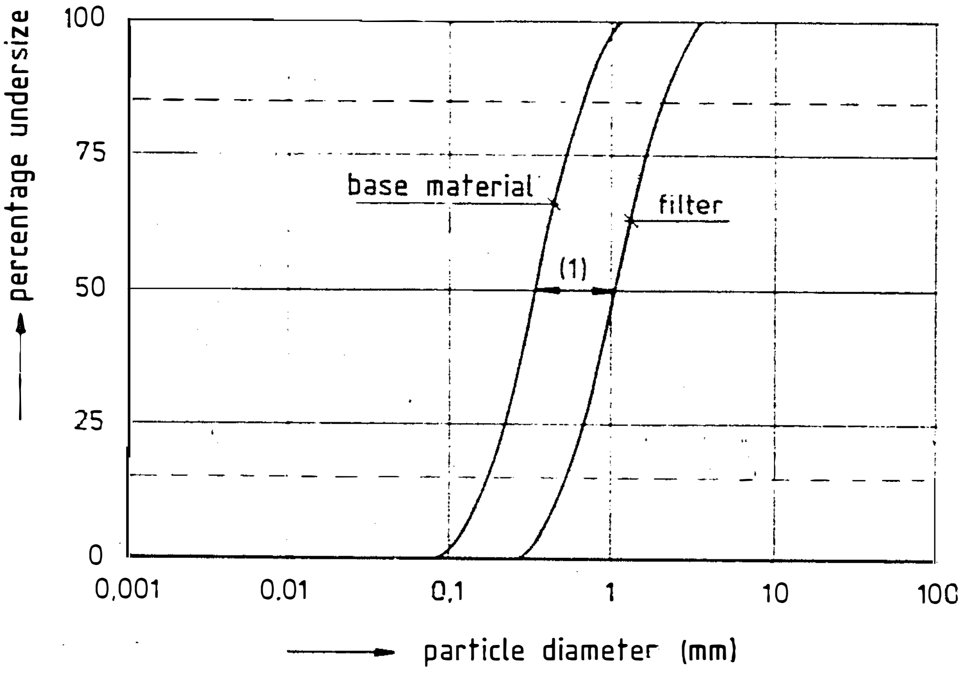

Fig. 10. Filter requirements for uniform material.

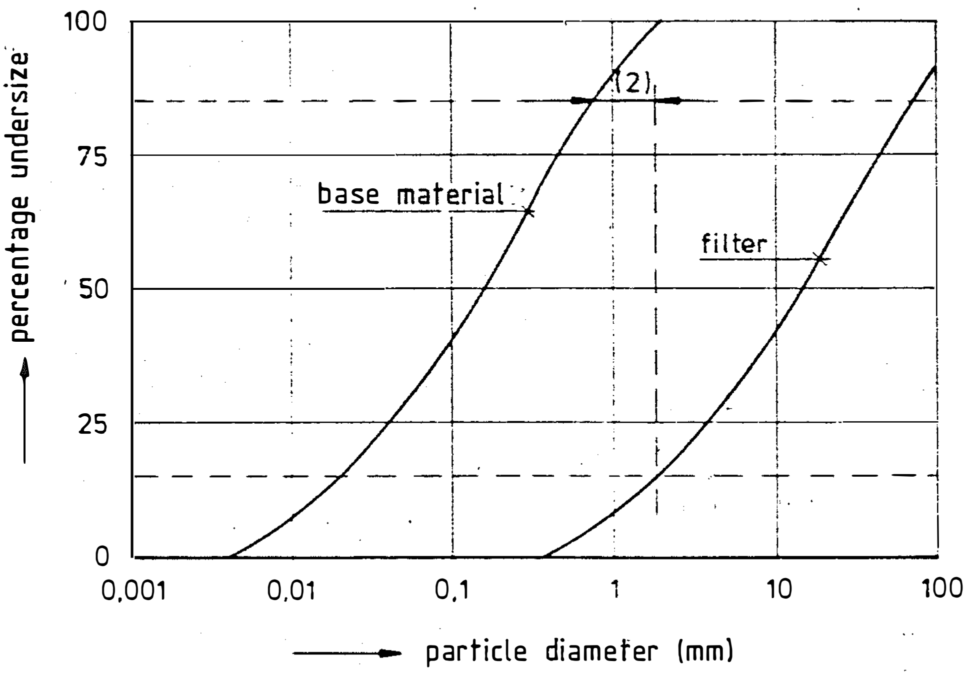

Fig. 11. Filter requirements for non-uniform material.

providing a more stable base for blocks under wave attack.

For the internal stability of granular filters the following general rule can be applied:

$D_{60\,f} \leqslant 10\ D_{10f}$,

which assumes that no internal migration will occur, irrespective of the steepness of the gradient.

It should be noted that for small gradients this criterion can be eased. However, no accurate values are known at present for the critical gradients relating to internal stability.

In order to prevent blockage of a granular filter, the following rule applies:

$D_{5f} \geqslant 75\,\mu m$

5.6 Quality control

If an embankment has to function efficiently for a long period of time, certain requirements will have to be satisfied by the construction as a whole, as well as in parts, and, therefore, the construction materials should also meet certain specifications. In the foregoing discussion various pertinent characteristics of construction elements and materials have been described and in some cases actual values have been given.

In order to ensure that the materials delivered to the construction site meet the (lasting) requirements, it is necessary to provide quality control. This means that the project design includes which material-characteristics are essential and what quality level will be required. The requirements have to be formulated in a way which makes control possible. It is also necessary to have test-methods available, or to develop them. It is then also necessary to investigate if the material as required can be produced and placed on site, and what effect the site construction work has on the characteristics of the material. Finally it has to be decided which quality checks have to be made at delivery and/or construction and what the consequences might be of that procedure.

Naturally, the procedure described above has inevitably to be based on feedback to and adjustment of the original starting points. A situation, where requirements and test-procedures in standards, specifications and guide lines are absorbed, such as is the case for concrete (Chapter 4) has not yet been reached for the various hydraulic construction materials. Developments are in progress, with (often interim) reports available. Much information presented in the preceding paragraphs was derived from such reports.

It is desirable that use should be made of the experience and knowledge which is available in the specialist organisations on quality control.

The ultimate advantages of a consistent quality control lie not only in the qualitative improvement of construction and greater understanding between supplier and client, but also in a more constant and/or improved quality of materials. This provides economy in construction and makes it possible to provide better calculated designs.

In carrying out quality control the following stages exist: preliminary investigation, investigation at delivery and/or during construction and checking at final acceptance. The preliminary investigation is directed at the determination of the suitability of a particular material or mixture (of materials) for the proposed application, also with regard to the availability of the required quantities. All this is based on the proposed requirements (whether or not included in the specifications). Investigations at delivery and/or construction include checking the desired quality, with the main purpose of maintaining a constantly high-level quality. Checking at final acceptance is aimed at the quality of the final total result as well as the quality of all the constituent elements. The checking at delivery and at final acceptance will determine especially any rejection, replacement, guarantees, etc.

Which phase of the quality control process the main checks are made in depends on the materials used and their application.

The responsibility for the delivered quality (of products or materials) lies with the supplier, producer or contractor. The employer checks on the quality through specified controls, carried out by or for him, or through information on tests carried out on behalf of the supplier (factory control).

For the materials mentioned in 5.4 (colliery shale, silex, gravel and slag), the filter requirements are expressed in particle size distribution limitations (see 5.5), which have to be checked by means of sieving at source or at delivery. Where the particle size distribution can alter due to mechanical or other attack, investigations have to be carried out on durability and strength (frost-thaw tests, boiling test, alkaline/lime content, crushing test, etc). In some cases the permeability/density is determined in relation to flow-stability or in connection with strength characteristics.

The determination of the percentage contents of sand, organic matter and clay fraction in clay has to be carried out in connection with the expected performance of the material, but also because of the importance of its uniformity. Investigation of the moisture content is of major importance for workability during construction. Erosion and shrinkage characteristics need to be investigated at the preliminary stages.

Geotextile filters are checked for permeability and sand-tightness. Strength and flexibility (elasticity) are of importance during construction and in connection with settlements in the site location; various checks exist to determine these parameters. The filter material is important for the life expectancy. For polypropylene there is for example an accelerated ageing test. As the geotextiles are produced in factories, control on quality should be carried out in the factory.

Bitumenised sand needs special attention on the percentage content and the type of bitumen used, and this should be determined at the production stage. Supervision of transport and placing on site (temperature, placing method) is important to ensure that the required layer thickness and uniformity are obtained.

6 LOADING ZONES ON EMBANKMENTS (SEA-WALLS)

6.1 Sea-walls

The degree of wave attack on a sea-wall during a storm-tide depends on the orientation in relation to the direction of the storm, the duration and strength of the wind and the extent of the water surface fronting the sea-wall.

The presence of ground levels at or above sea level in front of the embankment, ie saltings or sand banks, and the height and width thereof, or deep channels and the width of these channels, or the presence of harbour-breakwaters, all have their influence. High foreland will reduce the local wave-attack on the bank to a greater or lesser degree. If the sand-banks are located some kilometres away from the embankment, with appreciable depths in the space between sea-wall and sandbank, the initially reduced waves can grow higher again. Wave refraction and diffraction will, in the North Sea for example, change the character of the waves substantially when they approach the deep channels and sand-shoals near the (Dutch) coast. The wave motion further into estuaries is, therefore, not solely determined by the sea-waves. Tidal currents along the sea-wall also have an effect on the wave-action.

Propagation of wave-systems can to some extent be calculated provided that the area concerned is topographically not too complicated. Such calculations should, where possible, be supplemented and controlled by site observation (marks left by floating debris, use of aerial photography). Because the process of wave-distortion is so complicated it is mostly not possible to calculate more than an approximation to the wave-height at the bank.

There is 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. For sea-walls in the tidal region, fronting deep water, the following approximate zones can be distinguished (see Fig 12):-

I the zone permanently submerged;

II the zone between MLW and MHW; this is the zone where wave-attack occurs daily. The ever-present wave-loading is of importance, although the wave-height is mostly less than in Zone III;

III the zone between MHW and the design level; this zone can be heavily attacked by waves, but the frequency of such attack reduces as one goes higher up the bank;

IV the zone above design level, where there should only be wave-run-up.

For sea-walls with a high level "foreshore" Zone I disappears. Depending on the level of the foreshore, under or a little above MHW, Zone II may exist partially or not at all.

Embankments in the tidal area with a high foreshore have the benefit of the wave-damping effect of that foreshore, making wave-attack at lower levels less heavy than on banks lying along deep water.

A bank slope revetment in principle functions no differently under normal circumstances than under extreme conditions. The accent is, however, more on the persistent character of the wave-attack rather than on its size (see Fig 13). The quality of the sea-ward side slope can, prior to the occurrence of

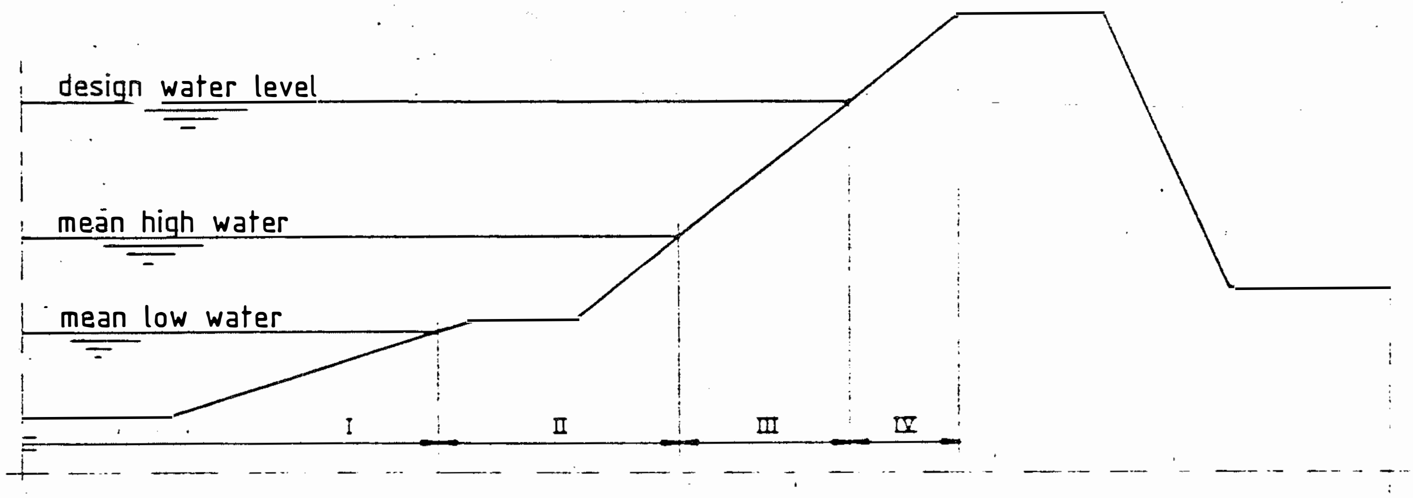

Fig. 12 Division of the slope into zones on sea walls in the tidal region without a foreshore.

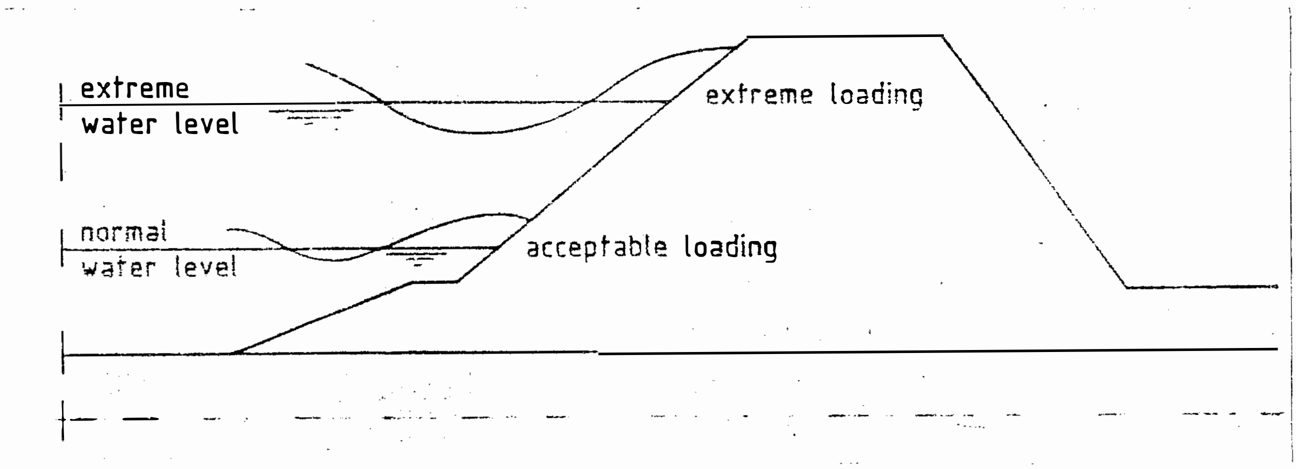

Fig. 13. Difference in intensity and location of wave attack under normal and extreme conditions.

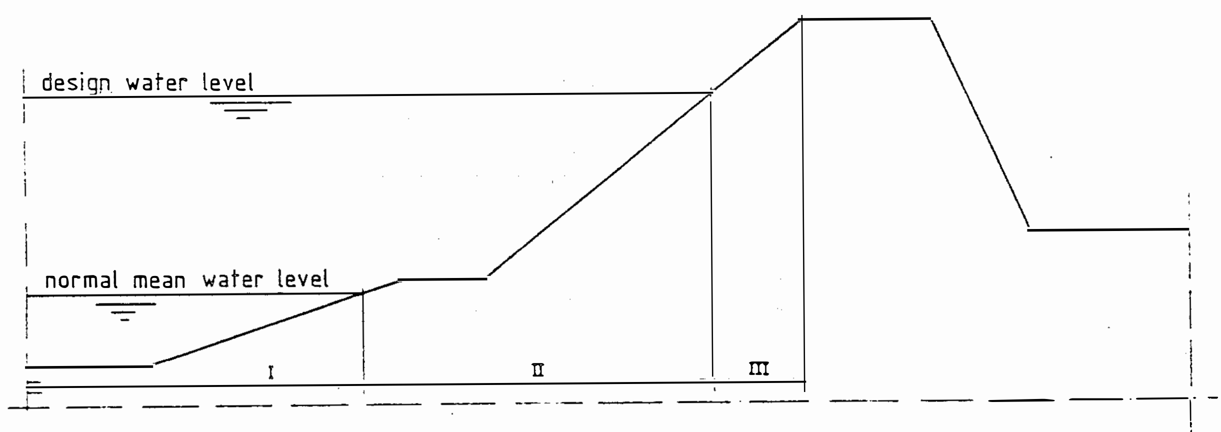

Fig. 14 Division of the slope into zones for lake banks.

the extreme situation, already be damaged during relatively normal conditions to such a degree that its strength is no longer sufficient.

The division of the slope into loading zones has a direct connection with the safety against failure of the revetment; for this reference should be made to Chapter 14.

6.2 Embankments along lakes

Where the water level against a sea-wall is changing continuously under the influence of tidal movement, very rarely will the wave-attack persist for any long period at the same level. In contrast along lake banks there will (apart from exceptional circumstances) be little variation in water level*, and wave-attack will occur daily at approximately the same level, ie the average water level.

Lake embankments without high foreland have the following approximate zone distinctions:- (Fig 14)

I The zone which is permanently submerged. This zone has to be protected against wave-attack and currents, by means of a construction to be laid and maintained below water level. Individual placing of blocks is in this case not possible. A different situation can occur in a closed-off sea-inlet where the average water level lies above the original low-water level, at a level some way up the revetment slope. This has important adverse consequences for the maintenance of the slope below water level.

II The zone between the normal mean water level and the design level. This zone is attacked by waves and currents, ice and other floating debris. In this zone waves break just below the ambient water level.

III The zone above design level, where only wave-run-up occurs. A grass mat is often adequate to meet the requirements in this zone.

6.3 River embankments

River embankments are generally constructed as green banks. Only where the banks are exposed to serious attack, a hard revetment may be applied:-

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* Translator's note: This is not necessarily valid for water storage lakes (eg Rutland Water) where wave-attack at draw-down is at very different levels!

- at locations where the channel cross-section narrows;

- at locations where the main current runs close to the toe of the bank slope, eg the outside of bends;

- at locations exposed to strong winds;

- in the proximity of locations where there are discontinuities in the cross-section, such as bridge piers, etc;

- where the bank slope is flanking deep water.*

The locations where revetments have been constructed were generally determined on the basis of experience of the controlling authorities, following on the history of damages in the past. Such revetments are usually protected only against currents; exceptionally, in cases where there happens to be a long wind-fetch, wave-attack can also be expected. In some locations additional attack can be created by waves and drag-velocities due to passing ships.

In contrast to the situation with sea-walls, river embankments often (*in Holland) have to hold high water levels during prolonged floods (lasting for weeks) (*large Dutch rivers). The banks can under those conditions become saturated. When flood waters recede again quickly (within a period of a week) the pore-water in the bank must be able to drain out freely. This means that revetments should be of an "open" structure in comparison with the permeability of the layer under the revetment.

7 EMBANKMENT PROFILE

The shape of the bank slope needs to be observed longitudinally as well as in cross-section.

7.1 Cross-section

Important aspects in the choice of the cross-section include, amongst others:-

- the slope angle or gradient;
- shape of the cross-section;

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* Translator's note: This applies to the large rivers in Holland, ie Rhine, Wasl, Maas, where depths for shipping are up to 5 m. Rhine from Hook of Holland to Rotterdam is about 12 m deep.

Gradient of the bank

The gradient may not be so steep that the whole slope or the revetment can lose stability (through sliding), both during construction as well as in use on completion. These criteria give, therefore, the maximum slope angle. A flatter, more gentle, slope leads to a reduced wave-force on the revetment and less wave run-up; wave energy is dissipated over a greater length. Using the wave run-up offers, for example, the opportunity to calculate the crest height of a trapezoidal profile of a bank and thus determine the volume of the embankment given a firm crest width.

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 surface increases as the slope angle decreases. Similar considerations can be compiled for banks with a sea-side berm.

The optimum cross-section (based on costs) can be determined when the costs of earth works per m^3 and those of revetment per m^2 are known. Careful attention is, however, needed because the revetment costs are not independent of the slope angle.

Another point to be taken into account in the choice of a slope angle is the space occupied by the embankment; this could be the decisive factor when compared with existing (and/or proposed) use, in the context of economic optimisation.

Shape of the cross-section

The slope can be flat, convex or concave; various opinions exist on these alternatives. With a concave profile the wave run-up is reduced at the lower water levels compared to run-up on a convex slope, but the converse is true at higher water levels. The flexibility of the block revetment, in connection with possible settlement and soil erosion, will be adversely affected on a convex slope, due to the arching action. A concave profile has in this respect an advantage over a convex profile.

It is not easy to determine the value of the advantage or disadvantage of the flexibility as a function of the shape and degree of curved slopes. The concave shape makes it more difficult for wave-action to remove individual blocks, because of the "upside-down" arching effect of the concave slope. A disadvantage of a pronounced convex profile is, that with many systems the block-joints will become much too open.

The water-side berm is an element in bank-construction, of which the function has changed in the course of time. Particularly in Zeeland* outer berms have been applied frequently. 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, even at lower levels, better than on the steeper angle of an uninterrupted slope. Moreover, the outer berm produced an appreciable reduction in wave run-up.

The condition for a satisfactory realisation of the two main functions, ie economies on stone revetment and reduction of the wave run-up is that the berm level has been adjusted correctly to the prevalent water levels which can occur. It must be obvious that the berm level should be higher if the bank has a less sheltered location; in different cases there can be differences of some metres in height. Further, the outer berm level, in order to obtain a substantial reduction in wave run-up, should be close to the mean water level of the design storm flood. If the berm lies too much below that level, the highest storm-flood waves would not break on the berm and the run-up will be inadequately affected.

Present practice is to place the outer berm at design water level as indicated in the Delta committee's report. For a storm-flood berm at such design level there are in general no problems with the growth of grass on the berm and the upper slope. However, there can be circumstances which require also the application of a hard revetment on the berm and even on a part of the upper slope (see Fig 15).

In order to avoid extra forces coming into action at corners in the profile of the outer slope, all such corners should be rounded as much as possible. An important function of the sea-side berm is its use as an access road for bank maintenance.

River (flood) banks often have a revetment starting high up on the slope. It then needs solid support along the bottom line. Care should be taken to prevent erosion of the grass mat at that junction with the revetment.

7.2 Longitudinal profile

Due to irregularities in the longitudinal profile of an embankment, in connection with the topography of

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* South-west Province, comprising mainly islands

the terrain in front of the bank, some reaches of the slopes could be subject to more than normal attack (for example due to refraction of waves).

Not all revetment systems are suitable for use on a curved slope, due to several complications:-

- systems which do not permit deviations from a straight line;

- going around curves leaves gaping joints;

- difficulties in placing the blocks by machine.

Mechanical methods for placing of blocks is in practice limited mainly to straight lines and to large radius bends with sufficiently large areas. Placing of blocks by mechanical means is not only economical, but it can also clamp the blocks tightly together, much tighter than can be achieved manually. This assists in producing a good slope protection.

It is however important to remember that any damage to a block revetment should be repairable manually.

8 TRANSITIONS AND BOUNDARIES

The experience of many controlling authorities is that much damage takes place at transitions from one type of revetment to another and at the line where the revetment ends. It is possible to give a great deal of care and attention to the revetment construction, but when the weakest link breaks, the safety of the defence system is at risk. This chapter is, therefore dedicated to these special aspects.

8.1 Toe construction of revetments

The toe construction has the function of revetment support and to protect it against erosive action on the bottom edge of the slope.

A distinction has to be made between a sea-wall and a lake embankment. The daily occurring wave attack on a lake bank, in contrast to that on a sea-wall, will be approximately at a *constant level. The daily wave-attack will have to be countered through that part of the revetment where the underwater slope meets the revetment on the slope above water.

On sea-walls the toe construction is usually, for constructional reasons, placed above low water.

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* Translator's note: except for water storage reservoirs (eg Rutland Water) where water levels can vary greatly, as stated previously

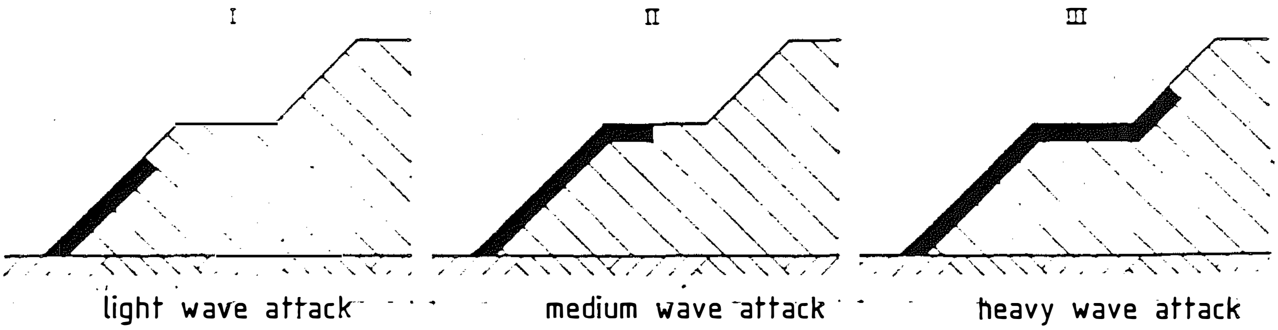

Fig. 15. Possible termination of the hardened revetment at an outer berm situated at design level.

Wave-attack of any significance will not often occur at that level, because the winds causing waves to occur generally also raise water levels, so that even at the official low water time, the waves will occur at a higher level. An exception to this is the sea swell which can indeed give heavy wave attack at lower levels.

When embankments have high-level foreshores, the toes of the banks are protected against the heavier waves by the breaking of waves on the high shore levels. Where a slope has to be revetted with rectangular blocks, a need is created for "straight lines". Such lines are provided by means of boards fixed to a row of timber piles. The boards could be either of wood or concrete. This method of construction is certainly not always to be taken as adequate for heavily attacked lake-bank toe constructions. Even if there is a berm-support in front of the toe, with a heavy mattress, there is still a chance that the wave-motion will gradually remove sand to the base level of the boards, so that the foundation under the lowest revetment blocks can be washed out and the toe will subside.

An improved construction in this case would comprise a sheet pile wall of tongued and grooved boards, preferably strengthened along the top with a waling (see Fig 16). This construction provides the toe with a more gradual transition from the slope to the level of the stone-protected berm, which is an important aspect considering the "to-and-fro" wave motion. This system also provides a better means of connection with the first row of toe-blocks.

Design of toe-construction requires attention to be given to the following points:-

(a) If blocks are placed directly against a solid line of timber posts, the danger of washing-out of base material exists because of unavoidable gaps between the posts; moreover, this method does not produce straight lines. The starting point for rectangular blocks has to be the toe-board. Exceptionally, the solid line of posts could be considered for use with polygonal blocks, but great care will have to be given to the sand-tightness of the construction.

(b) If the toe construction projects too far above the adjoining foreshore or support berm, there will be a chance of slope undermining, with the toe construction "toppling over"; a possibility

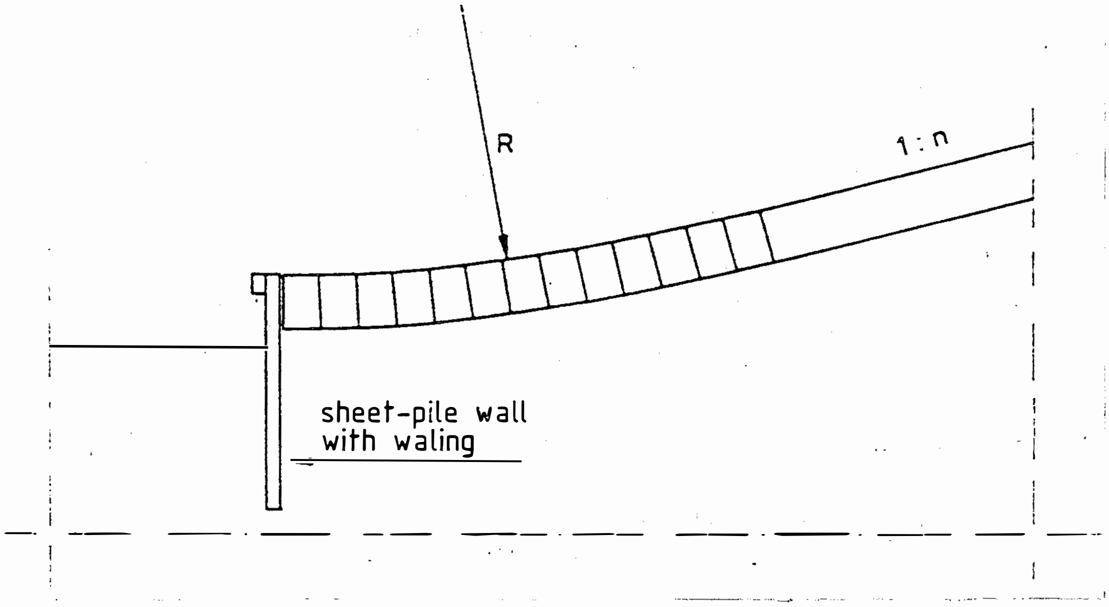

Fig. 16. Toe structure with timber sheet-pile wall as a possible solution for lake dykes.

which will usually manifest itself during the construction stage.

(c) When the dumped stone level on the support berm is pitched too high, the transition between berm and toe will be discontinuous, leading to awkward extra forces acting on the toe construction. The stability of the support berm is also negatively affected.

(d) When the toe board is not sufficiently deep the slope could be undermined and, with reduction of foreshore or berm level, the stability may be lost.

(e) With a weak sub-soil there is a danger that a sheet pile wall or a row of piles may not be sufficiently stable.

8.2 Top-edge to the hard revetment

The transition from the hard revetment to the grass mat often forms a weak point in the construction. A good grass mat is capable of resisting breaking waves to some degree, but these must not be large nor occur too frequently. The more frequent wave forces and run-up have to be absorbed by the hard revetment.

The location of the revetment/grass mat transition can theoretically be selected at a lower level if account is taken of the wave-run-up reduction due to the effect of a rough and open slope, the presence of an outer berm, the wave-reducing effect of a high-level foreshore, or a combination of all these possibilities. The determination of the transition location is, however, often a matter of experience combined with local circumstances (debris-line observations related to past occurences of damage; the "trial and error" method, etc) or the application of a practical "rule of thumb" (for example half-way between "design" level and bank crest level).

Knowledge on the resistance of grass mats against breaking waves or wave run-up is at present limited. Care has to be taken at a transition that the discontinuity is as small as possible in order to prevent the possibility of local undermining. It is, for that reason, not recommended to drive a solid line of posts along the revetment's top-edge, which projects above the plane of the revetment, as is sometimes done to prevent floating debris getting on to the grass mat!

The principle of the transition construction must be: a greater strength than the grass mat with a hydraulic roughness about equal to that of the grass mat. A strip of double-thickness "road-bricks*", placed "on edge" (0.10 m high), with top soil filled joints, would be satisfactory; this strip of bricks must, however, get the opportunity to be thoroughly integrated with a well-grown grass-mat.

A less labour-intensive and more effective possibility comprises the use of concrete blocks with earth-filled perforations for grass to grow through. The units used for this type of application are much bigger and heavier than road-bricks. Also, the discontinuity of the transition from revetment to soil is reduced, and it is to be expected that the grass will form better and deeper roots into the clay subsoil. Problems can be created in the context of mowing the grass if and when concrete elements are not placed to accurate lines and levels, or when they have settled to different levels.

When the upper edge is only protected by a concrete strip, a relatively small amount of undermining could cause substantial damage. In such cases it is recommended that the concrete strip is supported by posts driven to top-of-concrete or a little below that level.

When hollow concrete blocks for grass growth are employed, no support posts are needed as the blocks are sufficiently stable. That construction method has an important additional advantage when the embankments are grazed by sheep**, because the sheep's habit of forming a path along the revetment-top edge will be prevented.

8.3 Transitions to other revetments

When two different types of revetment adjoin, special attention should be given to the subsoil.

When for example a fine-grained soil exists immediately alongside a coarser material (see Fig 17) there is a danger that the finer material under influence of ground water flow and soil pressures will penetrate into the coarse material. This is a disadvantage to the drainage function of the coarse material and can also lead to local slope settlements.

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* Translator's note: Roads in Holland are often brick-paved, using specially developed bricks for that purpose.

** Translator's note: Common maintenance method in Holland to keep the clay well-compacted; and provides income!

The "sand-tightness" requirement can be satisfied through an effective granular composition of the coarse material (granular filter design). It is often easier and cheaper to use a geotextile as a filter, or use a concrete separator to prevent migration of the finer material.

Problems can often be created at the transitions between coarse materials, for example colliery shale, slag, gravel, rubble, etc., and the finer soil such as sand and clay. The situation shown in Fig 17, where the fine and coarse materials are indicated, also needs to be considered for the measures required in order to prevent undermining of the asphalt-concrete when damage occurs to the concrete block revetment.

Various methods of construction to reinforce the transition are possible. For example the following:-

(a) Let the asphalt-concrete extend a short distance over the colliery shale (instead of the sharp break shown Fig 17). A disadvantage is that wave-impact pressures will be transmitted underneath the asphalt-concrete, with the associated danger that the asphalt cover could be lifted.

(b) Increase the thickness of the asphalt-concrete in a strip along the joint. This can however cause problems during construction, where a trench has to be cut to provide the depth required.

(c) Construct a row of posts with boards (keeping tips just below the slope-surface).

(d) Fill the joints of the upper layers of blocks with bitument. This is only possible if the joints offer adequate space to do so.

It is possible with the application of an edge of concrete strips or boards (see Fig 18), at the transition from concrete blocks to basalt-columns, to prevent the clay penetrating the rubble base-layer under the basalt. The concrete edging should be sufficiently deep.

The experience of the authorities in charge of embankment maintenance shows that there is fairly considerable damage occurring at transitions such as that shown in Fig 18. The cause can be in the possibility that the basalt columns are not sufficiently tightly held by the concrete strip, and also in the foundation discontinuity at the transition. With the basalt blocks thus being less tightly packed, and wave-impact pressures in the rubble foundation layer building up as escape into the clay under the concrete blocks is not possible, the basalt blocks would be subjected to greater uplift pressures. Greater stability could be produced by penetrating say a strip of 0.5 m of basalt blocks along the transition line with liquid bitumen.

For concrete slope-revetment systems which have polygonal shaped elements, similar to the basalt block shape, and which also have to be placed on a granular filter, no special attention needs to be given to the transition. Care is needed in one respect, ie that the granular filter material under the concrete block cannot migrate into the rubble layer under the basalt, because it could lead to subsidence. In such cases a granular transition strip will be required.

In order to illustrate to what extent the pressures under a revetment can rise for instance as a result of the closure of a permeable filter with a concrete plank, Fig 19 is presented.

The flow-nets shown were determined by computer. For more information on this subject the CUR-VB/COW report "Background to the Guide to Concrete Block Revetments" (Ref 32) should be consulted.

It is recommended that, as a general rule, transitions to other types of revetment in the longitudinal direction of the embankment should be avoided as much as possible. It is desirable that a bank slope should be protected by one type of revetment from the toe to the top-edge, and especially an underlayer with the same permeability throughout. This is often, however, not possible, for instance when an existing revetment is being extended, and where at many recent embankments the hard revetment is altered from stone to an asphaltic revetment.

When concrete edge-strips or boards are placed at right angles to the longitudinal bank direction, the change-over is more favourably placed. In these cases only those waves which arrive on the slope at an angle could cause some extra pressures under the blocks in the corners along the top-edge. An advantage of concrete edge-strips placed in this direction is that the slope is then divided into separate compartments, thus preventing steady progression of any damage along long lengths of bank.

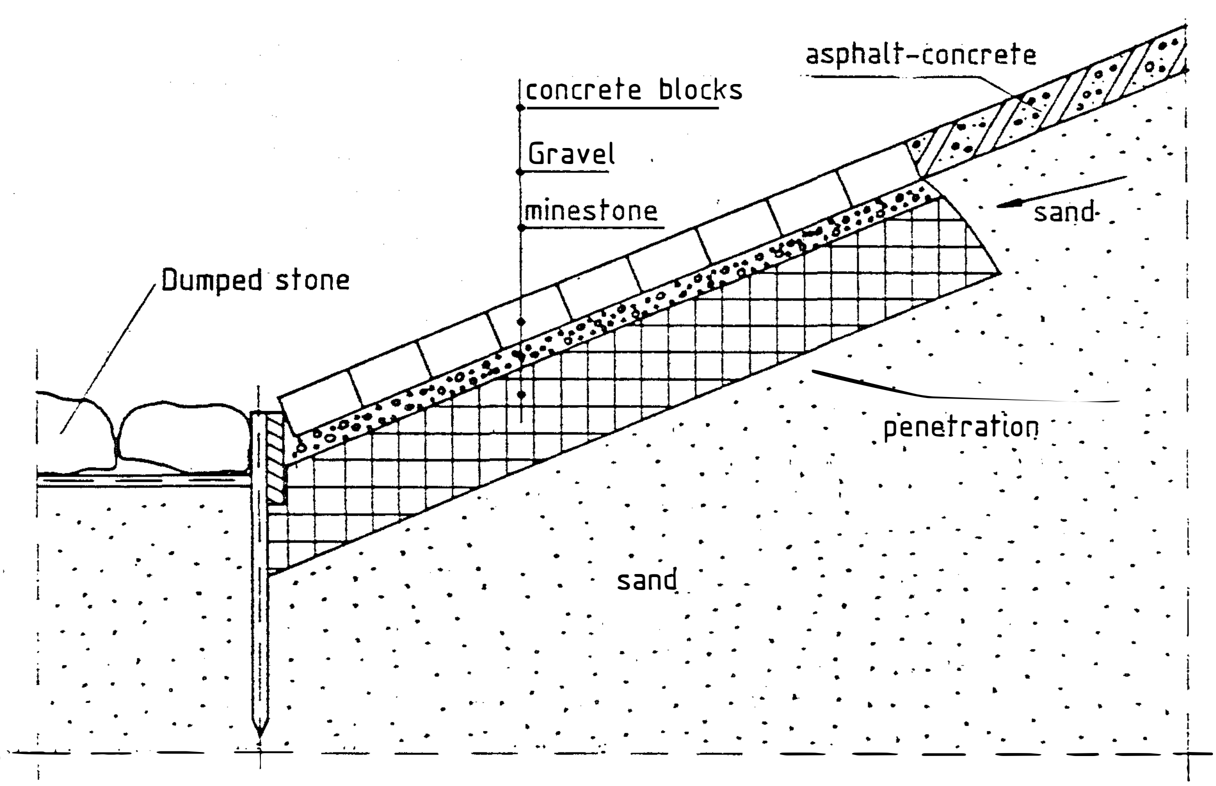


Fig. 17 Penetration of sand into the mine waste stone (colliery waste or shale)

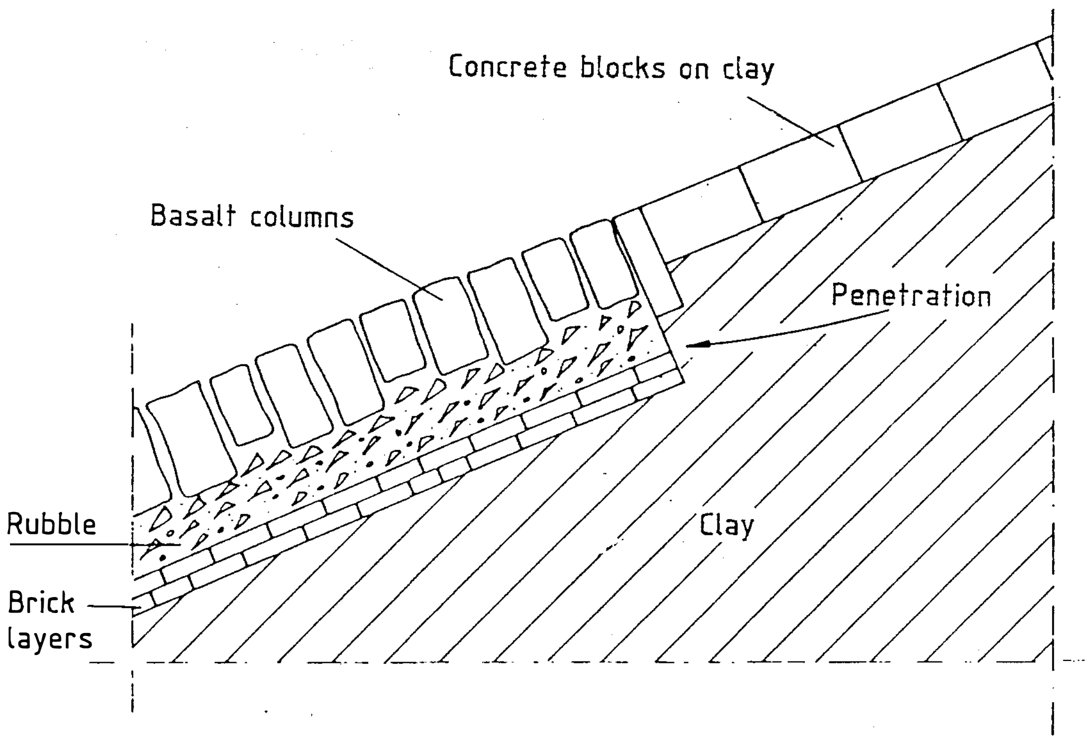
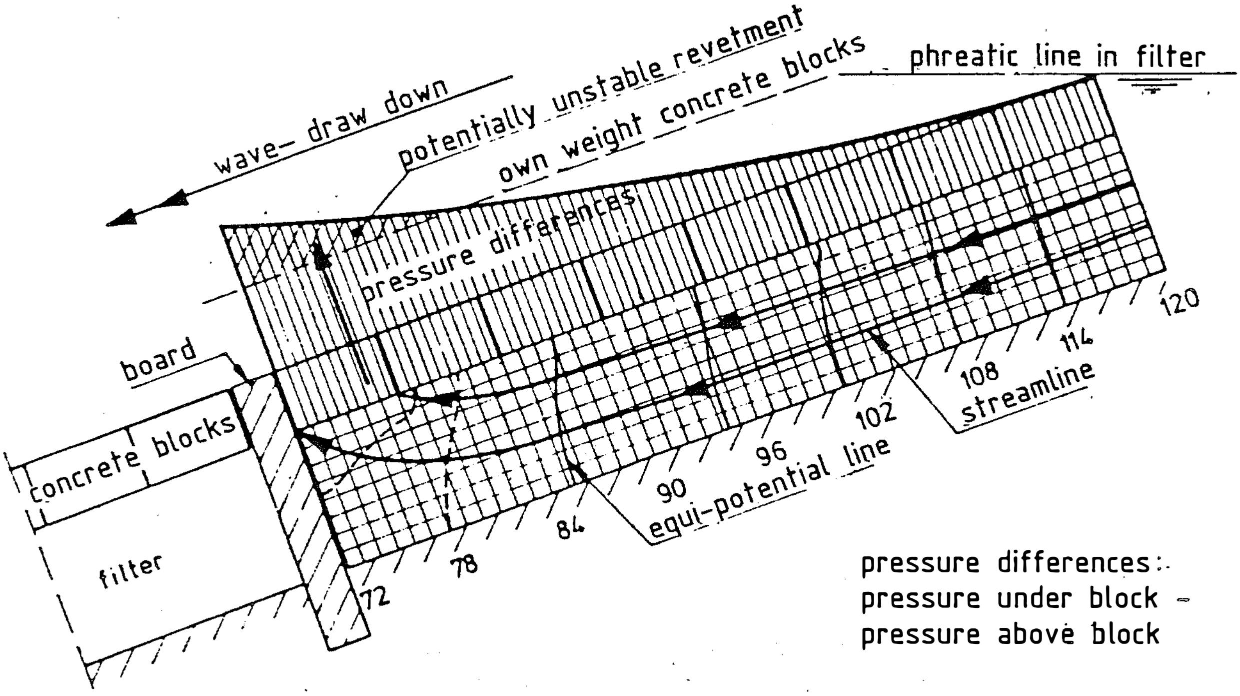
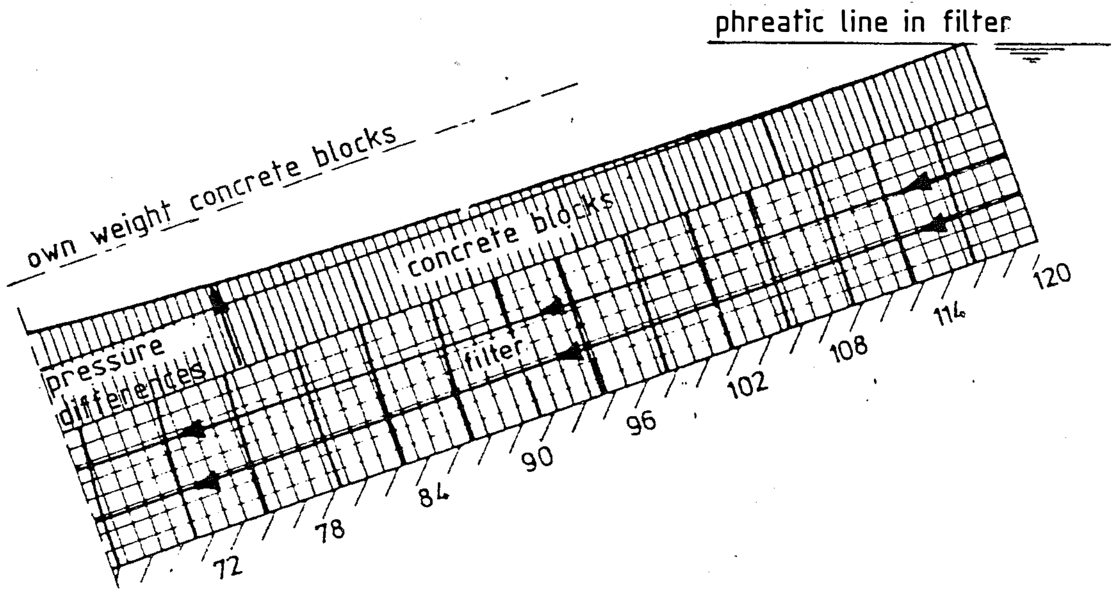


Fig. 18. Transition from basalt columns to concrete blocks.



a. pressure differences due to board in filter



b. pressure differences for continuous filter

Fig. 19. Increased pressures associated with disturbances in the permeable filter

In order to use concrete edgings as stops to further damage-development, these edgings have to be sufficiently heavy and deep. Making edgings too deep can be dangerous as it would make a separation cut in the foundation layer.

It is very important to ensure that blocks form a good and strong fit against the concrete edgings.

9 CONSTRUCTION ASPECTS

9.1 General

The concrete block revetment differs mainly from a natural stone revetment by being more regular in shape and also more accurate in measurement. This type of revetment can, therefore, be placed in the traditional "manual" method, using specialised labour (stone-setters) but also by means of machines operated by skilled men. Using cranes fitted with special clamps the blocks can be picked up and placed quickly and accurately, which is particularly useful for the larger size blocks. The increased production rates are often required as most site construction works on coastal defences have to be carried out (in Holland) between mid April and mid October, and even then probably only when tides permit access.

9.2 Product

The blocks, columns or slabs for concrete revetments should be manufactured in well-equipped factories under expert supervision in order to ensure a constant high-quality standard, both for the concrete composition (particularly important for the aggressive coastal environment) and for the dimensional accuracy needed for mechanical placing.

All appropriate revetment elements should be quality tested according to the standards specified (see Chapter 4).

9.3 Storage and transport

After fabrication the blocks have to be stored for hardening, and mechanical damage has to be prevented. Handling operations should therefore be limited to a minimum.

For transport to the site, the storage area must be easily accessible by road or water. As it will be very rare that blocks delivered from factory storage to site can be placed directly on the slope, it is generally necessary to have a temporary storage yard at site.

When blocks are destined to be placed mechanically, the units should be appropriately assembled at the factory so that they need no re-arrangement for site-placement. Site stacking should be done on level ground to prevent the stacks sliding or falling over.

It is important to check that storage and transport are arranged to eliminate soil adhering to the units as this would cause problems with accuracy of placing. Transport vehicles should be able to cope with the terrain at site.

9.4 Construction

Revetments comprising concrete blocks or columns are in general placed directly on the prepared slope materials, eg clay, colliery shale, broken stone (gravel size) or silex. Because the concrete blocks are dimensionally accurate, it is necessary to finish the base accurately to profile. Templates are in general use for this purpose, with planers or scrapers employed to produce the desired profile and levels, depending on the base material used, eg clay, filters, etc (see Chapter 5).

Placing larger units by hand requires much labour and limits production, so that this method will be used only to obtain good transitions to existing revetments, or for repairs to systems which are not suited to mechanical placing.

For mechanical placing cranes are used, to pick- up several elements at once (with clamps), for direct placing on the slope. These multiples of elements can cover areas between 1 and 3 m^2. These can be placed by the crane with some force to provide a good contact with the adjoining blocks. Production can of course be increased by employing several cranes.

Revetments with large joint spaces, such as for example occurs with polygonal concrete columns, need to have the joints filled with great care, for example with broken stone to produce a firm, immovable, but permeable surface.

Problems do occasionally occur with the joint filling in the latter case, when blowing sand fills the joints before the broken stone can be hammered in.

10 MANAGEMENT AND MAINTENANCE

10.1 Inspections

There is at least one annual inspection of the complete slope protection, primarily for safety reasons, carried out by those who are responsible for the maintenance.

The responsible bodies are commonly taken to be the "waterschappen"*; provincial and Central Government embankments also exist as do occasionally local Council embankments.

Attention is directed towards essential repairs of for example subsided parts of the slope and renewal of worn-out areas, or extension of existing slope protection. After each storm tide an extensive inspection is conducted at low tide to locate any possible damage. (This is usually done by the engineer or surveyor to the Board). Maintenance staff, retained by such Boards, take note of any damage during their daily work schedules.

As the Provincial Governments are usually charged with the task of overall supervision of coastal defences, the Provincial technical staff also carry out inspections. Due to the supervisory task the accent is on the control aspect of inspections rather than on the duty to find damaged bank slopes.

10.2 Maintenance

Annual maintenance covers broadly the following activies:-

- Repair of damage after heavy wave-attack. This can be limited to very local damage, for example a single block that was "lifted" and not returned to its own position. It may, however, also concern more extensive damage, for example at connections to other constructions.

  Depending on the nature of the damage, the repair would comprise the breaking out of a single element and replacing it with fresh concrete fill placed on-site, or a greater area has to be broken out, and after filling in the under-layer to the correct levels, the blocks or new elements are put back in again.

  Where damage occurs repeatedly in the same location, investigations would clearly be required into the construction of the slope in that area, and introduce appropriate counter-measures.

- Repairs on site of subsidence which occurred as a result of soil-losses or due to collapse

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* Waterschappen (or Polders): Something like Internal Drainage Boards, ie farmers, landowners, forming a legal group (Board) to take care of drainage, floodbanks, pumping etc, all to serve the combined benefit of their own community.

(displacement) of the toe-construction. In these cases the toe-construction and the underlying materials will be improved at the same time. Costs are often high because the whole slope revetment has to be removed from the toe up to the top-edge and on both sides at an angle of about 45° to the top.

- Extension of existing revetment areas above and below can also be counted as part of the maintenance. Downward extension when some of the foreshore is removed, and upward extension when the damage pattern indicates the need for extended protection.

- Improvement works of limited extent, or works of greater extent over a number of years, can be counted as part of maintenance. These improvement works could be fairly drastic and include renewal of toe-construction, under-layers and revetment construction.

- Repair of damage created by anglers lifting out blocks or other forms of vandalism.

- Repair of damage due to stranding of ships, deck-loads or drifting ice.

10.3 Repair feasibility

In comparison with natural stone revetments, the feasibility of repairing concrete block revetments is in some cases more restricted, because:-

- the inter-connection between blocks is often much greater, which makes removal of single elements more difficult;

- it is impossible to push single elements back in their own place in the revetment; it never fits. Moreover, it also generally strains the true alignment.

These limitations are not valid for concrete elements with shapes similar to basalt columns. Apart from the restrictions referred to above, other possibilities include the following:-

- It is possible to cast some elements in-situ, as is the rule when irregular gaps have to be filled. However, because precast blocks are generally of superior quality, repairs should as much as possible be carried out using precast blocks. If and when in-situ concrete has to be used, attention should be directed to the production of a good and dense concrete and good curing.

- The new blocks needed for the repairs can be delivered readily and in small quantities.*

10.4 Re-use

When blocks have been placed without mortar or asphalt joint-fillers, re-use of blocks is nearly always possible, unless blocks are too old or of poor quality. With large elements, such as slabs and step-slopes, no re-use is possible. Also, for constructions when cement mortar or asphalt were used to penetrate the revetment, re-use is rarely possible (for revetments). It could possibly be used as loose-dumped material.

11 HYDRAULIC BOUNDARY CONDITIONS

11.1 General

In view of the function of (coastal) water defences the loads will obviously be mostly due to the actions of long and/or short waves. In broad outline the following wave phenomena can be distinguished:-

(a) low-frequency water level changes, such as flood waves, tidal waves, wind set-up gradients and seiches;

(b) wind waves and swell;

(c) ship's waves in navigable waterways

- primary wave with the drag-wave forming part thereof;
- secondary ships' waves;
- combinations of primary and secondary waves;

These water level variations strongly influence the area which needs to be protected with hard revetment.

Water level variations on canals and water-storage channels** are comparatively small; probably only caused by lock-water, seepage, drainage and wind effects. Water levels on lakes can vary as a result

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* Translator's note: No mention is made of the obvious suggestion, ie keeping spare blocks in strategically located stores!

** Translator's note: The Dutch word "boezem" covers all waterways, in a flat polder area, used for run-off storage as well as for discharge to drainage pumps.

of wind set-up, inflow or outflow of water, and evaporation. Water levels in a reservoir can change markedly due to filling or emptying, but rainfall and wind set-up can also play a role.

Water levels on a river are detemined by the river's discharge regime, and in addition for the lower reaches (estuaries) by tides and also wind-set-up. For a coastal defence embankment water levels are governed by tides and winds.

The most complex situation occurs at coastal banks, where water level fluctuations can assume many forms. For this reason, and also to reduce the volume of this guide, the considerations here will be limited to coastal banks.

The purpose of this chapter is to provide background information on the various phenomena in preparation for the matters discussed in Chapters 12 and 14. In this context wind-waves and the wave-deformations in front of and on the banks will be described. For more in-depth considerations one should refer to the CUR-VB/COW report "Background to the Guide for Concrete Block Revetments" (Ref 32).

11.2 Wave characteristics

11.2.1 Individual wave

Wave theories have been known for 200 years. These theories are based on the assumption that the wave can be described by the wave height H and the wave period T (or by the wave length L), and that the theories refer to regular waves. Regular waves do not occur in reality at sea, but such waves are important because they contain the basic elements of irregular sea waves.

In general the real wave phenomena are very complicated and difficult to express in mathematical terms, due to non-linearities, three-dimensional characteristics and the random nature of waves. It is the task of the so-called deterministic theories to formulate mathematically as accurately as possible the form of the free surface and the motion of a regular wave for various wave heights and periods and at different water depths. In doing so it is usual to make a distinction between linear and non-linear wave theories depending on which order of flow forces is taken into account.

The significance of the most important theories is presented in Fig 20. That figure also gives a general view of the validity limits of the various theories. The dimensionless parameters H/gT^2 and d/gT^2 have been used.

The variables used are as follows (see Fig 21):-

L = wave length: the horizontal distance between two successive wave-crests;

T = wave-period: the time passage recorded at a fixed point between the passing of two successive wave crests;

f = wave frequency = 1/T

C = wave celerity: also known as wave velocity or phase velocity - it is the speed with which the image of the wave profile travels; it is therefore not the velocity of the water particles;

H = wave height: the difference in the levels of the highest and lowest points of the wave profile;

d = water depth: measured in relation to the still-water level.

There are two classic theories, one developed by Airy (1845) and the other by Stokes (1880), which describe the regular wave. These theories predict wave behaviour generally better, when the ratio d/L is not too small.

For shallow water the cnoïdal wave theory, originally developed by Korteweg and De Vries (1895), often gives a reasonable approximation for single waves. That theory is, however, fairly difficult to calculate. For waves in very shallow water the solitary-wave theory presents the best method to describe the waves. In contrast to the cnoïdal theory, the solitary-wave theory is reasonably easy to manipulate.

Sea-waves can be subdivided into:-

- developing sea, when waves are growing under the influence of the wind;

- swell: this comprises sea waves which have been withdrawn from the action of the wind (which originally created the waves), either because the wind died down, or because the wind-field has moved elsewhere, or due to the waves having moved out of the wind field.

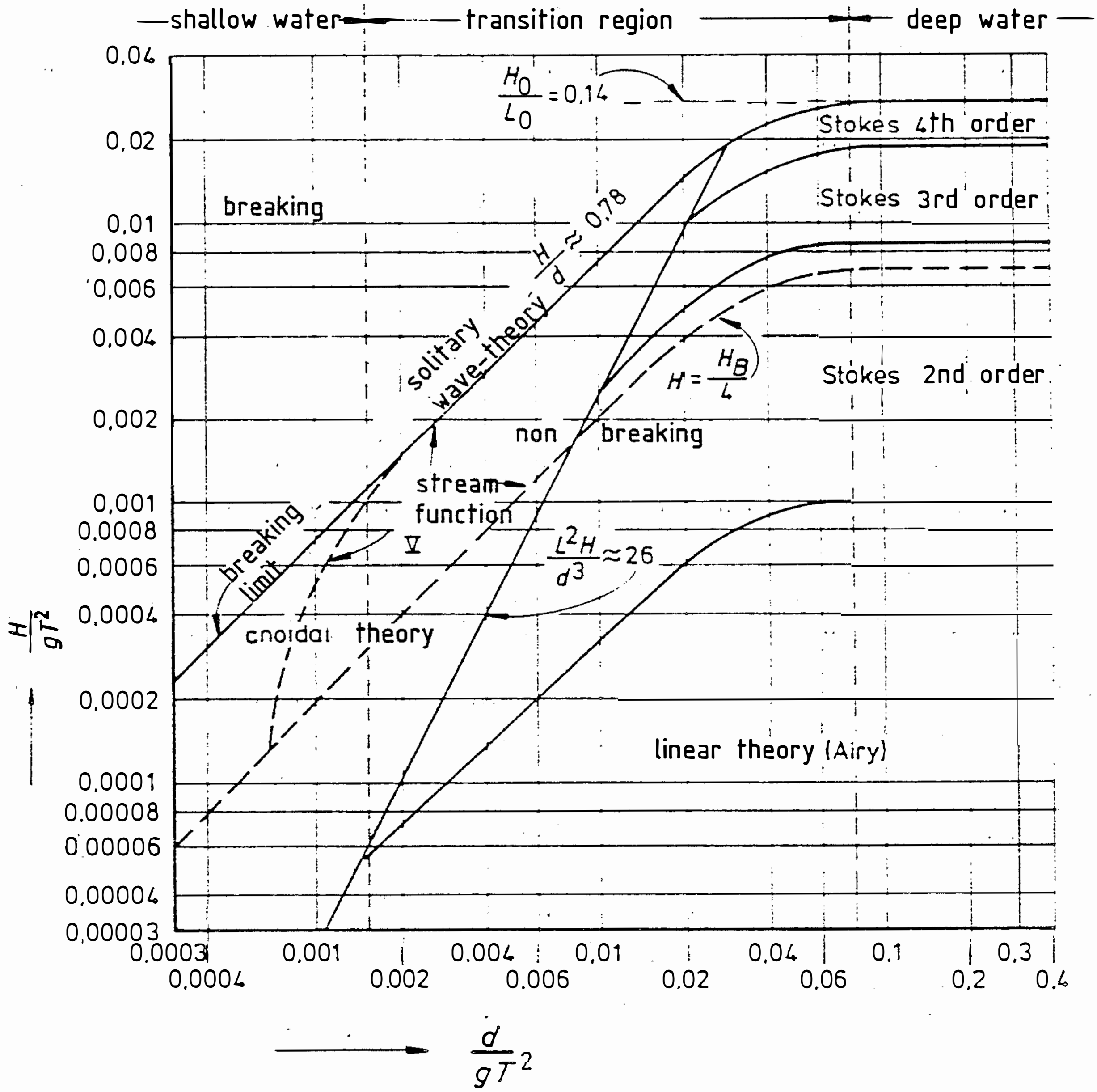


Fig. 20. Ranges of validity of the various wave theories.

	Shallow water $\frac{d}{L} \leqq 0{,}04$	Transition zone $0{,}04 < \frac{d}{L} < 0{,}5$	Deep water $\frac{d}{L} \geqq 0{,}5$
Wave celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi} \tanh \frac{2\pi d}{L}$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
Group velocity	$C_g = C = \sqrt{gd}$	$C_g = \frac{1}{2}\left(1 + \frac{\frac{4\pi d}{L}}{\sinh \frac{4\pi d}{L}}\right) C$	$C_g = \frac{1}{2}\, C = \frac{gT}{4\pi}$
Wave length	$L = T\sqrt{gd} = CT$	$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L}$	$L = L_0 = \frac{gT^2}{2\pi} = C_0 T$

Table 1. Most important formulae of the linear wave theory.

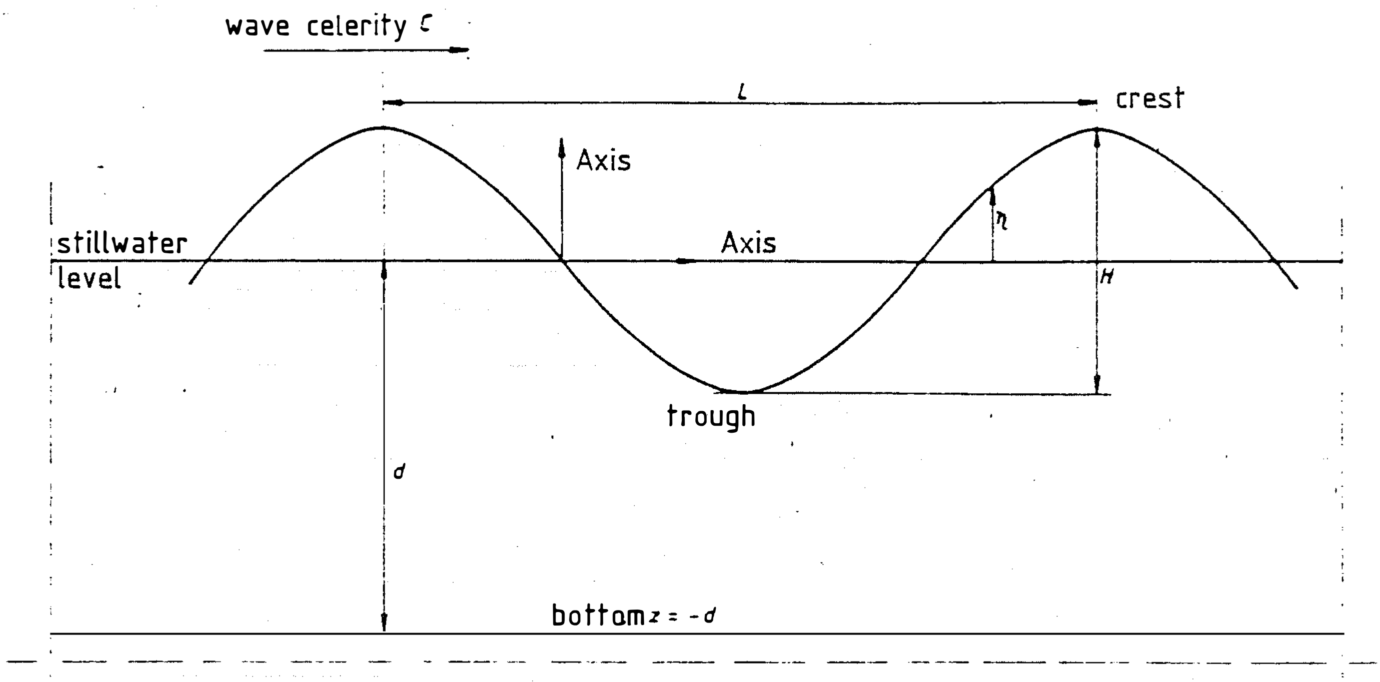


Fig. 21. Profile of a single wave.

Swell differs appreciably from developing sea waves under wind influence. Firstly because the appearance of the waves changes significantly and very quickly after the wind drops or reduces: the white crests have disappeared and the short small waves have been attenuated through internal friction, so that the waves look much smoother. Over longer distances these waves are subjected to two gradual changes, ie loss of height and extensions of the wave period. Because the stability of a concrete block revetment depends strongly on the wave period (see also Chapter 13), the design has to take swell into account in certain cases.

For the prediction of wind waves several methods are available, see the references (21) and (24).

Linear wave theory for small amplitudes

Reference is made in this Guide to the linear wave theory; a broad outline of this theory will, therefore, be given here.

The linear or Airy-Laplace wave theory commences with the following assumptions:-

- sinusoidal surface of the water;
- small amplitudes, ie $H \ll L$, $H \ll d$;
- flat bed
- "ideal" liquid, ie frictionless, incompressible, homogeneous;
- air movement does not influence the wave motion.

It is usual to split relative water depths d/L in the following zones:

- shallow water $\frac{d}{L} \leqslant 0.04$
- transition zone $0.04 < \frac{d}{L} < 0.5$
- deep water $\frac{d}{L} \geqslant 0.5$

The relationship between wave length L, wave period T and celerity C is as follows:-

$$C = \frac{L}{T}$$

This formula is valid in general, independent of wave height or depth of water.

The velocity with which a group of waves progresses is in general not equal to the celerity of the individual waves within the group. While the wave crests move at a celerity C, the whole group moves at a group velocity Cg. Both theory and experience indicate that in deep water the group moves at a velocity which is half that of single wave celerity.

In very shallow water the single wave celerity and the group velocity will become equal. For practical application the important formulae have been summarised in Table 1.

11.2.2 Local characteristics of the individual wave-field

Irregular waves are much more difficult to describe than regular waves. The chaotic character of waves produced by wind is the real feature of those waves. This apparent chaos can only be put into some sort of order by expressing certain phenomena in terms of chances of occurrence, hence it is necessary to make use of probability theory.

The statistical parameters in normal use can be employed to describe wave distribution. In practice for coastal defence works the smaller waves are often neglected, and the average of the highest 1/3 part of the waves is adopted as the significant wave height. This significant height is approximately equal to the wave height estimated by an experienced observer. Its value is commonly indicated by H_s.

A disadvantage of H_s is that it gives only a very approximate description of the total wave-picture. Because many of the rules governing probability processes (stochastic processes) do not have a completely arbitrary character, they can be described by means of theoretical distribution functions.

Wave heights of irregular wind waves can be described, with a reasonable degree of accuracy, by the Rayleigh distribution function (see Fig 22):

$$P_r\ (\underline{H} > H) = \exp\left[-2(H/H_s)^2\right]$$

in which:

$P_r\ (\underline{H} > H)$ = the chance of exceeding the wave height H

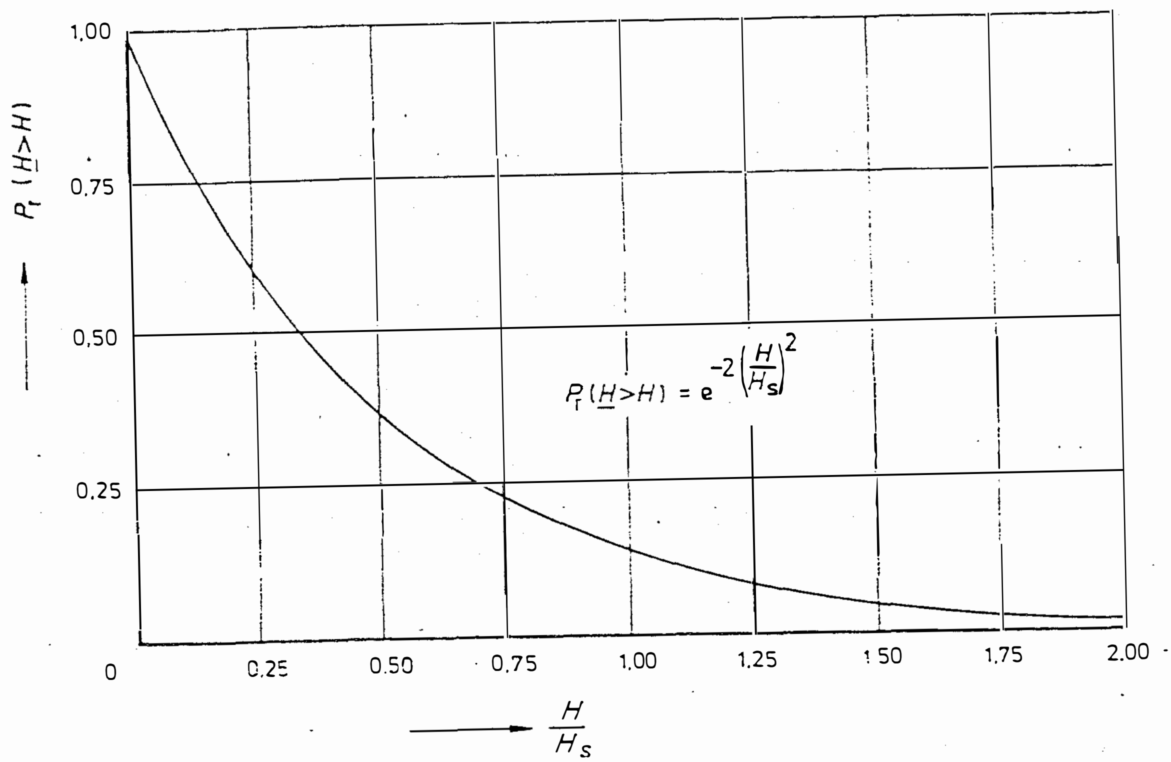


Fig. 22. Curve representing the Rayleigh distribution function.

$\underline{H}$ = individual wave height (stochastic value) (m)

H = individual wave height (m)

H_s = significant wave height (m)

Wave period and length in an irregular wave system are rather more complicated than the height. For practical purposes the period has been defined as the mean period $\overline{T}$, which is the mean value of the time T between two zero-level crossings of the significant wave period T_s, which is the mean value of the highest 1/3 rd part of the waves. For the description of irregular waves use can thus be made of two parameters: firstly the significant wave height H_s and secondly the mean wave period T or the significant wave period T_s.

Another possibility to describe the wave pattern is the wave-spectrum; in contrast to the significant wave-height system, this presents fully the statistical properties of irregular waves. In this method the energy - density E is defined as a function of the frequency f of the spectral components.

11.3 Wave deformations

11.3.1 Wave deformations in front of embankments

Waves as they approach our (Dutch) coast undergo in general substantial deformations before they reach the toe of the bank. This is due to:-

(a) Refraction

When a wave enters water with a different depth, the celerity changes, as do the length and height so that the circumstances, whereby the wave distribution does not coincide with the direction of the depth-change, also change the direction of the waves. One of the most important aspects on which refraction calculations can provide an answer is the question at which locations in front of the coast will wave-peaks meet (and through convergence create increased wave motion in front of the bank.)

(b) Diffraction

Without changing wave length or period, the wave bends around coastal projections or dams. This phenomenon is usually attended and dominated by refraction.

(c) Shoaling

Shoaling of waves denotes the process of transformation of the waves as a result of propagation in regions of varying water depth, where the wave-direction runs parallel to the shoaling. Where the bed level gradually rises the wave length reduces, whilst the wave height increases; the wave-period remains constant.

(d) Bed-friction

This can be neglected in relatively deep water, but near the coast the loss of energy can markedly reduce wave-energy. However, in many situations the influence of this is small in comparison with the phenomena referred to above.

(e) Local wind

In contrast to energy losses through bottom-friction, local winds can add energy. A wave-field approaching the coast can in this way absorb an appreciable amount of energy over, for example, the final 10 kilometres.

(f) Currents

Wave deformations are also caused by changes in currents. Waves have a tendency to converge to locations where their celerity is reduced the most or to diverge to locations where their celerity increases.

(g) Wave breaking

A high-level shore in front of the bank will reduce the wave-attack on the bank to a greater or lesser degree. The solitary wave theory expresses this for very shallow water with a fixed ratio between depth d_b where the waves break and the height of the breaking wave H_b.

$$H_b = 0.78 d_b$$

The depth d_b denotes in this context the vertical distance from the bottom to the wave trough, which for this type of wave with the solitary wave characteristic, nearly coincides with the still-water level at that location.

A formula much used for irregular waves is:

$$H_{max} = 0.5d_b$$

in which:

H_{max} = maximum significant wave height after breaking;
d_b = vertical distance from the bottom to still water level.

A high-level foreshore or sandbank in front of the bank therefore acts as a sort of filter for the waves: only the smaller waves pass through.

11.3.2 Wave-breaking on the bank slope

When waves break on a bank slope, various breaker shapes can be distinguished, depending on wave-steepness and the slope gradient. The breakers will also differ in energy-dissipation and exerted forces.

A characteristic parameter which is used for various breaker types is the wave-breaking parameter:

$$\xi = \frac{\tan\alpha}{\sqrt{\frac{H}{L_o}}}$$

in which:
α = slope gradient
H = wave height
L_o = wave length in deep water

$$= \frac{gT^2}{2\pi}$$

g = gravitational acceleration
T = wave period

The different types of breaker can be roughly classified according to the following table (smooth slope):-

$\xi < 1$	"spilling"	increasing
$1 < \xi < 2.5$	plunging	wave-steepness
$2.5 < \xi < 3.2$	plunging-collapsing	
$3.2 < \xi < 3.4$	collapsing-surging	decreasing
$3.4 < \xi$	surging	slope gradient

The wave-breaking parameter ξ is also used to describe the stability of placed concrete block revetments.

Spilling breaker:

Breakers of the over-foaming type occur on very flat gradients (see Fig 23). These breakers continue running up over a substantial distance, losing energy continuously through breaking with foam-formation at their crests, until these have disappeared altogether.

Plunging breaker:

Breakers of the overturning type (plunging breaker) occur on somewhat steeper bottom gradients and when the development of the wave-crest in shallow water is not disturbed appreciably by other effects such as wind, crossing waves, currents, irregularities on the bottom, etc (see Fig 24). The overturning breaker is characterised by the occurrence of a water-curtain which is as it were "poured down" and detached from the front of the wave-crest. Such breakers have, after turning over, which takes place in a short space of time, only a little residual energy left.

On a comparatively steep bottom gradient the wave deformation process, until final breaking, takes place over a shorter distance. There is thus less chance of disturbing effects than on a flatter bottom gradient, and the steeper gradients have therefore an advantage in general for the development of overturning breakers.

Furthermore, a small initial wave-steepness also encourages the overturning breaker formation, provided that other circumstances do not oppose such action. With a small initial wave steepness the development to a solitary wave continues further and attains eventually a greater height, starting out from a given initial height, so that a greater "lift" of the crest occurs than can be reached from initially steeper waves.

Collapsing breaker

Breakers of the nearly-overturning type (collapsing breaker) come between the "plunging" and the "surging" breaker types (see Fig 25). In this form of breaker a vertical crest exists which has only partially collapsed.

Surging breaker:

Heaving breakers (or surging breakers) can be observed when the bottom or slope gradient is steeper still (and particularly when the wave-steepness is very small) (see Fig 26).

The front of the wave crest is then, as it were, lifted up on the slope before the wave can turn over and a vertical up-and-down water movement is created with a comparatively thin foaming layer of water. The wave energy is now mostly reflected.

If the energy-delivery of the plunging breaker is compared qualitatively with that of the spilling breaker, the picture shown in Fig 27 is obtained. It has to be realised here that in the foregoing diagrams the reality has been substantially simplified.

The considerations presented above are most accurately applied to regular waves which progress towards a flat and smooth slope; a situation which can be achieved in a laboratory. In the natural situation there are many factors which can disturb the picture presented above.

The wave-impact caused by the plunging breaker occurs at a depth which varies between 1/3 H and 2/3 H below still water level. The size of the wave-impact is difficult to determine on a theoretical basis. The reasons for this are:-

- when a wave breaks on a slope, a volume of air is enclosed between the wave-front and the slope. The thinner the air-cushion the greater will be the maximum pressure and the shorter the duration of the impact.

- It is also possible that wave-impact takes place in the return flow of the previous wave. The thicker the layer of returning flow, the smaller the impact will be. The influence of the return flow increases as the slope-angle decreases.

Fig 28 shows a schematic diagram of a plunging breaker. When the revetment has some degree of permeability, the external wave-load forces will be able to penetrate partially into the filter under the revetment, with the pressures underneath having a phase-difference with the pressures on top of the revetment. The characteristics of pressure propagation in the filter are, however, also difficult to model or to describe theoretically.

Much research has been carried out into the wave-impact phenomenon; nobody has as yet succeeded to describe it adequately, so that no theory can be indicated which can be applied properly. In order to get some impression of values, some measured data are presented in Table 2 of the wave impact sizes. These values apply to a smooth impermeable slope. The duration of the pressure peak is in the order of 0.05 to 0.25 second.

Slope Angle	Impact pressure in m water column
1:2	2.3 H
1:3	2.7 H
1:4	2.3 H
1:6	2.0 H

Table 2 - Measured values of the size of the shock pressure at a wave impact

For a description of the wave run-up phenomenon, reference should be made to (22) and (25, part XV).

12 PARTICULAR LOADS

Amongst the particular loadings can be counted:

- wreckage and deck-loads from ships
- drifting ice
- floating debris
- recreation
- growing vegetation and sea-organisms
- grounding of ships or ship-impact

<u>Wreckage and deck-loads from ships</u>

Depending on the dimensions of wreckage and deck-loads, such items when carried by waves can exert great forces on revetments. For normal practice it is, however, often not too bad; it is mostly within acceptable limits so that any damage can be repaired as part of the normal repairs.

<u>Drifting ice</u>

Drifting ice is something which occurs much more frequently in lakes and rivers than along the sea coast, and for lakes and river banks can give rise to damage of embankments.

In order to obtain a reliable basis for the considerations in predicting future ice-conditions, an accurate analysis of existing data is required in relation to the factors which influence ice-formation. The following aspects are important:

(a) Factors which determine water movement

- hydrographic data: topography (surface and depth), exchange of water between sea and estuary;

- hydraulic data: water levels (tides), velocities and tidal capacity.

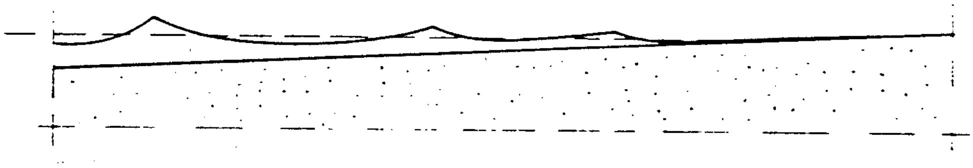


Fig. 23. Shape of the spilling breaker.

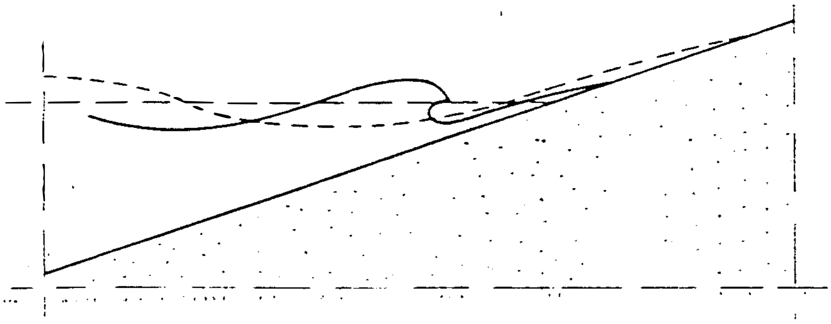


Fig. 24. Shape of the plunging breaker.

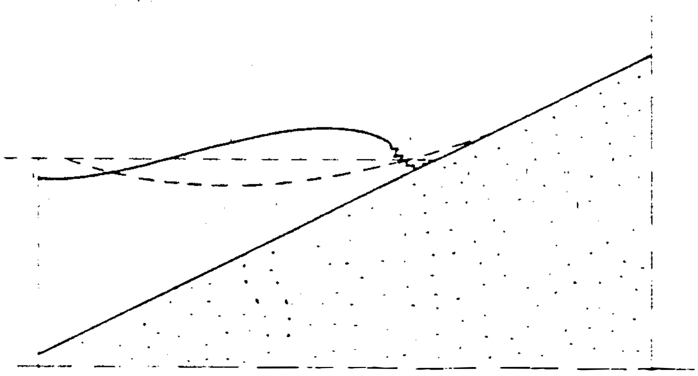


Fig. 25. Shape of the collapsing breaker.

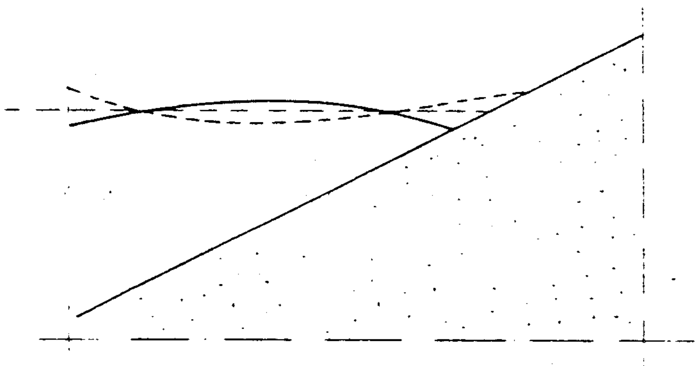


Fig. 26. Shape of the surging breaker.

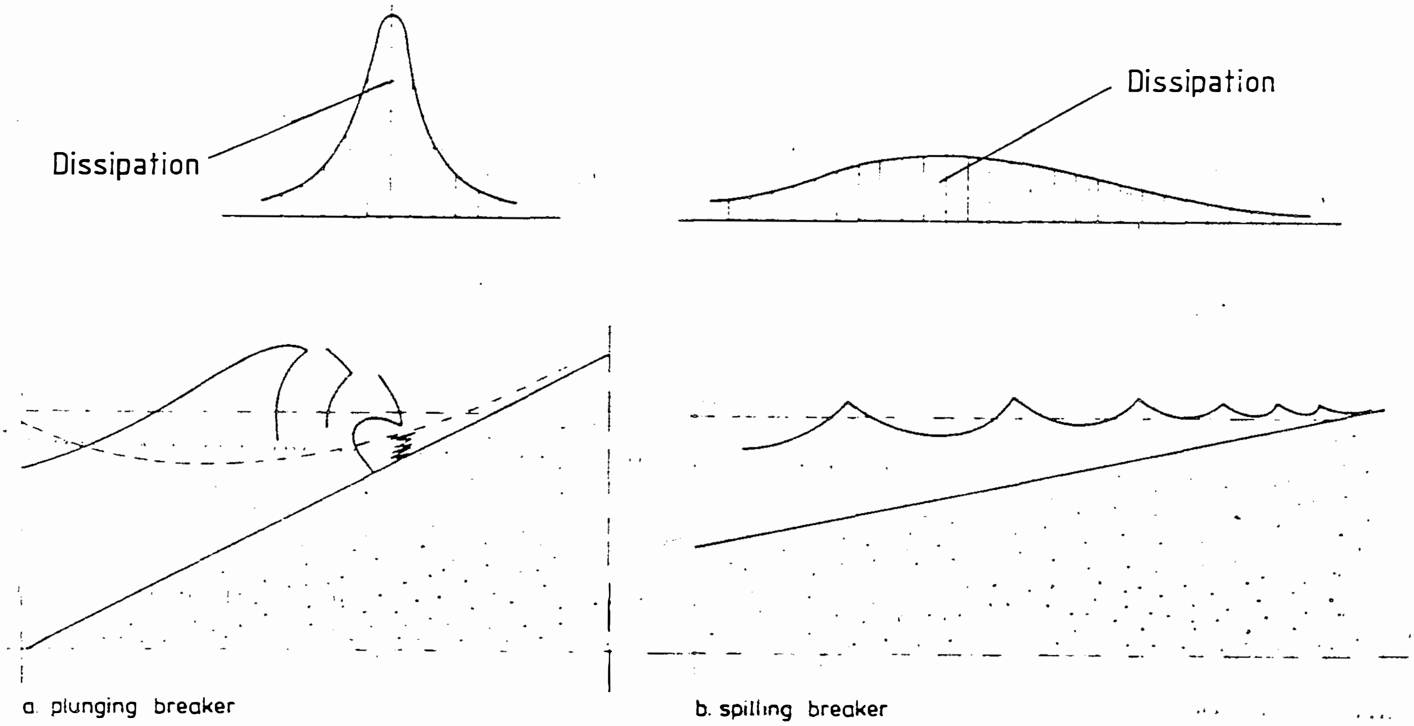


Fig. 27. Comparison of the energy dissipation of the plunging breaker and of the spilling breaker.

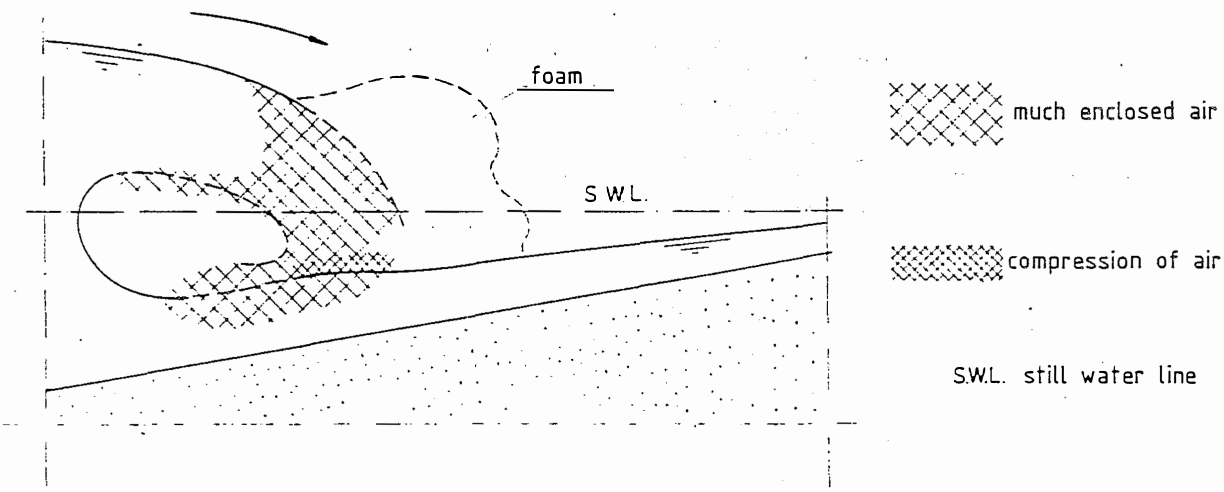


Fig. 28. Diagram illustrating the descent of the breaker tongue.

(b) Factors which influence ice-formation

- climatalogical factors: air and water temperature, wind, etc.,
- water salinity;
- hydrographic factors;
- ice-supply from the river.

(c) Ice-observations

On a lake and in saline water in the shallower areas, an ice-field of great thickness can develop, which can locally increase appreciably where layers drift on top of each other. In general the ice-fields start to move under influence of the wind after thaw sets in and this can cause serious damage to revetments.

In a lake the situation is more serious because the point of attack of the ice-layers on the bank will be at or just below water level, i.e. at the toe-construction and the adjoining lower slope. In this context the strength of the ice is important; saline water ice is usually weaker (softer) than fresh water ice. Parts of the toe-construction including the dumped stone lower slope protection could be picked-up by the ice.

Such ice-fields will not be stopped by bank slopes or obstacles on the slopes. The ice field will fit the water level contour along the slopes accurately and anything protruding above the slope will probably be sheared off.

In this connection it is pointed out that projections from a placed-block revetment, such as rows of piles or raised blocks to curb wave run-up, form points of attack for the large forces which a moving ice-field can exert. As a result of such forces, damage need not be limited to the projecting elements, but can extend to the slope protection by lifting out of elements.

In order to limit ice damage as much as possible it is desirable to finish slopes and their revetments as smooth as possible.

Floating debris

This will in general not cause any damage to hard revetments. Grass mats may suffer where the debris covers it. On rivers damage can be caused by tree-trunks.

Recreation

Concrete revetments in general suffer little damage from this source, but grass mats can suffer from certain forms of recreation. Anglers can occasionally cause damage, by removing a single element.

Vegetation and sea organisms

Vegetation can usually obtain a toe-hold in the joints between concrete blocks. Such vegetation is in general not damaging to the quality of a concrete block revetment.

In the zones which are regularly awash, the growth of algae and barnacles occurs; the higher-order growth occurs especially where areas are not regularly inundated.

Accidents to shipping

A storm has a direct influence on shipping disasters; chances of coincidence are real. This has to be taken into account along busy shipping routes.

In such accidents the contribution of the revetment to the stability of the sea defences against such loads is not large. When there is a demand that the sea-wall should be able to resist such extreme loadings, the main resistance will have to be found in the mass of the sea-wall body.

13 STABILITY OF THE HAND-SET BLOCK REVETMENT

13.1 General

The stability of a revetment comprising placed (concrete) block elements is influenced by several variables, such as:-

(a) the characteristics of the construction (revetment and under-layer):

- weight and/or dimensions of the block elements;
- volumetric mass of the material;
- friction between elements and the under-layer and between the elements themselves;
- compressive forces (pre-stress) in the plane of the revetment;
- interlock between the individual elements;
- permeability of revetment and under-layer;

- slope angle and shape of the slope;

- sand-tightness and erosion resistance of the under-layer;

- influence of transition construction on the strength of the revetment;

- roughness and water-storage capacity of the revetment;

- long term characteristics (durability?).

(b) the hydraulic boundary conditions:

- wave spectrum (wave heights and periods);
- grouping of waves;
- angle of wave attack;
- breaker-type and breaking location;
- wave run-up;
- wind effects;
- wave deformation in front of the sea-wall;
- currents;
- instantaneous water level;
- frequency of occurrence of a specific hydraulics boundary condition on a specific location on the slope.

This summary of variables is probably not complete, but it does indicate the size of the problem. The quantitative effect of many of these variables is as yet insufficiently known.

The failure of a construction or part of a construction occurs if the loading exceeds the strength. As with traditional building construction the question may be direct failure due to excessive loading, and a failure due to medium but frequently occurring loadings (e.g. erosion). The first type of failure is of importance for the revetment, and the second type for the under layer and hence indirectly for the revetment.

The notion of stability has to be viewed for individual elements and for the slope as a whole. The resistance against lifting out a single element relies on its weight, possibly increased by frictional resistances imposed by adjoining blocks. When the frictional forces between elements are large, or if cohesion is obtained in another manner (interlock system), then the stability is determined not by the single element but by the slope protection as a whole.

Where blocks have been placed alongside each other without interlock or jointing materials, the safest assumption is to calculate stability on the basis of the loose block. The first considerations presented here will concern the loose-lying element, after which some thought will be given to the stability of interlock or friction revetments.

13.2 Loose-lying elements

It is necessary to differentiate between elements placed on a permeable or on an impermeable base layer.

13.2.1 Permeable base layer

For the disturbance of the stability of a loose element on a permeable base, present views distinguish about 8 possible mechanisms: (see Fig 29).

(a) When wave run-up has reached its maximum level, the flow returns under influence of gravity. During this return flow, pressures on the revetment reduce. When the revetment is (hydraulically) rough, the return flow can result in current forces, inertia forces and lift forces.

(b) Depending on the permeabilities and the geometry, the water in the filter, which has penetrated through the revetment, cannot flow back directly, which results in forces attempting to lift the revetment. In general the height reached by the wave run-up above still water level is greater than the depth to which the wave withdraws below still-water level. Because of that the water penetrates over a greater surface area into the filter, than the surface area where outflow occurs. This results in a rise of the phreatic line in the filter and through that a rise in pressures underneath the hand placed revetment. This effect is cumulative for a number of waves.

(c) When the following wave arrives at the slope, the pressures on the slope increase under this wave. These pressures can be transmitted through the filter immediately in advance of the wave front, again producing uplift pressures under the revetment. Such pressures will only occur over a limited area adjoining the wave front.

(d) Following the phase given under item (c), there are also substantial changes in the velocity-field due to the approaching wave. The stream lines become curved, comparable with eddies. As a result of the curved velocity-field the pressures above the revetment can be reduced.

(e) A breaking wave will produce wave-impact on the slope, with strongly rising pressures with a duration of an approximate order of ¼ to 1/20 of a second. These pressures on the revetment can be transmitted through the filter and cause short-duration pressures under the revetment.

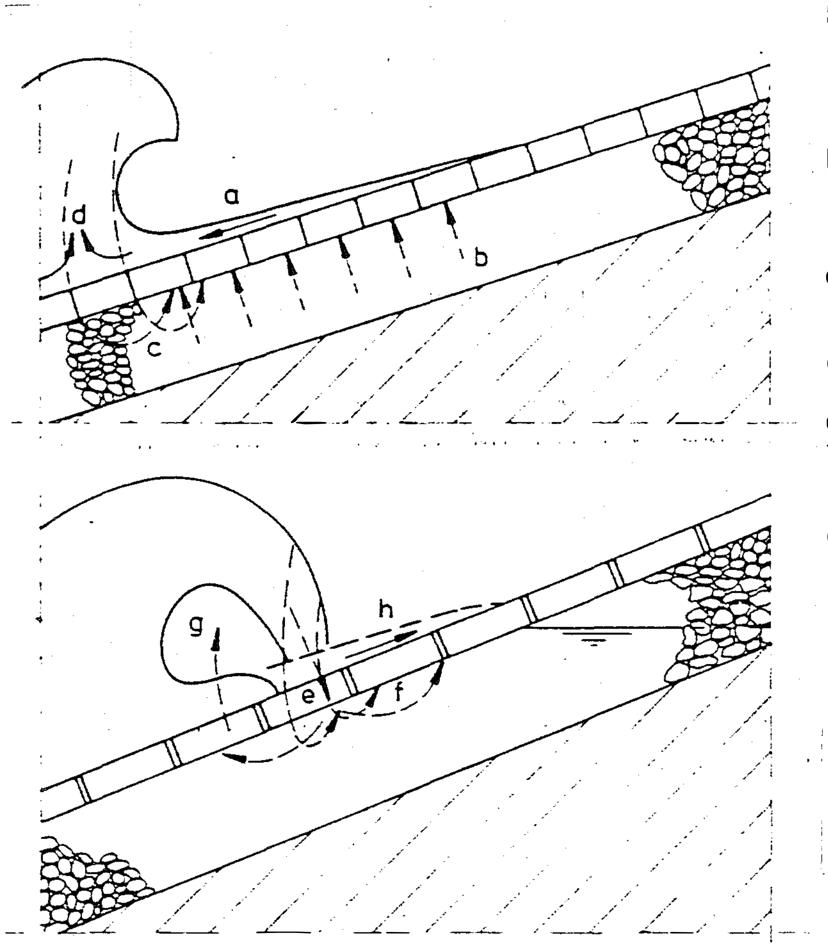
(f) After this short-lived phenomenon, a large mass of water falls on the slope. The high pressures can be transmitted under the revetment, just in front of the point where the wave breaks, and thus result in uplift pressures under the revetment.

(g) After the wave makes contact with the slope, a large reduction occurs in the pressures on the revetment (even negative pressures, ie below atmospheric). This phenomenon is explained as being caused through oscillations of the air "cylinder" enclosed in the breaking wave.

(h) After the wave breaks, run-up takes place. During this last phase pressures on the revetment increase. There are at that stage, however, no critical circumstances in existence; except when the revetment is rough or when blocks are partly lifted out of the revetment. In these cases current forces, inertia forces and lifting forces occur.

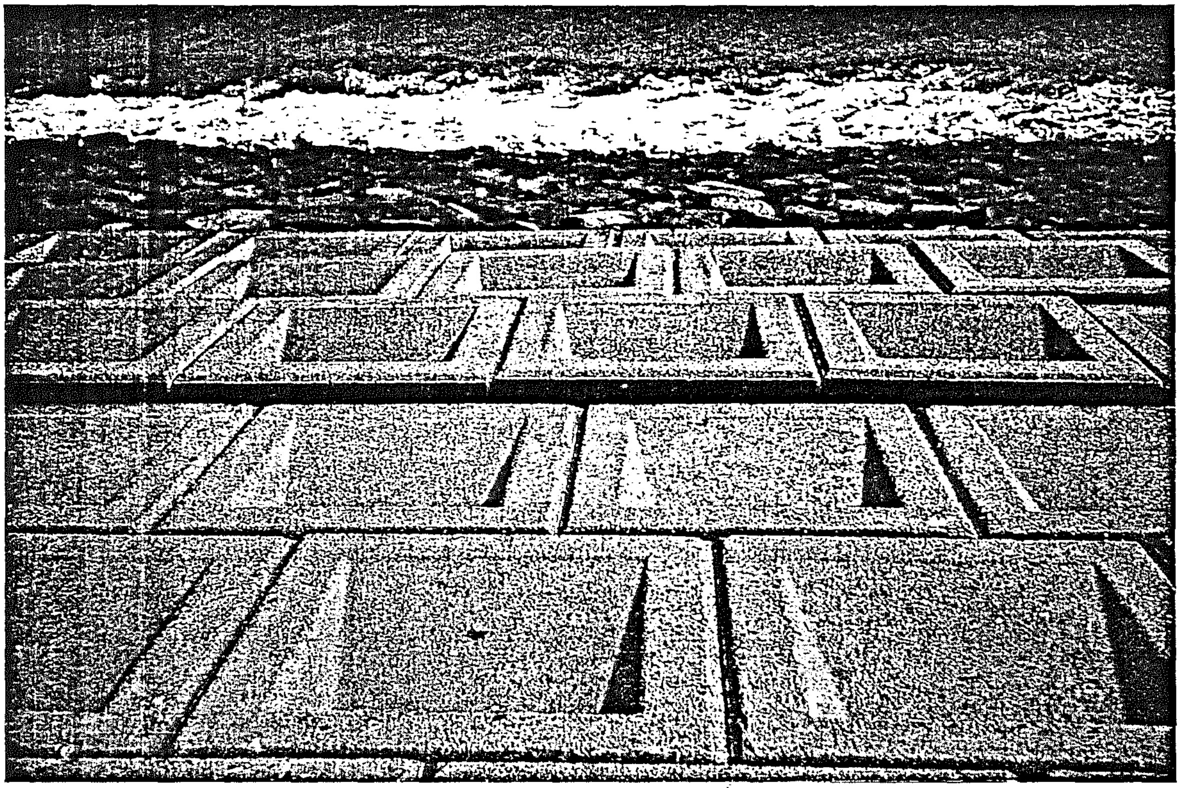
It can be concluded from research carried out at the Delft Hydraulic Laboratory and the Delft Soil Mechanics Laboratory that, under the circumstances prevailing during the tests, the failure-mechanisms (b) "quasi-stationary pressure-differences" and (c) "pressures due to the approaching wave-front" are very important for the stability of a block revetment. The mechanisms discussed can, however, not be viewed separately; a combination of mechanisms can occur. For example, both wave-characteristics as well as slope angle will have influence on the importance of the various mechanisms.

Some examples of computer calculations for mechanisms (b) and (c) have been illustrated in Fig 30. For the calculation the time was stopped at the instant of wave breaking, whilst the phreatic line is near the still-water level. To the left of the wave-breaking point the excess pressure of the breaking wave pushes water through the revetment into the filter (mechanism c). On the right-hand side flow takes place from the elevated phreatic level (mechanism b). Both mechanisms influence each other. For the revetment stability it is more important for both mechanisms (b) and (c) to occur together than for each to occur separately.



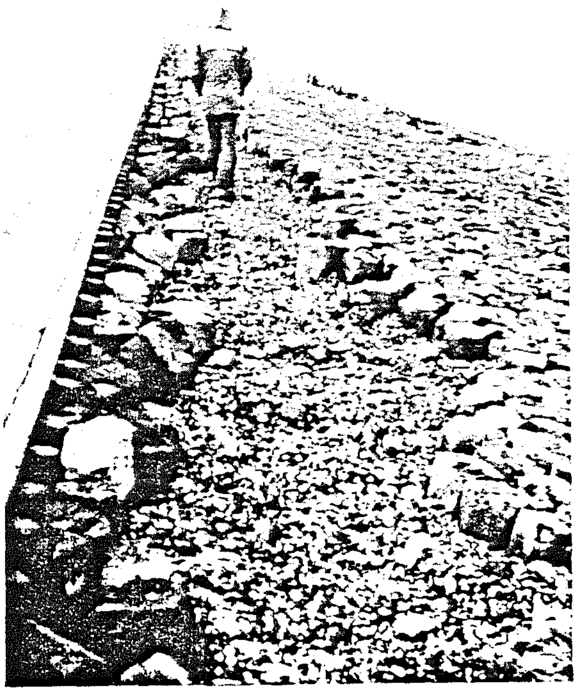
a = forces due to wave run-back
b = quasi-stationary pressures from within the filter
c = pressures due to the approaching wave front
d = changes in the velocity field
e = wave impact
f = pressures due to the mass of the breaking wave
g = low pressures due to air escaping from the enclosed cylinder of air
h = forces due to wave run-up

Fig. 29. Schematic representation of the possible failure mechanisms.



Row of units forced upwards by pressure from within the filter.

Pattern of damage associated with individual stability of the units.



Damage at the transition from a basalt to a concrete slope.

From the differential pressures (pressure above revetment minus pressure below revetment) shown in Fig 30, it is clear that the revetment immediately to the right of the wave-breaking point is potentially unstable, which means that the differential pressure is greater than the weight of the revetment.

The mathematical approach illustrated here is, however, too cumbersome for practical purposes; for further information one should consult the CUR/VB/COW report of Ref 32. "Background to the guide on concrete bank-revetments".

The foregoing considerations make it clear that it is not a simple matter to produce a program for an analytical computer model, which can be used simply in practice, for all possible failure mechanisms; this has so far only been reasonably successful for mechanisms (b) and (c), using the necessary schematical representations.

The analytical computer model indicated that the permeability ratios between the revetment and its underlying filter are of great importance. This is expressed in the seepage-length λ:-

$$\lambda = \sin\alpha \sqrt{\frac{kbd}{k'}},$$

where: α = angle of the revetment slope
k = permeability of the filter layer
k' = permeability of the block revetment
b = thickness of the filter layer
d = thickness of the concrete revetment

The working-out of the computer model for mechanism (b) provides a relationship between the maximum difference in water pressure-level below and above the block work $\Delta\Phi$ and the vertical water level variation H. An indicative example of this is given in Fig 31. It can be seen from this figure that a permeable revetment (k' is large) and a dense filter (k is small) have a beneficial influence on the stability ($\Delta\Phi$ is small). Also, a relatively thin filter layer (b is small) produces less high pressures under the blocks. Corresponding conclusions are valid for the simultaneous occurrence of mechanisms (b) and (c). These qualitative results correspond with the results of the tests carried out by the Delft Hydraulic Laboratory.

The various mathematical models which have been developed only offer application on a limited scale for the calculation of the weight of a revetment. For the time being one has to use mainly the results obtained from laboratory tests.

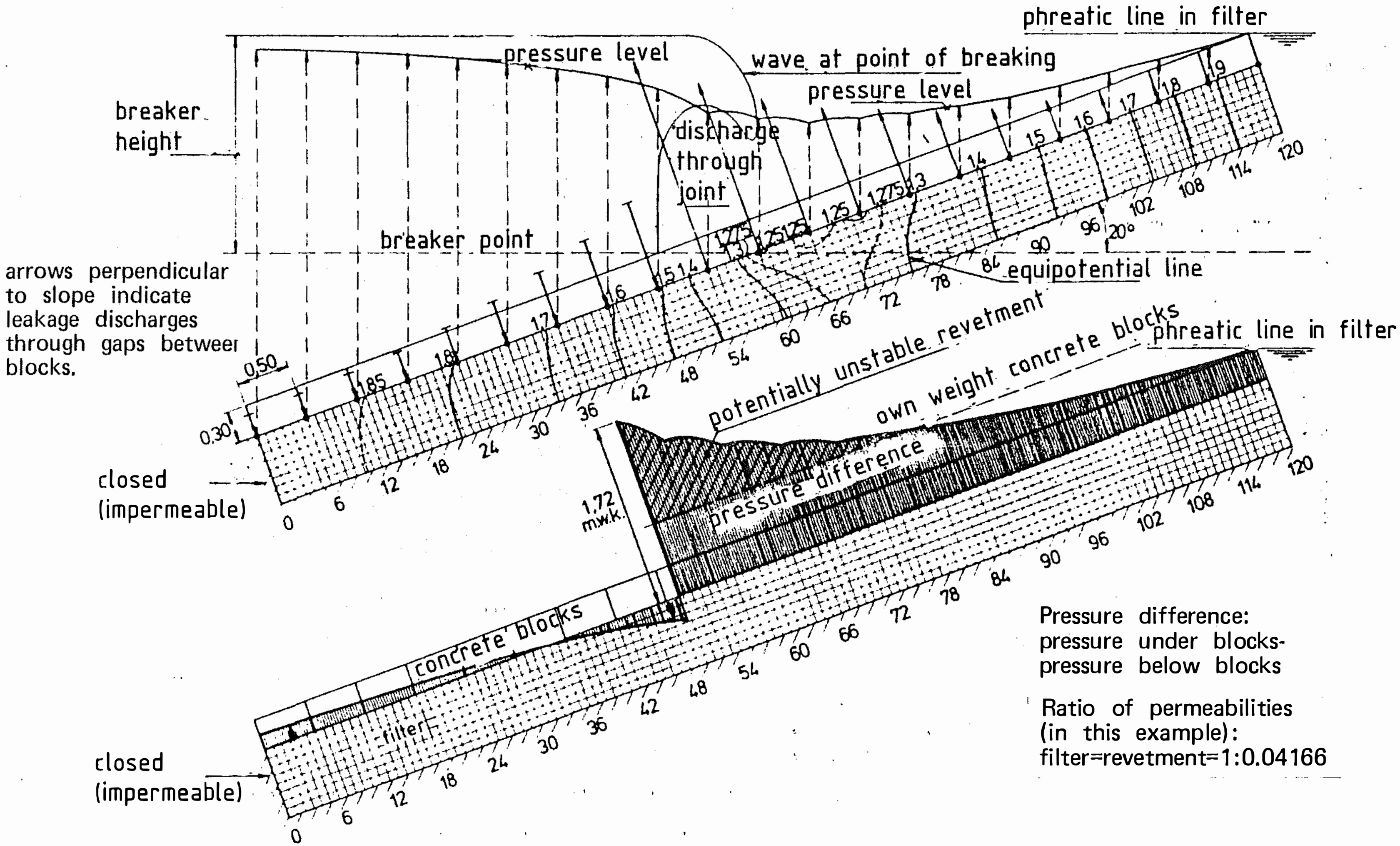


Fig. 30. Results of a computer analysis for the mechanisms b and c at the instant when the wave breaks.

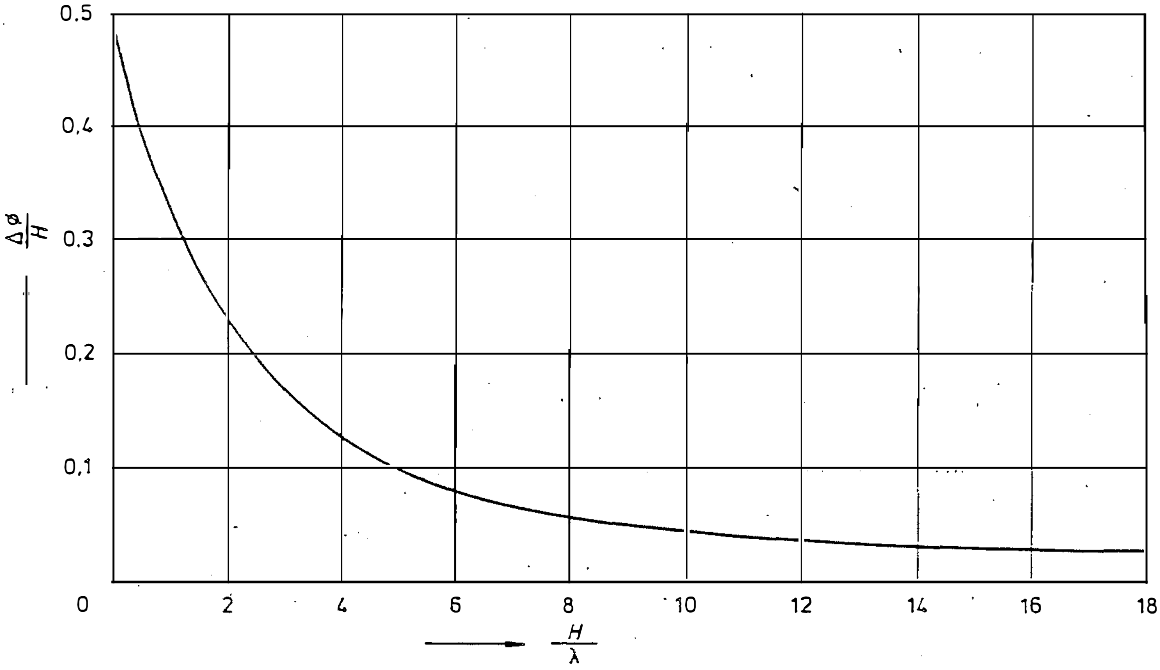


Fig. 31. Maximum difference between the water head below and above the revetment $\Delta\Phi$ as a function of the seepage length λ.

Through the application of the combined research data referred to in the literature, an attempt can be made to arrange an empirical formula. This can be achieved by using a much simplified equilibrium examination to derive a formula and then check the coefficients employed against the test-results.

An example of this is given in Fig 32, where the wave breaking parameter ξ (see also para 11.3) is set out as a function of parameter $H_s/\Delta d$. The equilibrium consideration at right-angles to the slope gives the formula:

$$\frac{Hs}{\Delta d} = \frac{\cos\alpha}{c\,\xi}$$

where: Hs = significant wave height

α = slope angle of revetment

Δ = relative density of a block $\dfrac{\rho_b - \rho_w}{\rho_w}$

ρ_b = volumetric mass of the block

ρ_w = volumetric mass of water

d = thickness of the block

ξ = wave breaking parameter $= \dfrac{\tan\alpha}{\sqrt{\dfrac{Hs}{L_o}}}$

L_o = wave length in deep water; based on the significant wave period

c = coefficient to be checked against test results

With the assumption that $\cos\alpha$ is approximately equal to one ($\cos\alpha \approx 1$), Fig 32 represents a somewhat 'safe' approach with $c = 0.25$. The influence of the wave breaking parameter ξ on the stability of the placed blocks is clearly indicated.

Apart from the thickness d of the revetment, the relative density Δ of the blocks is also of great importance for the stability. Through a relatively limited increase of the volumetric mass ρ_b, the under-water weight of the blocks increases markedly.

The "black-box" approximation presented here is fairly "rough" only; thus for example the effect of the permeability ratio of revetment to filter is not accounted for, nor are the surface block dimensions or the thickness of the filter.

13.2.2 Impermeable underlayer

From tests carried out by the Delft Hydraulic Laboratory for the design of the Oesterdam, in the "delta" flume, it appeared that a revetment of blocks placed on a clay base layer possesses greater stability than is the case for block revetments on permeable base soils. The explanation is that the

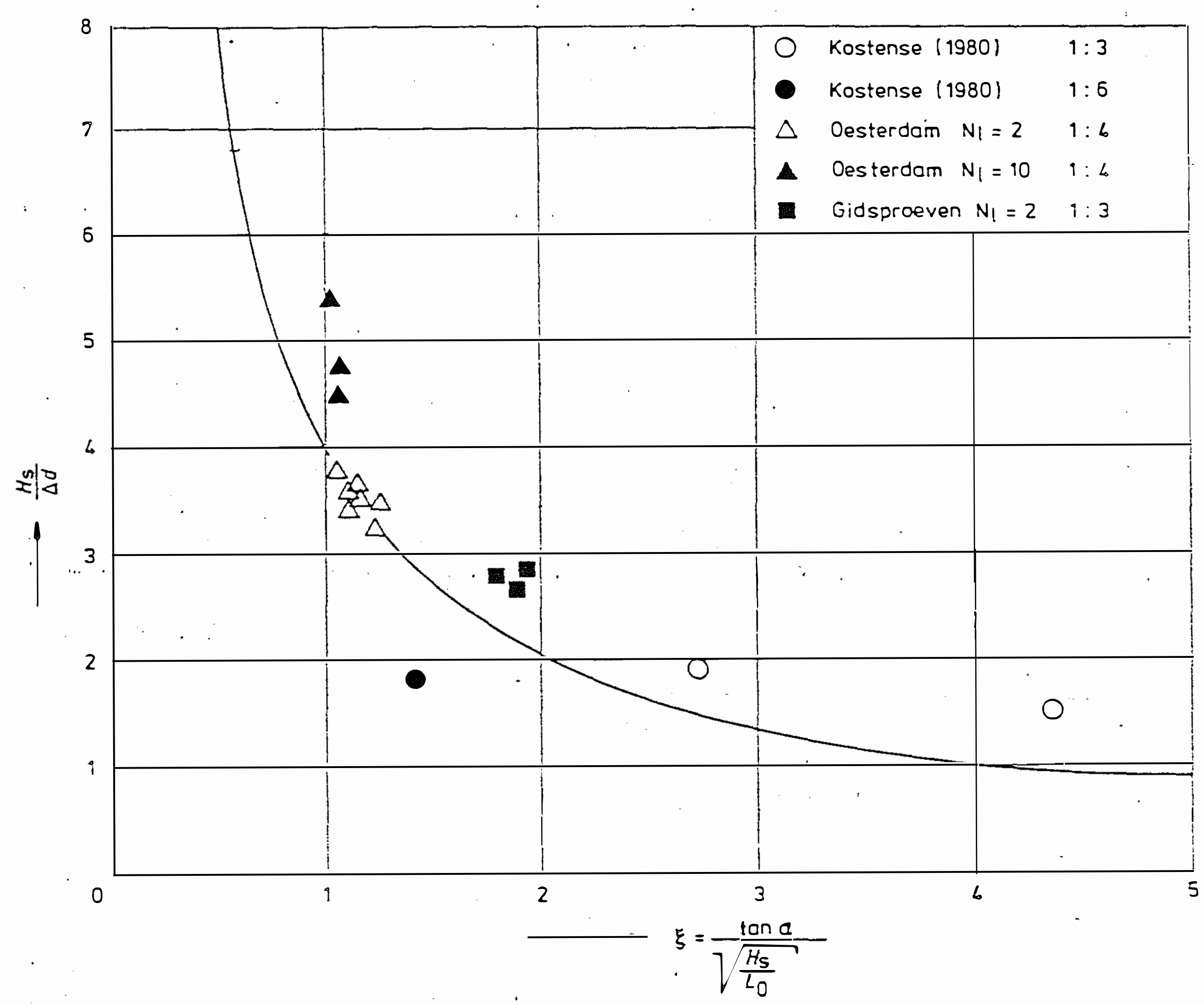


Fig. 32. Wave breaking parameter ξ as a function of the parameter $H_s/\Delta d$ for irregular waves and a permeable foundation layer.

pressure build-up under the revetment is made more difficult by the impermeable character of the clay. As soon as a block is lifted slightly by uplift pressure, water will flow in under the block to fill the hollow space; this is also less easy with a clay base than with a granular filter underlayer.

However, in order to benefit on a permanent basis from the increased stability, the demand on the clay quality needs to be high (see Chapter 5). The presence of sand-lenses is especially detrimental, whilst application in the tidal zone can be problematical.

Little is known so far about the failure mechanism.

13.3 Interlocked or friction block-revetments (tightly-fitting blocks)

Equilibrium considerations concerning uplift are here not valid for one block, but for a number of blocks. Computations for the stability of a slope as a whole are made more difficult by:-

- the unknown factor of the size of the area which will act more or less as a "slab";
- lack of knowledge on the degree of pressure between the blocks (pre-stressing);
- lack of information on the deformation characteristics of the "slab";
- the indistinct nature of the failure mechanism.

Specially for the placing of revetments which possess a reliable interlock and hence high stability under wave attack, the strength of the other parts of the construction should not be neglected. It is, for example, possible that the permissible waves for the block revetment would develop large pressure variations in the underlying filter. This could cause washing out of material, migration from the filter parallel to the slope and penetration of the sand from the body of the bank could occur, which in the long term could cause large settlement in or undermining of the revetment.

13.4 Summary of the research in the "delta-flume"*

Various large-scale experiments (blocks to scale 1:2) were carried out in the delta-flume of the Delft Hydraulics Laboratory (at De Voorst). Figures 33 and 34 give a summary for regular waves and irregular waves respectively. Translating the results of the tests with regular waves to those with irregular waves is not yet possible.

The figures present the (dimensionless) wave height, which produces the damage, plotted as a function of the wave breaking parameter ξ (corresponding to Fig 32). The revetment type and thickness, the wave period and the slope angle are given in the figure. From the figures it can be found that:-

- Variation of the permeability of loose blocks on a filter had little effect on the stability. The revetment was, therefore, apparently sufficiently permeable in all cases investigated (see Fig 34).

- Blocks on good clay have greater stability, which needs the additional comment that in short-duration tests the stability increased even more:

  $Hs/\Delta d = 6.9$ (Fig 34)

- Blocks with a well-defined interlock possess a very large stability (see Fig 33 and 34). The behaviour of the underlying filter can, however, be a limiting factor.

With the Armorflex and Basalton blocks the stability is appreciably increased by filling the spaces between the blocks with granular material, which increases frictional forces. It is not even possible to give the exact failure limit as the Delta flume is incapable of producing sufficient force to make the model block-slope fail for this type of revetment.

This research also showed that, when the spaces between the blocks are filled with granular material, the exact form or shape of the blocks is not of great importance. For all the block shapes investigated, the failure limit proved to be beyond the wave flume capacity, due to the filling of spaces with granular material.

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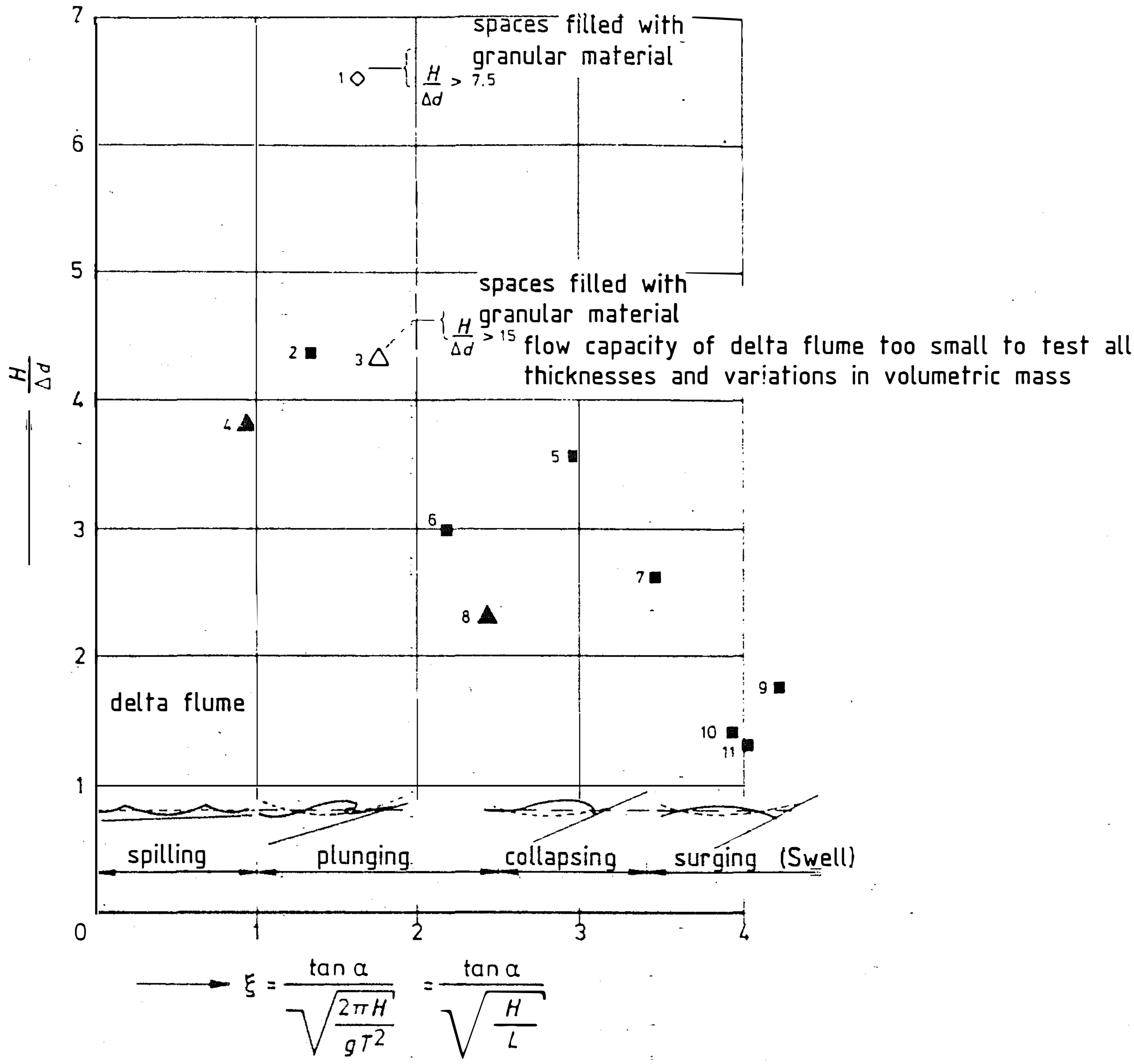
* Translator's note: The Delta-flume, see plate, is a very large wave flume built specially for research work connected with the Delta Project.



1◇ ARMORFLEX 0,33 × 0,30 d= 0,115 tan α= 1/3 T= 4s no infill	4▲ 0.25 × 0.25 d= 0.10 tan α= 1/4 T= 3s	7■ zie 2 T= 6s	10■ zie 2 T= 5s
2■ 0.25 × 0.25 d= 0.15 tan α = 1/3 T= 3s	5■ zie 2 T= 6s	8▲ zie 4 T= 6s	11■ zie 2 T= 5s
3△ BASALTON d prisma's tan α= 1/3 T= 4.4s no infill	6■ zie 2 T= 4s	9■ zie 2 T= 6s	slope angle tan α

Basalton= concrete with basalt aggregate

Fig. 33. Stability values of various block revetments on a filter base, for regular waves.

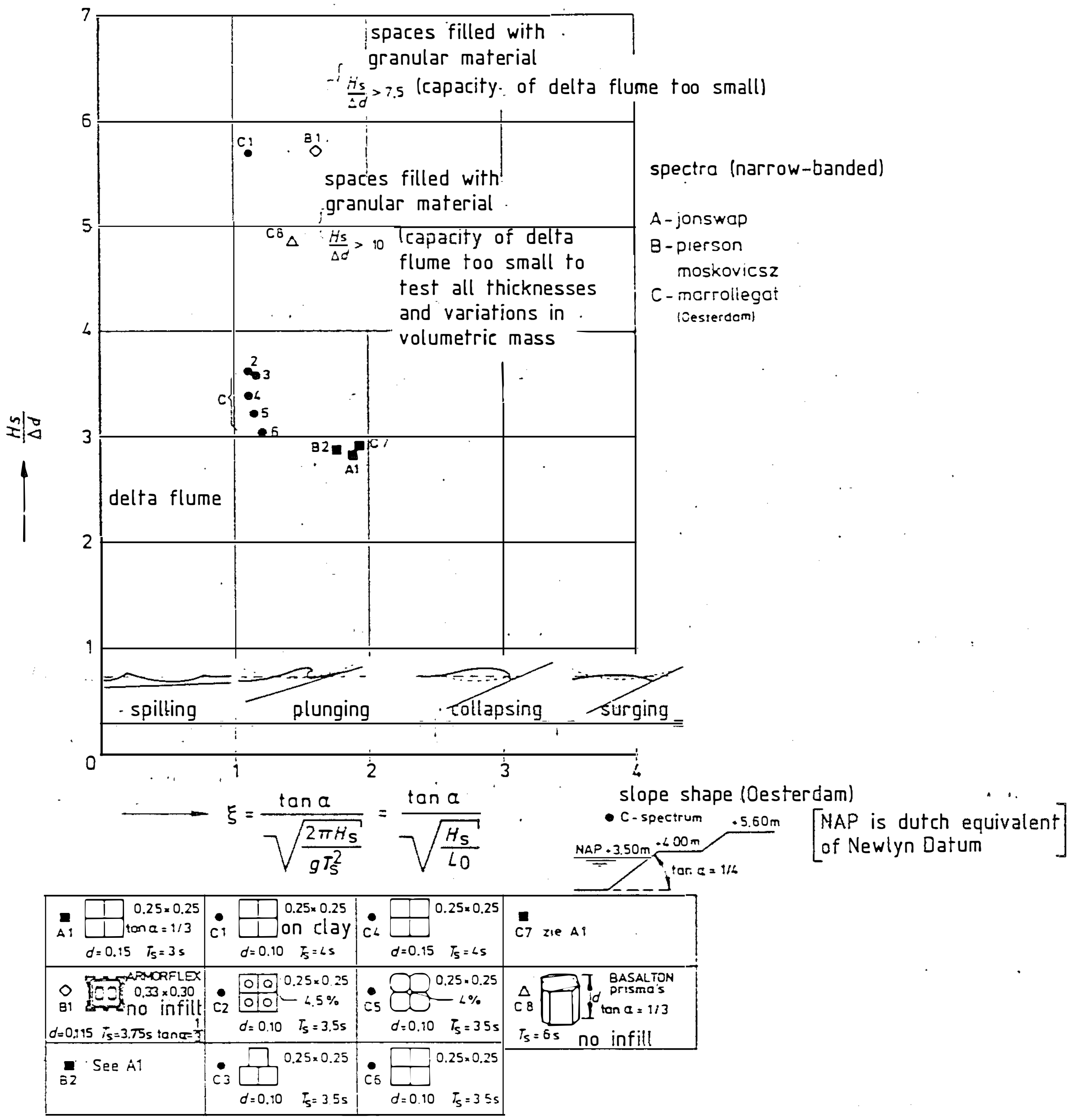


Fig. 34. Stability values of various block revetments on a filter base, for irregular waves (except test C1, blocks placed on clay).

Model investigation into the stability of dyke revetment of stone pitching in the delta flume of the Laboratory of Fluid Mechanics at De Voorst, (Netherlands). The photograph shows a breaker just about to strike the slope.

Although the stability of the blocks in the graphs have been given as a function of two, dimensionless, parameters, it is nevertheless not permissible to extrapolate the results shown, to areas which are clearly beyond the measured values presented in the figures, such as for example a different slope angle.

The cause for this is that the parameters $H/\Delta d$ and ξ only very partially represent the failure mechanism. Some caution in the application of the results of the Delta flume tests is therefore essential.

For Dutch estuaries (without taking swell into account), the wave steepness (H_s/L_o) is often found to vary between 0.035 and 0.065. When an average value of 0.05 is selected for H_s/L_o, then the ξ values are sharply limited:

$$\begin{aligned} \tan\alpha &= 1/3 \rightarrow \xi = 1.5 \\ \tan\alpha &= \tfrac{1}{4} \rightarrow \xi = 1.1 \\ \tan\alpha &= 1/5 \rightarrow \xi = 0.9 \\ \tan\alpha &= 1/6 \rightarrow \xi = 0.75 \end{aligned}$$

For these ξ values Fig 34 shows for loose blocks on a filter layer no lower value than 3 for $H/\Delta d$. This may therefore be taken as a lower limit.

For loose blocks on clay a minimum value of 5 can be adopted for $H/\Delta d$, provided that the clay criteria described in Chapter 5 have been satisfied.

With systems having a good and trustworthy interlock, higher ($H/\Delta d$) values have been found. The performance of the underlayer can, however, form a limiting factor.

14 SAFETY CONSIDERATIONS

14.1 General

Loads and strength parameters can in general not be predicted in advance as to the precise values they will have in a structure. The loads and the strength of a structure are evidently in practice subject to variations, so that these qualities will have to be taken as stochastic values.

A stochastic quantity is defined as one which is not characterised by a single value, but is described in terms of statistical parameters.

The opposite of a stochastic quantity is the deterministic quantity, of which the value is known with certainty. In practice deterministic quantities for loads and strengths do not occur.

Sometimes the spread of values which occurs is, however, so small that it is considered to be justifiable to take the quantity concerned as a deterministic one. Like the loads and strengths, the dimensions of structures are also stochastic.

Practice shows that most of the failure cases on record are not caused by stochastic variations in strength and/or loading, but by constructional deficiencies and human errors. In this there is the chance of calculation errors in the strength computation, or not recognising a major failure mechanism.

Many failures occur during construction, whilst errors during execution in general form a source for possible construction failures. Neglect of the necessary inspection and maintenance can also have serious consequences.

The following discussion is only intended to highlight the philosophy and possibilities of some fairly recently developed methods, with a final word about the levels of safety to be adopted.

14.2 Description of probabilistic methods

If the chance-distribution of load and strength is known, then the chance of failure, as contributed by exceeding the strength can be expressed as:

$$Z = R - S$$

where: Z = the reliability function
R = strength of the structure (resistance)
S = the load on the structure

In order to achieve a purpose designed structure the value of Z should be greater than zero; no failure would occur in that case.;

Strength R and load S will in their turn in general be functions of other variables. The reliability function can be written in the form:

$$Z = R(X_1, \ldots, X_k) - S(X_{k+1}, \ldots, X_n)$$

It is usual to denote the variables $X_1, \ldots, X_n$ from the reliability function as "base-variables".

It should be noted that the reliability function Z is in no way to be viewed as a formula; Z may also come out of a complex mathematical model, eg from a computer program.

With bank revetments it is not known in advance what the dominating situation will be, ie whether it will be water level, wave height, wave steepness, etc. If it is viewed from the failure chance of the revetment, then it is often not the waves occurring during the most extreme water levels (super storm) which determine the design criteria, but the waves at a lower level. This is because, although the wave attack at lower levels is less heavy, the chance of a lower level storm is greater than for an extreme level storm. This aspect will only be shown in its true context with the aid of a probabilistic calculation, in which load and strength are treated as inseparable.

The Delta commission (Ref 29) in their design approach for sea and estuary embankments also introduced a stochastic element, but they returned, on practical considerations, to limiting this to indicating only the loading exceedance chance. Primary sea-defences have to be designed, according to the Delta commission, in such a way that they can <u>fully</u> withstand a storm flood with a given (selected) exceedance chance. In practice the sea-wall will often be designed such that, apart from hidden safety factors, the average strength just equals the design load. This means, however, that there is a 50% chance that the embankment fails during the advent of a "design" storm flood, which does not accord with the requirements of the Delta commission.

Fig 35 shows schematically the method employed in the current (Dutch) code of practice for concrete, in which probability distribution is studied in order to look at the likely strength distribution. With the aid of a central safety-coefficient sufficient distance is maintained between the "chance-densities" of loads S and strength R.

The core of the safety considerations is formed by the so-called probabilistic calculations. Generally speaking it means that the chances of failure are determined on the basis of the uncertainties in the loads imposed and in the structural strengths.

A probabilistic calculation in which these uncertainties are included in a purely formal manner, will soon lead to complicated or insoluble mathematical formulations for the chance of failure. Various simplifications have, therefore, been gradually introduced; these simplifications refer to the way in which the uncertainties are incorporated.

To arrange the various possible procedures in a certain order, some four levels of calculation have been distinguished, which vary from fully deterministic to completely probabilistic. These levels are:-

Level 0: A deterministic calculation. Both load and strength are given defined fixed values and the mathematical model is regarded as fact. By means of one general safety coefficient all uncertainties are accounted for.

Level I: A semi-probabilistic calculation. The loads and strengths are based on characteristic values. By means of partial safety coefficients, ie coefficients which refer to separate quantities, the remainder of the uncertainties are accounted for.

Level II: A probabilistic calculation,in which well-defined simplifications have been introduced in the assimilation of stochastic quantities; several methods are available for this.

Level III: A completely probabilistic calculation. The calculation is based totally on the theory of stochastics.

The calculations on levels 0 and I are not really probabilistic, because the result of the calculations does not give a failure chance. The values of the safety coefficients can, however, for the 'standard' problems be derived with a calculation on levels II or III. Implicit with that can be a calculation on level 0 and I with reference to a particular failure chance.

The considerations here (in this report) will, because of the practical possibilities, be limited to probabilistic calculations on level II.

The simplifications which are introduced in level II calculations are primarily aimed at reducing a complex reliability function to a linear function. Subsequently, the distribution of the reliability function is approximated by replacing it with a normal distribution, of which the mean and the standard deviation are derived from the corresponding parameters of the base-variables. The introduction of the simplifications in the reliability function can be done in various ways; two main methods are employed in practice:

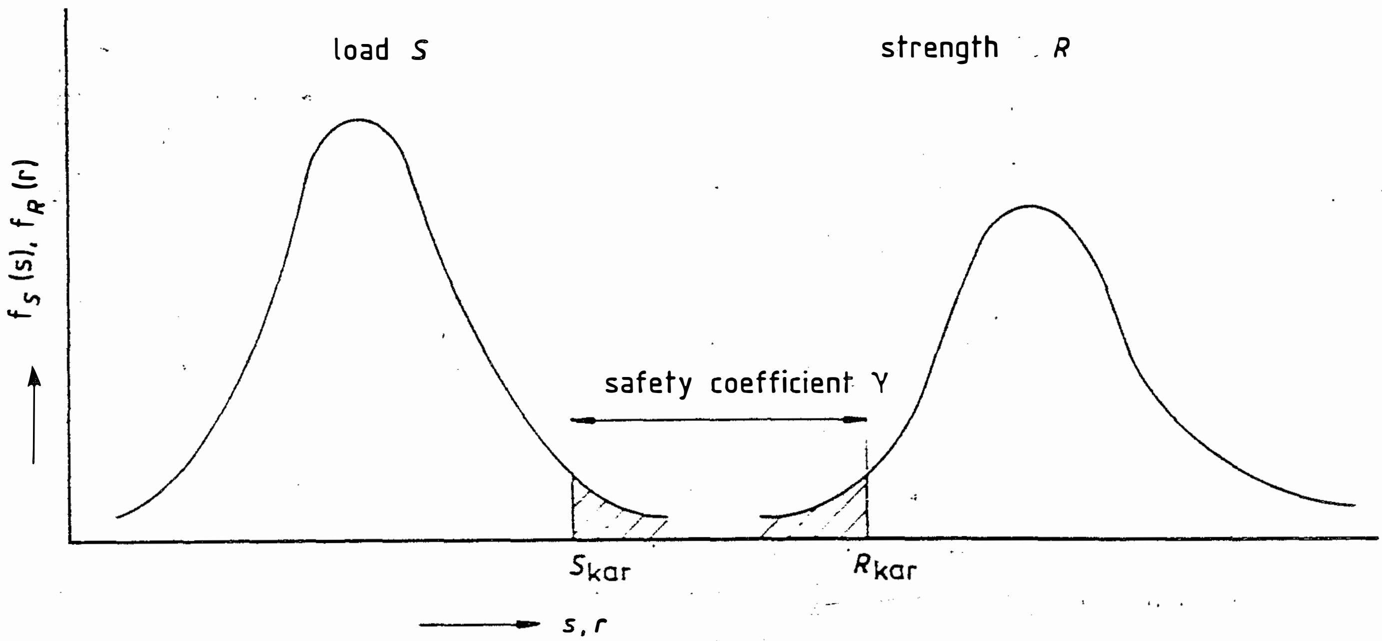


Fig. 35. Design of a structure in accordance with the Code of Practice for Concrete VB 1974/1984 (level I).

- the "mean value" approach;
- the "advanced" approach;

"Mean Value" Approximation

In this method, the reliability function Z, for the mean of the various base variables, is developed in a mathematical (Taylor) series which is terminated after the linear terms.

Linearisation in the mean, signifies in general that it is linearised in a point which does not lie on the failure limit Z = 0. This method is also not insensitive to the manner in which the reliability function is formulated. However, on the other hand it can be said that the "mean value" calculation is relatively simple and can even in many cases be carried out completely by hand.

More complex iterative calculations are required to find a better design "point" eg the following "advanced" approximation.

"Advanced" Approximation

In order to overcome the objections against the "mean value" approximation, improved reliability analyses have been worked out at Level II. The improvement in the "advanced" approximation concerns the choice of the design "condition".

Linearisation no longer occurs in the 'mean' but instead in a point on the failure boundary. On the failure boundary the design condition is in addition chosen in such a way that the probability of occurrence of that value of Z is as large as possible. Where the base variables deviate from the normal distribution, care is taken to ensure that the replacement normal distribution has the same chance-density and exceedance chance. The definitive design-point is determined by means of an iterative method. One consequence of the iterative process is that the calculation can in general no longer be carried out by hand.

14.3 Load and strength

For the information of the reliability function Z, it is necessary to know the probability distribution functions and mutual relationships of the base-variables. In this context the following points arise:-

(a) the probability distributions of the high water level;

(b) the relation between wave-height and high water level;

(c) the probability distribution of the wave-steepness;

(d) the wave-breaking criterion for wave-breaking on the foreshore;

(e) the model for describing the stability of a placed revetment;

(f) the probability distribution of the various parameters which determine the strength of the revetment, for example the slope angle, the block-thickness and the clamping-friction between the blocks.

In the CUR-VB/COW report, "Background to the guide for concrete block revetment on banks", (Ref 32) the lead was given for a mathematical model, in which the above points (a) to (f) have been incorporated. With the aid of a number of computed examples (Fig 36) the possibilities for a probabilistic approach have been illustrated.

In a probabilistic calculation the failure chance is calculated as well as the design-condition. The design condition comprises the optimum combination of base-variables to make the chance that Z will be less than zero as large as possible, in other words the failure chance is greatest, for example in relation to the worst water level for failure. This means that the revetment is safer for other water levels.

Information is also obtained on the contribution of every base-variable in the variation of the reliability function Z. In this way it is possible to estimate how sensitive the exact values for the probability calculation of the variable concerned are on the end-result. The introduction of a wave-breaking criterion enables the quantification of the favourable influence the breaking of waves on a high foreshore has on the stability.

Fig 36 illustrates some results of computer calculations using the "advanced" approximation, derived from the aforementioned report of Ref 32. The ordinate of the graph presents the failure chance (or probability) and the abscissa indicates the water level at which failure probability is greatest. The lines on the graph were derived by varying the thickness of the revetment and the depth of the foreshore.

The figure shows the influence of the breaking wave on a high-level foreshore, on the location of damage and the safety against failure. The boundary between breaking waves and waves not breaking on a foreshore is influenced by the water level and the wave height. A particular relationship exists between water level and wave height: a greater wave height is related to higher water levels. These considerations lead to the conclusion (as shown in Fig 36) that one can only indicate areas where waves break and where no waves break; the exact limit cannot be shown.

14.4 Safety level

In order to include the revetment weight in the probabilistic calculation method, the acceptable failure chance has to be established. Between the "daily practice" in the sea-wall construction industry and the design-philosophy laid down in the Delta report there is sometimes a discrepancy with regard to the revetments. The practical method designs sometimes on the basis of experience, ie under actual usage conditions. The Delta report in contrast, starts out from a design storm which lies outside the area of experience. There is a fairly wide gap between the two methods.

At lower levels in practice one finds occasional cases of damage, say a failure chance in the order of 10^{-2}. If this relatively large failure chance is valid for the whole slope, it would mean that the revetment needs to be carried up the slope to a much lower level than is usual at present, because the associated chance of a water level occurring at higher levels is smaller than 10^{-2}.

The Delta commission has, however, given a design water level with an exceedance chance of 10^{-4} for the calculations of the sea-wall top level. The sea-wall has to maintain a complete defence system up to that level.

Between the uplift of a block at this high level and the failure of the sea-wall, there remains the mechanism of progression of erosion of the intermediate and under-layers and the body of the sea-wall itself. It is not known how much additional safety this means.

The starting point has to be that the revetment with a failure chance of 10^{-4} still retains a certain reserve of strength. The accepted total failure chance at the super-storm level, according to the Delta report will then lie in the order of something less than 10^{-4}.

As an illustration, Fig 37 shows the course of the water levels of two storms, I and II; phase

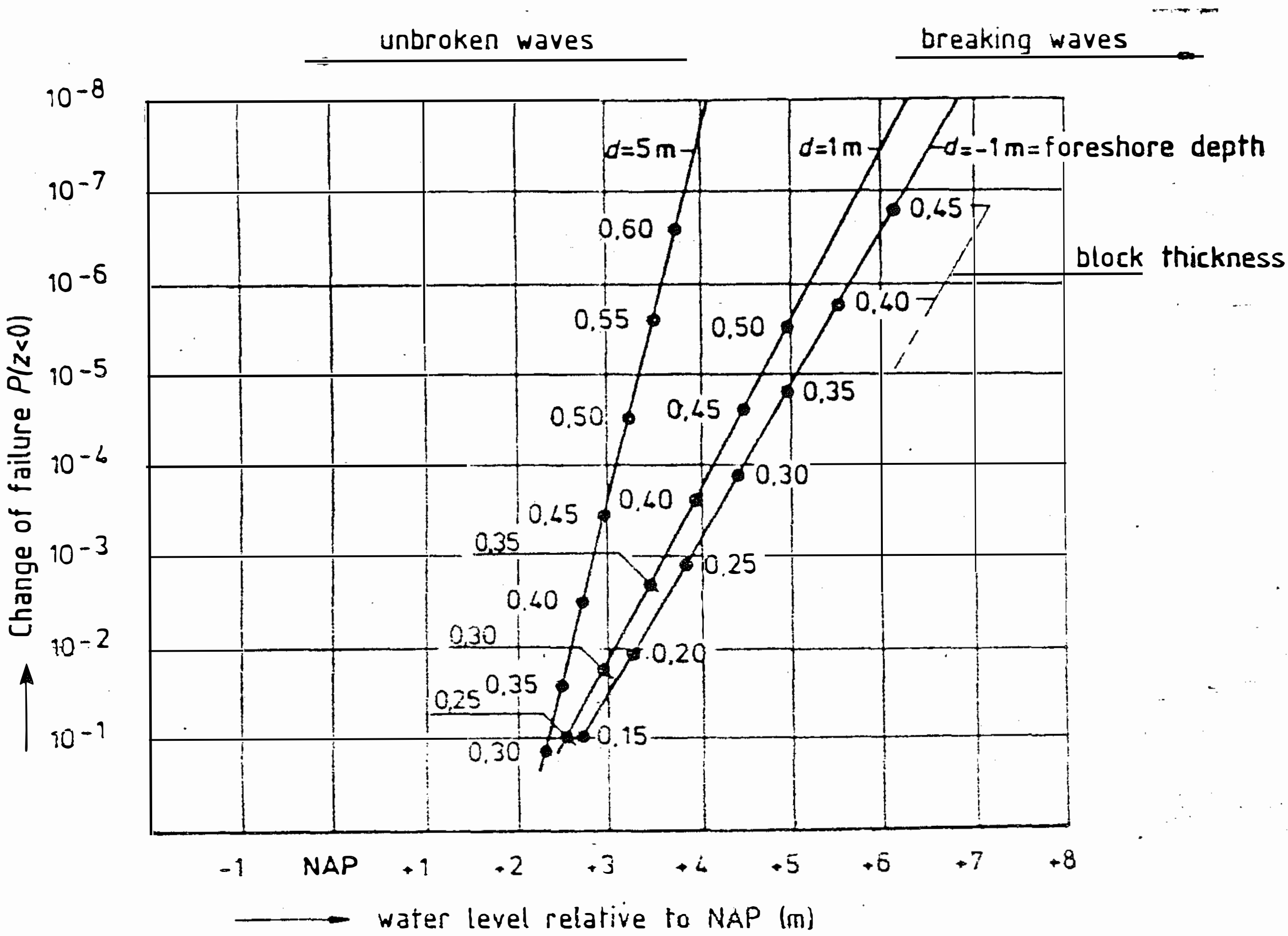


Fig. 36. Example of the results of a probabilistic calculation.

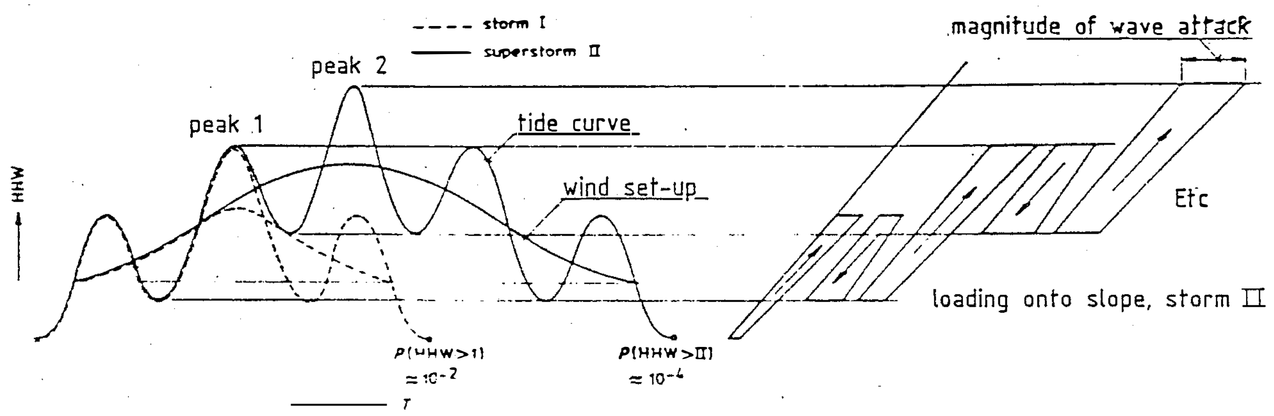


Fig. 37. Water levels and wave attack.

differences between tide and wind set-up have not been taken into account. Storm II lasts longer, reaches a higher water level and produces at its maximum greater waves than storm I.

It is assumed that up to peak I the 2 storms are identical. Storm I has of course a greater probability of occurrence than storm II. Because storm I diminishes in strength after peak I, and the sea-wall at this (comparatively) low level through its great thickness possesses a large residual safety margin, it is perhaps acceptable to have a relatively great chance of failure of the revetment. After the storm the damaged part of the revetment and bank-body can be repaired.

If this argument is pursued, higher failure probabilities would be acceptable for the lower levels. This argument is, however, incorrect.

When storm II occurs there would be no time available to repair the damage caused by peak I. The result is that this storm would, so to speak, "roll-up" the revetment from the bottom upwards, which would create the danger that, through rapid erosion the revetment higher up on the slope would be undermined. This is not acceptable.

The damages occurring at the lower levels would result in damage higher up, despite the constructional safety of the higher level revetment. The strength of the chain is in its weakest link.

It is therefore concluded that at all levels the same failure probability in the order of 10^{-4} should be adopted. In consequence in Holland there should be rarely if ever any serious damage observed to revetment and under layer.

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