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BREAKWATER DESIGN

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Prof.ir. K. d'Angremond

TU Delft

Technische Universiteit Delft

Faculteit der Civiele Techniek
Vakgroep Waterbouwkunde
Sectie Waterbouwkunde

B R E A K W A T E R   D E S I G N

PART ONE

General

1. INTRODUCTION

1.1 Systematic Design

The design of structures off-shore and along the coastline is not essentially different from the design of structures on land like bridges, houses, etc.
When designing a house, it is logical to start with a number of basic questions like the number of rooms required, the purpose and use of several spaces, etc. A qualitative and quantitative analysis of these requirements leads to a preliminary choice of design.
Then, relevant data are collected about natural conditions (rainfall,snow, wind, temperatures etc.), which can influence the design considerably. Also the load conditions are analysed, both for the structure as a whole and for sections of it. Foundation methods are studied and strength calculations are made.
Finally, construction materials are prescribed, construction methods are selected and a cost calculation is made.
A large part of the structual design is based on local experience, often contained in codes of practice.

When analyzing this simple example systematically, one can discern certain elements that form a systematic design procedure:-

- analysis of functional requirements;
- analysis of external conditions;
- choice of functional design;
- strength calculations;
- cost calculations.

The design of coastal or off-shore structures should essentially follow the same systematic line.

It is fair, however, to indicate that the design of off-shore structures is slightly more complicated than the design of a house or a bungalow.

This is mainly due to a few facts. When ordering a house, both, client and architect know approximately what they want. The definition of functional requirements is relatively simple. In the case of off-shore or coastal structures it proves to be more difficult to formulate requirements with a certain accuracy.
This is the more dangerous since over-asking is extremely costly.
Another difference is formed by the structural analysis. Load conditions and strength calculations to be used are not simply prescribed in a code of practice for off-shore structures. Moreover the number of structures built is so small and their design is so much site-specific that generalization is hardly possible. Structural design is only possible on the basis of thorough statistical analysis of boundary conditions and structural behaviour. The varying nature of the external loads makes it likely that design conditions are exceeded during the life time of the structure. In a sound design of an off-shore structure an analysis has to be made of the consequences of such overloading and the possible causes of failure. Recently attention is focussed on such probabilistic design procedures.

For a proper use of probabilistic techniques, insight is required in the functional behaviour of the structure in question. In the lectures on breakwater design it will be attempted to give this insight.

1.2 Functions of a breakwater

The functions of breakwaters and harbour moles can be:

a) Protection against waves	(IJmuiden, Ashdod, Beirut, Scheveningen etc.)
b) Guiding of currents	(Abidjan, N.breakwater Europoc
c) Protection against shoaling	(IJmuiden, Abidjan, Maracaibo Scheveningen)
d) Provision of dock or quay facilities	(Assab, Takoradi, Saba, Antif

Of course, combinations of functions are also possible. Each function, however, leads to certain characteristic requirements for the structure.

Ad a) Protection against waves.
The degree of protection is determined by:-

- lay out;
- permeability of the breakwater;
- crest level;
- energy absorption (i.e. reflection).

Ad b) Guiding of currents.
The qualities as training wall are determined by:-

- permeability;
- crest level;
- roughness (in relation to flow pattern).

Ad c) Shoaling.
The efficiency of protection against shoaling is influenced by:-

- lay out;
- permeability;
- crest level.

Ad d) Dock or quay facilities.
The provisions for dock and quay facilities require:-

- all points mentioned sub a (protection);
- special measures at the lee side.

1.3 Types of breakwaters

Several types of breakwaters can be discerned, depending on their principle of operation and design. The most important types are:

a Rubble mound breakwaters

A structure consisting of one or more layers of loose blocks of natural stone or concrete. The blocks can move with respect to each other and derive their stability mainly from their weight with some additional help by interlocking effects.

The structure is relatively porous, and absorbs therefore a greater part of the wave energy.

The structure is flexible, not sensitive for uneven settlement. It remains functioning even when heavily damaged.

Especially in deeper water rubble mound breakwaters require vast quantities of material.

b Monolithic breakwaters

A monolithic breakwater is a massive structure consisting of a small number of very large elements that are basically immovable with respect to each other.

Such a monolithic breakwater can consist of concrete caissons, (with a vertical, sloping or porous front wall), cellular sheet piling, stacked block walls, etc.

The most general appearance is a vertical front wall, therefore this type of breakwater is often referred to as vertical wall breakwater.

Wave energy is not absorbed but reflected.

The structure is very sensitive to uneven settlement. Damage leads often to a complete destruction and loss of function.

c Composite Breakwaters

Composite breakwaters consist of both, a rubble mound and a monolithic structure in one cross section.

d Floating Breakwaters

A floating breakwater can be either riged or flexible. In general this type of breakwater is cheap, quickly fabricated an thus very well suited to provide temporary protection.

The wave damping characteristics, however, are rather poor, especially in long waves.

e Hydraulic and pneumatic Breakwaters

Hydraulic an pneumatic breakwaters damp the wave action by discharging air or fluid from a submerged porous pipeline. The outflowing medium causes currents, which disturb the orbital movement in the waves and thus initiate the breaking. This system of protection against waves is very energy intensive and not effective in long waves. The method is only feasible for temporary protection.

1.4 Design procedure

Before a preliminary design of a harbour and its breakwaters can be started, it is necessary:

a) To collect all relevant information on the natural conditions.
b) To ascertain the availability and cost of construction materials.
c) To formulate the list of requirements:-
   - for the harbour as a whole;
   - for each of the structures in the harbour.

On the basis of this information preliminary drawings can be made of various alternative designs of each structure. As fas as the breakwaters are concerned, the design conditions should also be determined (i.e. the highest waves that the breakwater shall reach without it being damaged). The selection of design conditions and of the most prospective alternative is based upon the optimum design procedure, in order to establish a structure which involves the minimum total cost.

This design procedure is followed in general in these lecture notes.

2. DATA COLLECTION

2.1 Soil Conditions

Information is required on:-

- bearing capacity of the sub soil;
- grain size } in relation to erosion and scour.
- cohesion }

In general, sufficient information is not readily available. Some information may be obtained from geological maps or files. In a great number of cases a soil mechanical survey will be needed. Applicable methods are:-

- sampling from bore-holes;
- cone penetration tests;
- geophysical methods;
- surface sampling.

2.2 Hydrographic conditions

- Bathymetry:

  In all cases sufficiently detailed information on water depths should be available. Generally, the existing hydrographic maps are not detailed. Possibly the original sounding-sheets can be obtained from the hydrographic office concerned. In any case a comparison of historic and recent maps is worthwhile. Attention should be paid to the location of possible outcrops of rock, wrecks and other isolated objects, which are not shown on the record of the echosounder.

- Tides:

  a Vertical tides

  For vertical tides consult Admiralty or local tide tables and the data sheets of the International Hydrographic Office at Monaco. Local observations should cover an uninterrupted period of at least one month.

All this information yields a reliable assessment of the normal H.W. en L.W. levels, the mean seal level (M.S.L.) etc.

b Horizontal tides

Tidal currents are often referred to as horizontal tides. Information is available on hydrographic maps, in pilots etc.. In some cases currents can be calculated on the basis of observations of the vertical tide. If measurements are carried out they should cover at least a period of 13 hours, both, during spring tide and neap tide.

c Waves

Waves in nature have an irregular character. Each individual wave has a different height and period. Single waves in a wave record are distinguished by the zero up or downcrossing method. (Fig. 2.1)

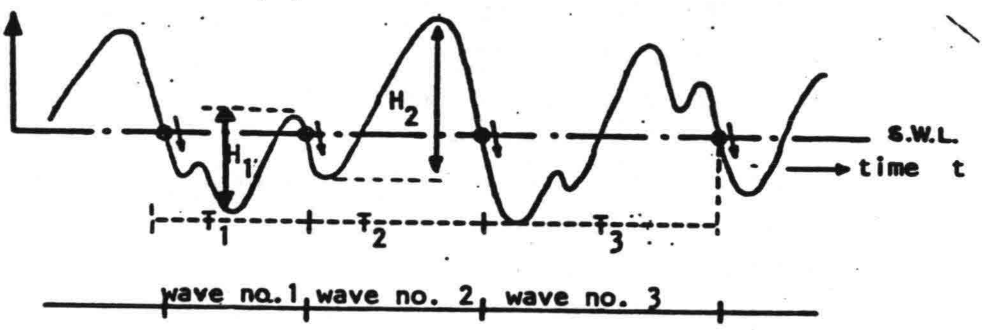

Fig. 2.1

The properties of such a series of irregular waves can be described on the basis of statistical analysis. This statistical analysis makes sense if the number of waves in the record is sufficient to yield reliable statistical information and if the duration of the record is not that long that the wave climate changes considerably during the period of observation. A reasonable wave record counts 100 to 200 waves.

The most common statistical evaluation is the cumulative wave height distribution. This wave height distribution indicates the probability of exceedance of certain wave heights within <u>one</u> storm. When plotting the wave height distribution on Rayleigh paper, (Fig. 2.2) it shows more or less a straight line through the origin. The intensity of the storm can thus be characterized by the slope of the wave height distribution. The steeper this line, the higher the waves. The intensity of the storm can more easily be indicated by the significant wave height H_s, which is the wave height exceeded by 13.5% of all waves.

So far, this analysis only refers to the statistical properties of wave heights within one storm or part thereof. Therefore, the distribution is often called the micro distribution.

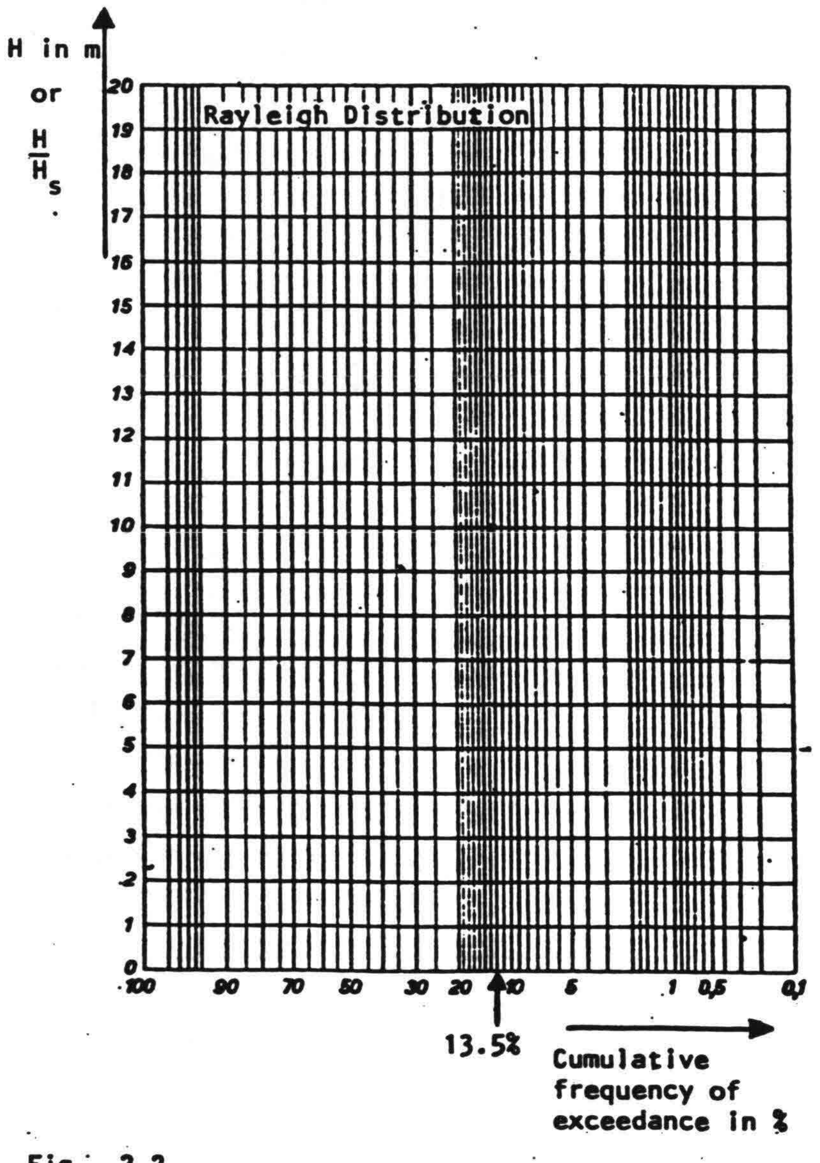

Fig. 2.2

When designing a structure along the shore, also the probability of occurrence of wave heights over a longer period (for instance 100 years) is important. In fact, this is more related to an analysis of storms and their intensity. In this way a macro distribution of wave heights is derived.

From the above, it will be clear that it is most important to have the disposal of long series of wave observations. It is worthwhile to start observations and file the results in a systematic way, even if there are not (yet) concrete plans for a project along the coast.

Standard information on waves may be obtained from wave atlasses, from the standard work "Ocean Wave Statistics" [1] and from shipboard observations collected by the meteorological institutes.
If direct data are not available over a sufficiently long period, waves can be calculated from wind data. Depending on the local conditions refraction and diffraction calculations may be required.

When designing a breakwater, as already mentioned, one is mainly interested in the macro-distribution of wave heights. This refers, however, not only to the rare condition of severe storms, but also to the frequent conditions in relation with the workability of equipment and the exposure during construction phases.

Again, it is emphasized that the macro-distribution is closely related to the frequency distribution of storms, rather than to the heights of individual waves. The intensity of the storm is characterized by one representative wave height: H_s

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In shallow water, the wave heights are restricted, due to the breaking of high waves. In literature, one finds many data for the breaking of individual waves. For considerations of breakwater design one is more interested in the significant wave height in shallow water. A practical rule is:

$$H_s \leqslant 0.5\ d$$

if d is the actual water depth.

Note: Due to wind sep up, the water level may be higher than normal during a storm!

2.3 Meteorological conditions

a Winds

Data on winds are important because winds may act as:-

- generating forces of waves;
- generating forces of storm surges;
- direct forces on ships and structures;
- driving forces of spray.

b Visibility

Visibility is of importance for operational reasons for when the harbour is completed and during the construction phases.

c Other factors

Other factors which may influence the design are precipitation and temperature.

Meteorological data can, in general, be obtained from the national meteorological institutions (airports, agricultural departments etc.)

2.4 Construction materials, equipment, labour

a Materials

The most important construction materials for breakwaters and especially rubble mound breakwaters are rock and concrete.

The following data should in any case be obtained:-

- specific weight or density;
- durability in air and in (sea) water;
- resistance to (mechanical) abrasion;
- maximum size to be obtained in case of rock.

Because the selection and operation of a quarry is a key item in design and construction of breakwaters, a separate chapter is dedicated to this subject.

b Equipment

The selection of equipment for construction is greatly influenced by the design and vice versa. It is therefor necessary to make an inventory of locally available equipment and if necessary to assess the possibilities and impossibilities of mobilizing heavy equipment from elsewhere.

c Labour

Also the availability of local labour will strongly influence design and working methods.

In most cases special facilities are to be provided for the accomodation of personnel. These facilities should be available right from the start of the activities.

Poor working and living conditions will have a negative influence on quality an quantity of the work performed.

3. DEFINITION OF REQUIREMENTS

A complete discussion of all the requirements for a harbour design is beyond the scope of these lectures. Only those requirements which are relevant to the design of breakwaters will be discussed.

Degree of protection:
Depending upon the local circumstances, enormous differences can occur in the protection a breakwater will provide.
In principle, two characteristic cases can be discerned.

a) The breakwater does not protect fixed structures.
b) The breakwater directly protects fixed structures in a harbour basin.

In case a) the breakwater either serves as a training wall for tidal currents or it has the function of improving navigability in the access channel, or it prevents siltation, etc.

In case b) the interior harbour is situated directly behind the moles.

Case a) requires, in general, no more than a low-crested or even a submerged breakwater. The crest level is determined on the basis of an acceptable frequency of hinder to navigation due to wave action.
Note: This criterion applies to sailing vessels, which are not very vulnerable. Moreover, exceedance of certain wave heights does not involve loss of goods.

Case b) requires, in general, a high crested breakwater. In this case the crest level is determined in terms of acceptable wave motion in the interior basin. In this respect two criteria should be considered:-

- acceptable frequency of (un) workability in the harbour basin for small vessels and for seagoing vessels moored along a quay wall;
- damage to the harbour installations.

Note: Moored vessels are rather vulnerable. Exceedance of certain wave heights will cause damage. If not too frequent, (un)loadin can be interrupted during extreme conditions.

Examples of low crested and submerged breakwaters are found in the Netherlands (Europoort, IJmuiden, Scheveningen) and abroad: Tel-Baruch, Israel; Santa Monica Beach, California; Abidjan etc. Examples of high-crested breakwaters can be found along rocky coastlines of the Mediterranean (Ashdod, Gabes, Genova, Beirut) end elsewhere (Taconite Harbour, Crescent City, Santa Cruz).

PART - TWO

Rubble mound breakwaters

4. PROPERTIES OF RUBBLE MOUNDS

4.1 Introduction

The possibility to produce large quantities of durable rock at a reasonable cost has led to the application of rubble mound structures along many coasts. This development started at the end of the 19th centrury.
The first systematic scientific approach to the design of this type of structures dates back to a period just before World War II. It was R.Iribarren, who published in 1938 an article entitled "Une formule pour le calcul des digues en enrochements". [3]. Thereafter many other authors have published articles on the same subject. [4 to 16].

In this chapter a rough theoretical model of the stability of a rubble mound will be presented. Then the stability formulae will be analysed. Finally, attention will be paid to run-up and overtopping of slopes consisting of stone.
Much useful information on design and construction of breakwaters can be found in [2].

4.2 Theoretical considerations on stability

A slope consisting of stones is subject to gravity forces and wave attack. Iribarren considered the equilibrium of a single stone on the slope, where:

W = weight of a stone (Newton)
W_{subm} = weight of a stone when submerged (Newton)
V = volume of a stone (m3)
d = characteristic dimension of the stone (m)
α = angle between slope and horizontal
μ = friction coëfficient
ρ_r = density of rock (kg/m3)
ρ_w = density of water (kg/m3) (note: $\rho_{seawater} \approx 1025$ kg/m3)
H = wave height (m)
F = force exerted by the wave on the stone (directed upward, F_{up} or downward, F_{down}) (Newton)

Considering the downward wave force, equilibrium exists when:

$\mu \cdot W. \cos\alpha > F_{down} + W \sin\alpha$ (Fig. 4.1) (1)

As the stone is submerged, $W = (\rho_r - \rho_w)\, g.\xi.\ d^3$
in which ξ is a shape parameter.

The wave force F cannot be easily calculated because the mechanism of wave attack, often caused by breaking waves, can hardly be described mathematically. If it is assumed (and this assumption is subject to criticism) that

$F = \chi \cdot \rho_w \cdot g.\ H.\ d^2$ (χ is a coefficient)

the equilibrium formula (I) develops into:

$$W_{subm.}\ (\mu\cos\alpha - \sin\alpha) > \chi\, \rho_w \cdot g.\ H.\ d^2 \qquad (2)$$

or

$$(\rho_r - \rho_w)\, g\, \xi\, d^3\, (\mu \cos\alpha - \sin\alpha) > \chi\, \rho_w\, g\, H\, d^2 \qquad (3)$$

or

$$\xi^3\ d^3 > \frac{\chi^3\ H^3}{\Delta^3\ (\mu\cos\alpha - \sin\alpha)^3} \cdot \left(\Delta = \frac{\rho_r - \rho_w}{\rho_w}\right) \qquad (4)$$

$W = \xi.\ \rho_r \cdot g \cdot d^3$

Thus

$$W = \frac{\rho_r \cdot \chi^3/\xi^2 \cdot g \cdot H^3}{\Delta^3\ (\mu\cos\alpha - \sin\alpha)^3} = \frac{N \cdot \rho_r \cdot g \cdot H^3}{\Delta^3\ (\mu\cos\alpha - \sin\alpha)^3} \qquad (5a)$$

If the upward wave forces are considered, (5a) changes into:

$$W = \frac{N\rho_r \cdot g \cdot H^3}{\Delta^3\,(\mu\cos\alpha + \sin\alpha)^3} \qquad (5b)$$

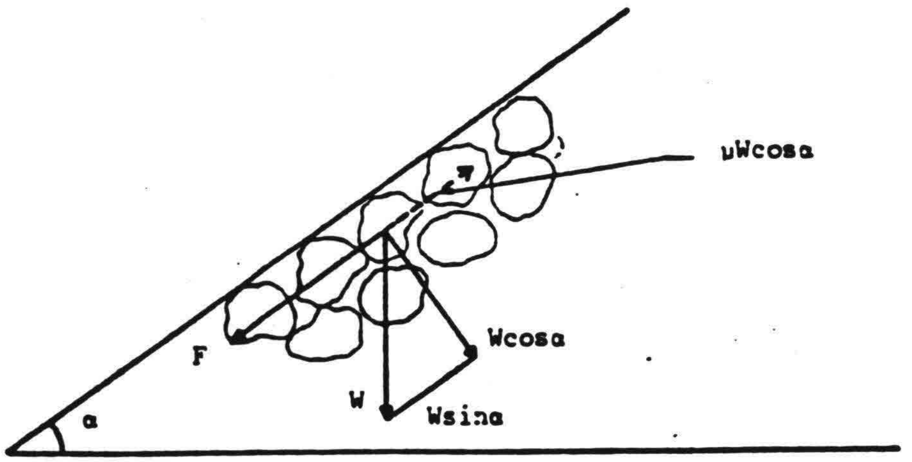

Fig. 4.1 Equilibrium of forces.

These formulae are the same as those given by Iribarren [8] and others [12, 13]

It is clear that ρ_r, N and μ are constants of the material which have to be determined before the formula can be used.

Iribarren did so for the constants of his formula.

Since 1942, systematic investigations into the stability of rubble slopes have also been done at the Waterways Experiment Station, Vicksburg, (Miss.). On the basis of these experiments Hudson proposed the following formula, [14, 15, 16]:

$$W = \frac{\rho_r \cdot g \cdot H^3}{\Delta^3 \cdot K_D \cdot \cot\alpha}$$

The formula is applicable for slopes not steeper than 1 : 1½ and not flatter than 1 : 4

4.3 Comparison of stability formulae

When comparing the Iribarren and the Hudson formulae (but also many others) it appears that no doubt exists about the relation between W, H, ρ_r and Δ. To compare the differing opinions on the influence of material constants (μ and N versus K_D, $(\mu \cos\alpha \pm \sin\alpha)^3$ versus $\cot\alpha$) the stability formulae are written in a different way:

$$\frac{W.\Delta^3}{\rho_r \cdot g \cdot H^3} = \frac{1}{K_D \cot\alpha} \qquad \text{(Hudson)}$$

$$\frac{W.\Delta^3}{\rho_r \cdot g \cdot H^3} = \frac{N}{(\mu \cos\alpha \pm \sin\alpha)^3} \qquad \text{(Iribarren)}$$

A graphical representation for rough angular quarry stone is given in Fig. 4.2

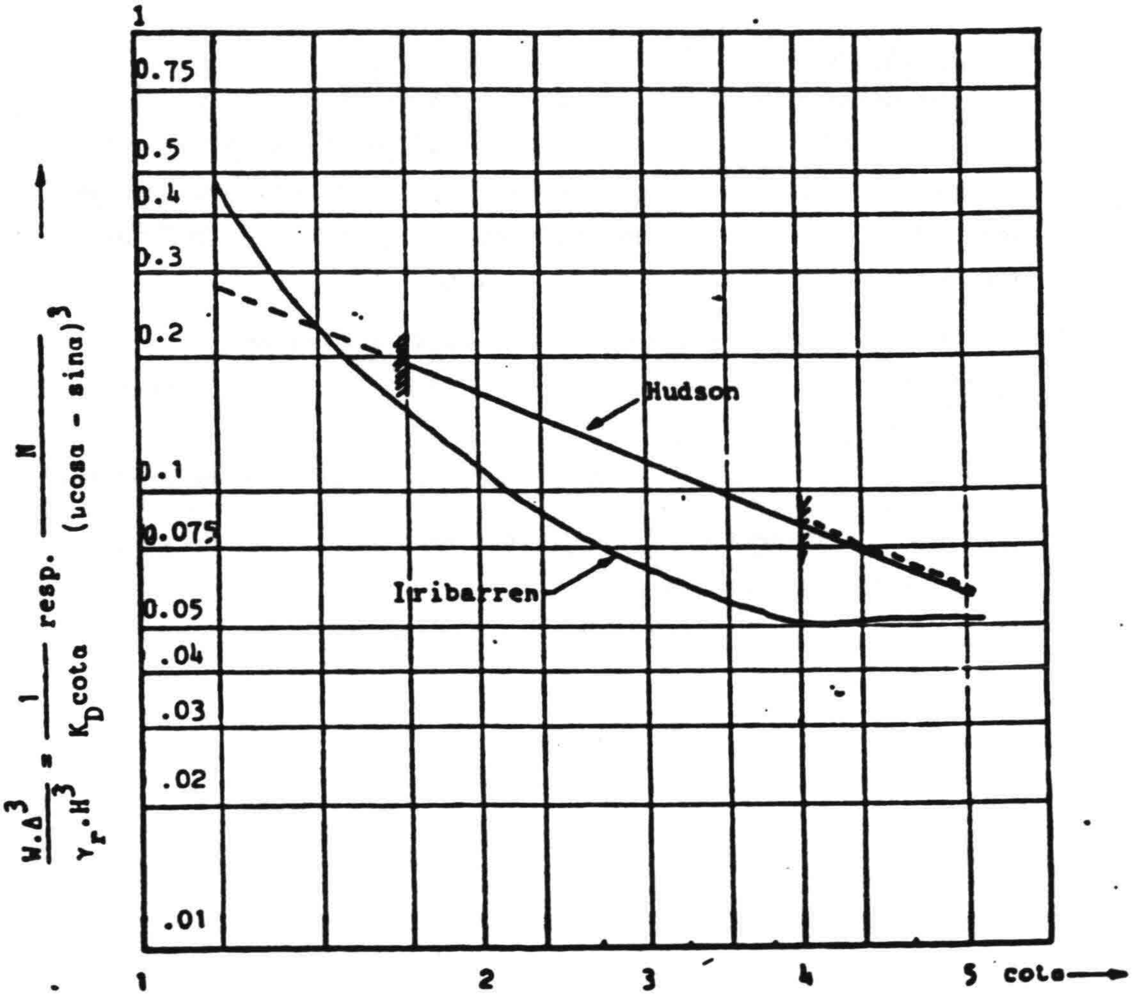

Fig. 4.2 Comparison Hudson/Iribarren.

A careful comparison shows that Iribarren indicates relatively high weights for extremely steep slopes, that the difference in the region of practical slopes (1 : 1½) is relatively small, and that for extremely flat slopes both formulae come together. In the region of 1 : 3 slopes, however, Iribarren gives very (probably too) low weights.

It is also evident that a minor change of the coefficients brings the formulae together. At the same time, this is the reason why the Hudson formula is slightly in the advantage:-

- only one constant is to be determined;
- much more experimental background exists for the Hudson formula.

The Hudson formula can be applied only for slopes between 1 : 1½ and 1 : 4.

4.4 Determination of constants

For the determination of the constants in the formulae of Iribarren and Hudson, data are required on the stability of slopes, in relation to : block weight, slope, wave height etc. These data have never been collected from prototype but only from small-scale tests. The reasons for this are:

1) Difficulty to measure wave heights in prototype.
2) Difficulty to determine the moments of incipient damage in prototype.
3) The (happily) small number of complete failures in prototype.

The constants recommended by Iribarren and Hudson (W.E.S.) are given in Tables 4.1 and 4.2

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Table 4.1 Stability numbers (Iribarren)

Type of stone	Downward stab. $(\mu\cos\alpha - \sin\alpha)^3$		Upward stability $(\mu\cos\alpha + \sin\alpha)^3$		Transition slope between upward and downward stability
	μ	N	μ	N	$\cot\alpha$
rough angular quarry stone	2.38	0.430	2.38	0.849	3.64
cubes	2.83	0.430	2.83	0.918	2.80
tetrapods	3.47	0.656	3.47	1.743	1.77

Table 4.2 Stability numbers (Hudson)

K_D VALUES FOR USE IN DETERMINING ARMOUR UNIT WEIGHT

No - Damage Criteria

Armour Unit	Number of Layers	Placement	K_D Normal cross section	K_D Breakwater head
rough angular quarry stone	1	Random	2.6	2.1
rough angular quarry stone	2	Random	3.7	2.8
rough angular quarry stone	3	Random	4.2	3.8
cube	2	Random	7	5
tetrapod	2	Random	8	6
dolos	2	Random	20	15

It must be realized, that the constants given here are valid for the situation of incipient damage, which is defined as the condition when ca. 1% of the total number of stones from the outer layer is lost. Although this is an important figure, it is essential to know which safety margin exists, i.e. how fast the damage increases with increasing wave height. This information is important because:-

- the accuracy of the design wave height is not great;
- the accuracy of the block weights is not great.

The information is essential if the design is based on the so-called optimum design procedure. (See further chapter 6)

Little is published on this aspect of breakwater stability. The only reliable data can be found in [17, 18, 19] (See also Table 4.3)

Table 4.3

Damage (D) in percent as a function of excess of no damage wave height:

$H_{D=0}$ is the design wave height calculated with K_D ~~from Table 4.2~~

H is the actual wave height

		Damage D. in %						
Armour Unit		0-5	5-10	10-15	15-20	20-30	30-40	40-50
quarry stone	$H/H_{D=0}$	1	1.08	1.19	1.27	1.37	1.47	1.56
akmon	$H/H_{D=0}$	1	1.3	1.45	collapse			
dolos	$H/H_{D=0}$	1	1.1	collapse				

As the constants in the Hudson formula are based on small scale experiments, one should be aware of the risk of scale effects. Recent tests at the Waterways Experiment Station and the Coastal Engineering Research Center indicate that this risk is small as far as stability is concerned [20]. For the permeability of rubble mounds, scale effects play an important role [21] and [21A] . Usual scales for stability tests range from 1 : 25 to 1 : 100.
It is recommended to use only stability figures from the W.E.S. published in [2, 14, 15, 16]. If figures from other sources are used, it must be ascertained that the tests are performed with the same procedure as at the W.E.S.. For special types of stone, reference tests with well-known types of armour units should always be carried out. This is essential because the resulting K_D factor depends on:-

- the way the stone is placed;
- the way H is measured;
- the way ρ and Δ are determined;
- the way the damage is measured;
- the foreshore in the model;
- the wave period;
- etc..

Examples of tests yielding questionably high figures are [22 and 23]. An example of carefully performed comparative tests is [17].
In general tests yielding K_D factors greater than 10 to 12 should not be regarded with too much confidence.
In both formulae, the significant wave height H_S can be used instead of H.

In this respect the utmost care is recommended with the use of dolosses. Recently, serious damage occurred to breakwaters with a dolos cover layer. Although the actual reasons for these failures have not yet been established beyond doubt, the following points emerge from discussion amongst experts:-

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- the mechanical strength of large size dolos is small. Thus breaking of units occurs, rusulting in smaller block weights;
- due to the strong interlocking effect, the mechanism of damage changes. The dolos layer fails eventually when the complete layer moves along the slope. In this way, a slight excess of the design wave height does not simply lead to the loss of a few single blocks, but causes failure of the complete armour layer at once.

4.5 Sensitivity of Hudson formula

The parameters included in the Hudson formula cannot be determined very accurately. Therefore it is good to analyse the influence of a small error in one of the parameters on the final result, i.e. the required block weight:

- Influence of wave height
  a 10% increase of wave height leads to a 33% increase of block weight.
- Influence of the density of the armour unit
  a 10% increase of the density of the armour units reduces the required weight by approcimately 50%.
- Influence of the density of (sea) water
  a 2.5% increase of the density of water (from 1000 kg/m3 to 1025 kg/m3) causes a 13% increase of the required block weight.
- Influence of slope and K_D
  Both parameters have a linear influence.

Apart from the influence of the parameters on the size of the armour units, it is good to realize that the size of the armour units is often estimated from the diameter. An error of 10% in this size leads to a 33% error in the block weight.
This fact illustrates the importance of installing a weighbridge at the quarry to check the weight of the armour stone instead of visually estimating its size.

4.6 Influence of wave period

The wave period has a strong influence on the breaking pattern of waves approaching a structure. The breaking pattern as such influences the character of the wave attack (and the wave run-up). Therefor, influence of the wave period on the stability of armour units must be expected, although this influence is not demonstrated by the Hudson formula nor by the Iribarren formula.
The relevant literature shows no conclusive results. There is a trend that longer wave periods give rise to greater damage and thus require heavier armour units.

It appears also, however, that there is a strong interrelation between depth and shape of the foreshore on one hand and wave length (or period) at the other hand. It is evident that this has also a great influence on the stability of the breakwater.

4.7 Review

The question remains regarding the value of stability formulae in general. In fact, they are nothing more than a primitive tool for the designer to make a preliminary estimate of required block weights. A primitive tool because:-

- the influence of the shape of the foreshore is neglected [24];
- the influence of overtopping is neglected;
- the influence of the wave period and the spectrum is neglected;
- the influence of the construction is neglected.

Therefore, it is recommended always to perform model tests on the final design, if possible in a flume with irregular waves.

4.8 Increase of stability

If a certain stone proves to be unstable on a given slope under a given wave attack, there are several methods to solve this problem.

1) Increase ρ_r
   by selecting a different quarry
   or by producing concrete with heavy aggregates.
2) Increase W (quarry, handling weight!)
3) Decrease slope (be careful; upward stability)
4) Increase K_D by selecting a special shaped block.
5) Grout blocks together.

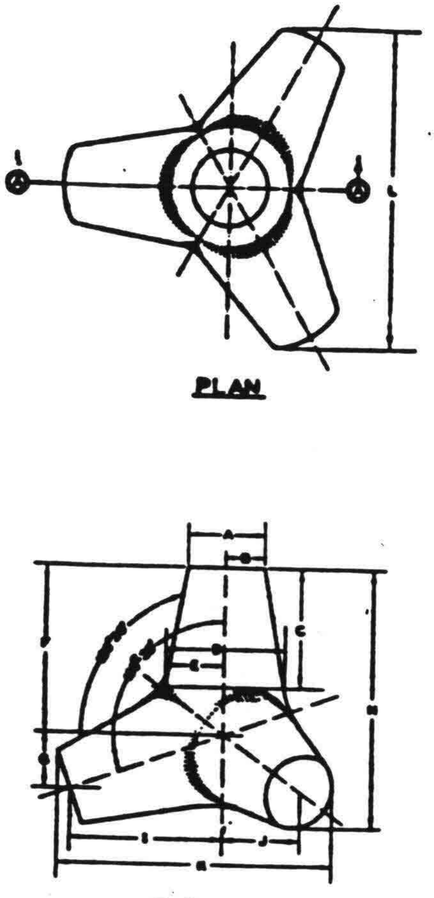
Ad 1) This method is very effective, especially because ρ_r is part of Δ, which is cubed in the formula!
To increase ρ_r of concrete, sometimes iron ore is added.
$\rho_{natural}$ stone varies between 2200 and 3000 kg/m3
$\rho_{concrete}$ varies between 2000 and 3000 kg/m3

Ad 2) Increasing W depends on the possibilities of the quarry.

Ad 3) Increase of cot α soon requires enormous quantities of material. Often one of the other possibilities is chosen when the slope would exceed 1 : 3

Ad 4) In addition to the blocks mentioned in Table 4.2, several other types have been developed. Although higher values are sometimes published by the inventor, it is not recommended to use K_D factors exceeding 10 to 12. See Figure 4.3

Ad 5) The experience with asphalt grouted groins and breakwaters has shown great successes and great disappointments. The utmost care is required (See 6.[illegible].2) [26]

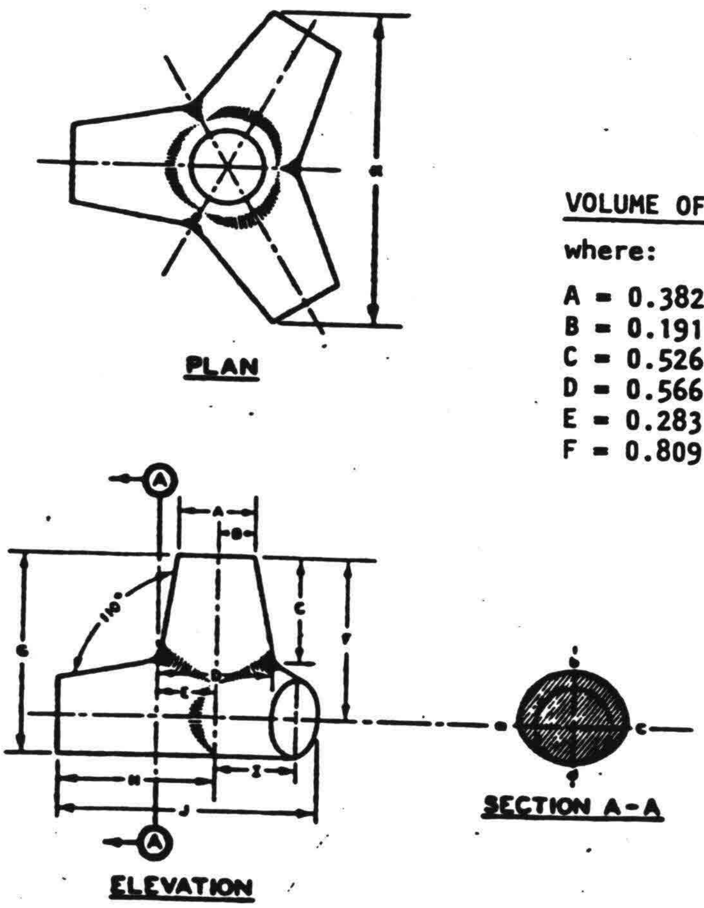


VOLUME OF INDIVIDUAL ARMOUR UNIT = $0.280\ H^3$

where:

A = 0.302 H
B = 0.151 H
C = 0.477 H
D = 0.470 H
E = 0.235 H
F = 0.644 H
G = 0.215 H
H = Overall dimension of un..
I = 0.606 H
J = 0.303 H
K = 1.091 H
L = 1.201 H

a. Tetrapod



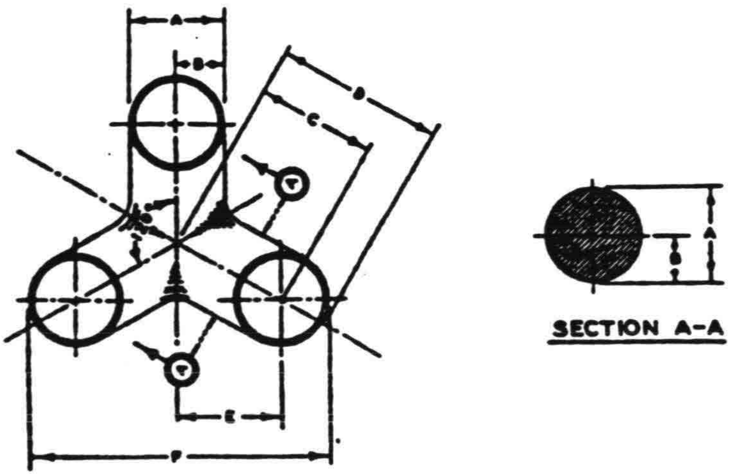
VOLUME OF INDIVIDUAL ARMOUR UNIT = $0.495\ G^3$

where:

A = 0.382 G
B = 0.191 G
C = 0.526 G
D = 0.566 G
E = 0.283 G
F = 0.809 G
G = Overall height of unit
H = 0.809 G
I = 0.405 G.
J = 1.379 G
K = 1.592 G

b. Quadripod

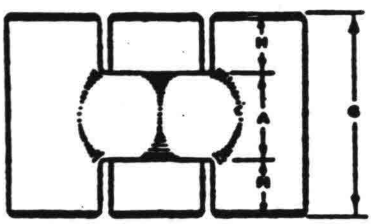
Fig. 4.3 Armour units



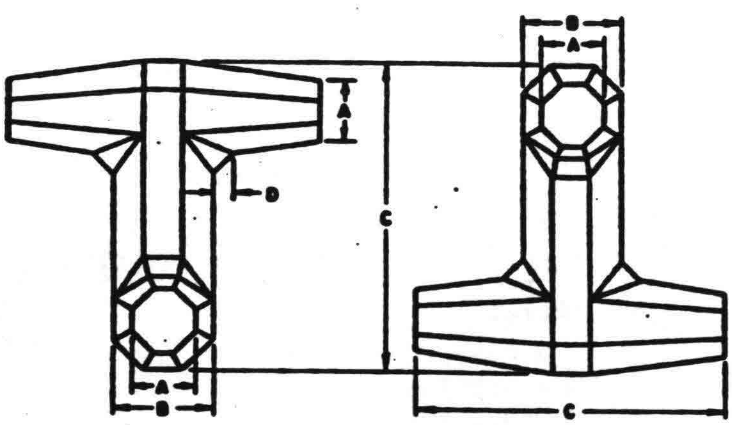
VOLUME OF INDIVIDUAL ARMOUR UNIT = $6.48\ A^3$

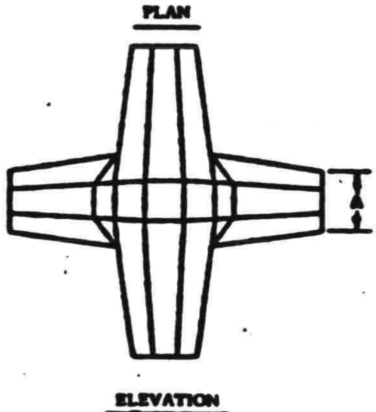
where:

A = Diameter of leg
B = 0.5 A
C = 1.25 A
D = 1.75 A
E = 1.08 A
F = 3.16 A
G = 2 A
H = B = 0.5 A



c. Tribar





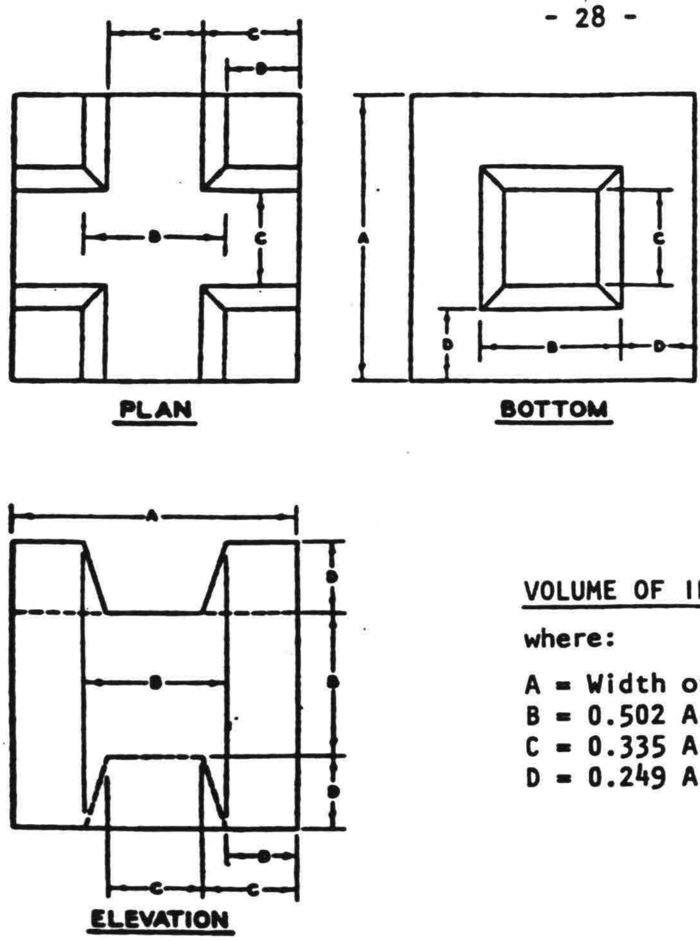
VOLUME OF INDIVIDUAL ARMOUR UNIT = $0.16\ C^3$

where:

A = 0.20 C
B = 0.32 D
C = Overall dimension
D = 0.057 C

d. Dolos

Fig. 4.3 Armour units

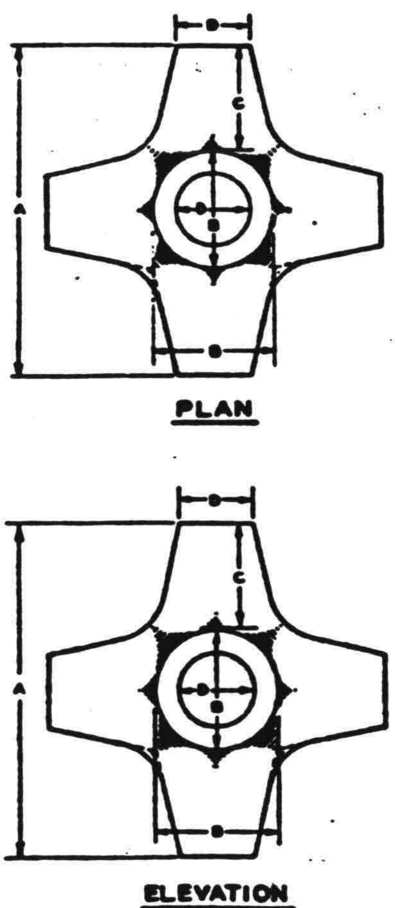


VOLUME OF INDIVIDUAL ARMOUR UNIT = 0.781 A^3

where:

A = Width of cube
B = 0.502 A
C = 0.335 A
D = 0.249 A

e. Modified-Cube



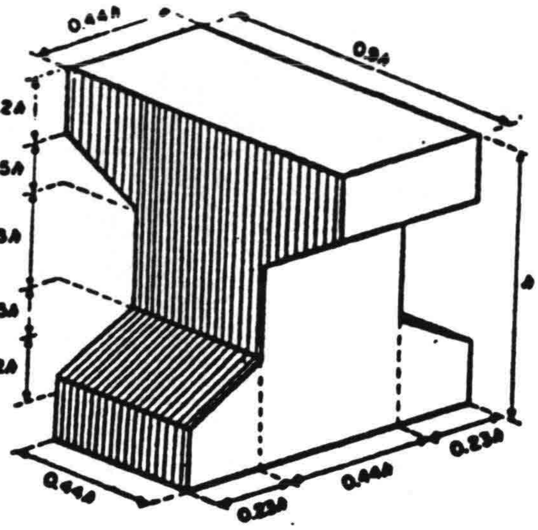
VOLUME OF INDIVIDUAL ARMOUT UNIT = 0.176 A^3

where:

A = Overall dimension of unit
B = 0.357 A
C = 0.322 A
D = 0.215 A

f. Hexapod.

Fig. 4.3 Armour Units



Volume of block : 0.280 h3

g. Akmon

Fig. 4.3 Armour units.

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4.9 Run-up and overtopping

Run-up is defined as the vertical distance between the still water level and the highest elevation of a wave tongue on a slope.

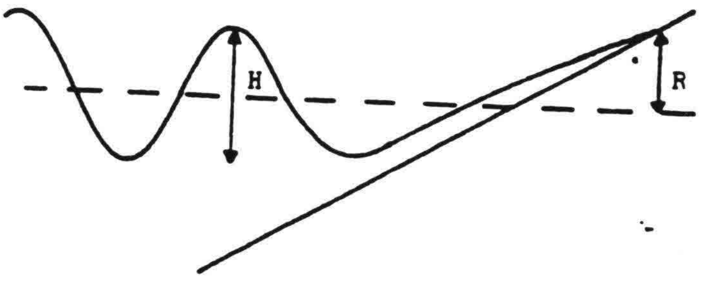

Fig. 4.4 Wave run-up

The wave run-up is directly related to:
wave height H, wave period T, slope, roughness and porosity of the slope, foreshore etc. Much experimental work has been done especially for smooth impervious slopes.
Much less is published for rubble mounds of rubble covered slopes.
Reliable data can be obtained from [2]
See also Figure 4.5

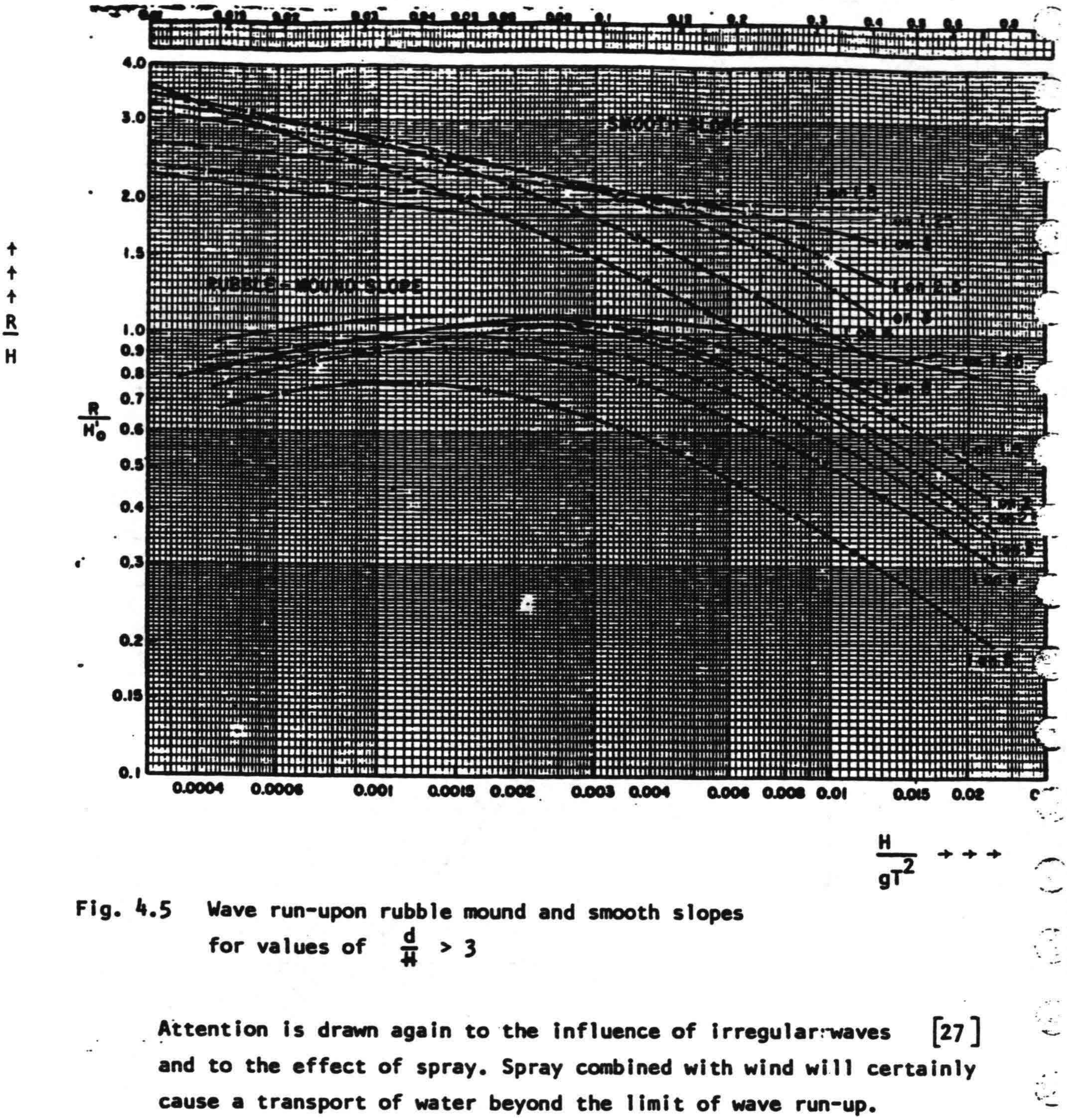

Fig. 4.5 Wave run-upon rubble mound and smooth slopes for values of $\frac{d}{H} > 3$

Attention is drawn again to the influence of irregular waves [27] and to the effect of spray. Spray combined with wind will certainly cause a transport of water beyond the limit of wave run-up.

Overtopping is defined as the quantity of water per unit length of structure and per unit of time passing over the crest of the structure. Overtopping occurs when the crest level is lower than the level of wave run-up. The quantity of overtopping is important when it is pumped away or drained from a basin, a polder, a harbour area etc.

The rate of overtopping is to be measured in model tests, applying the actual shape of the structure. The small amount of published data will generally not meet the actual design.

For a breakwater in the usual sense of the word, often the rate of overtopping is not important. In many cases one is interested only in the waves created at the lee-side by the overtopping masses of water. In such cases one speaks of wave transmission.

This situation occurs particularly when the crest level of the breakwater is below S.W.L...In this case one speaks of a submerged breakwater. A fairly good summary of the available literature is given in [28] . Figure 4.6 shows the general trend of wave transmission as a function of crest level.

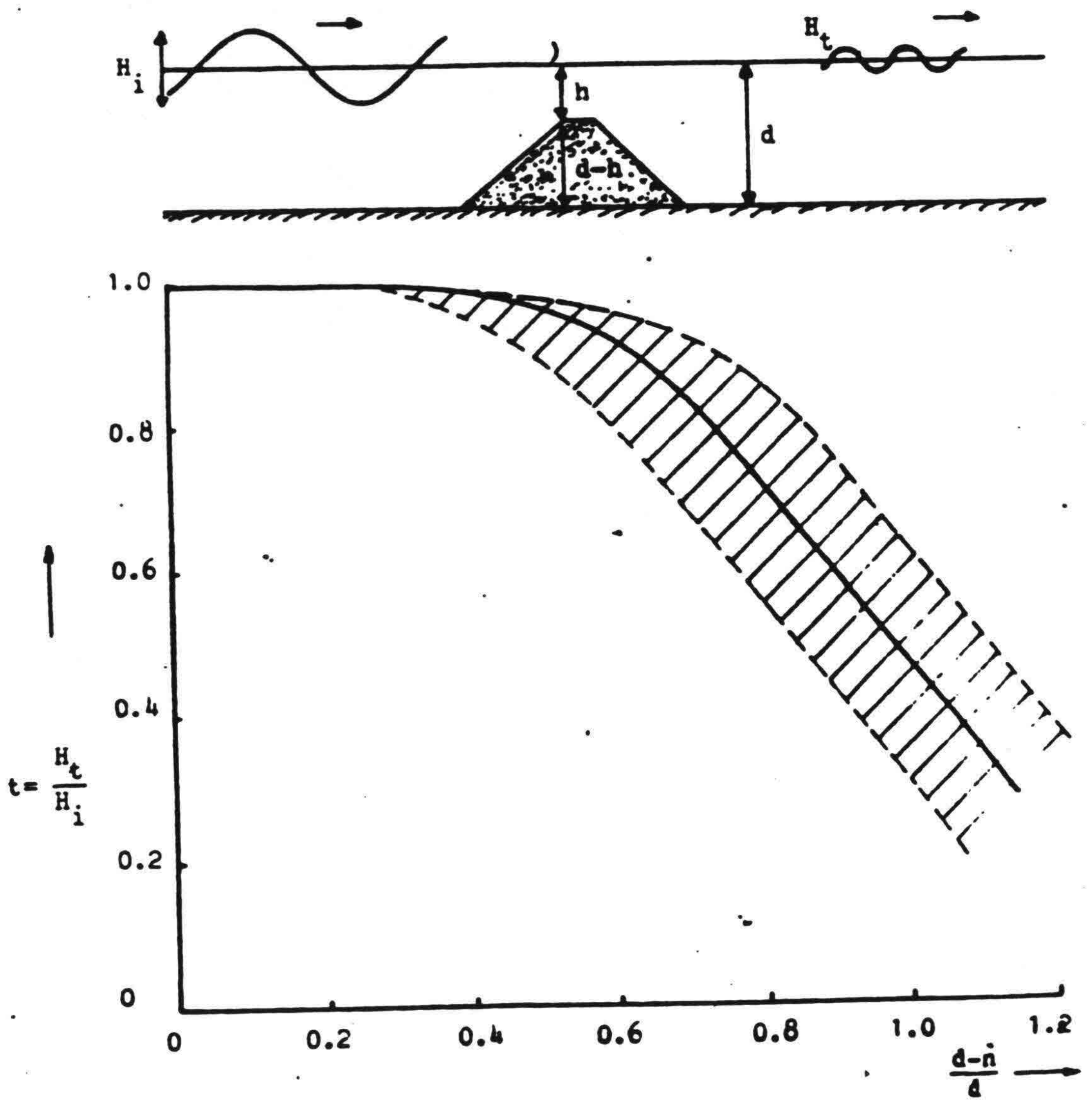

Fig. 4.6 Wave transmission

4.10 Porosity and layer thickness

The porosity of the structure is important for two ~~seasons~~; wave penetration and assessment of quantities.
The porosity is also directly influencing the thickness of a layer consisting of n stones.

This layer thickness r equals the product of the number of layers, the relevant size of the armour unit and a constant, depending on the shape of the armour unit.

Thus:

$$r = n.k_\Delta \, . \, (\mathrm{Vol})^{1/3} = n.k_\Delta \, . \, \left(\frac{W}{\rho_r \, g}\right)^{1/3}$$

~~Values for the coefficient K_Δ and the percentage of porosity (P) can be found from table 3.~~

Porosity and coefficient K_Δ can be found from table 4.4

Table 4.4

- Porosity and layer thickness		
Material	K_Δ	Porosity (%)
Quarry stone	1.0 to 1.15	38 - 40
Modified cube	1.1	47
Cube	1.1	47
Tetrapod	1.04	50
Akmon	0.90	57
Dolos	1.0	63

4.11 Oblique wave attack

All data and figures mentioned in the previous paragraphs refer to perpendicular wave attack. The available data on the effect of oblique wave attack are not consistent. It cannot be assumed that oblique attack is less dangerous than a perpendicular one.

4.12 Pell mell or special placement

Random placement of stones is also referred to as pell mell. It is obvious that the special placement leads to a better stability. It should be realized that in many cases the special placement is completely impossible. A further disadvantage is that maintenance or repair is virtually impossible. Finally, one should realize that special placement may increase the zero-damage value, but possibly the damage may increase very rapidly thereafter. Thus it is risky to accept high K_D values for special placement.
In the Netherlands and among the Dutch contractors working abroad, the special placement is generally not accepted for these reasons.

4.13 Special conditions

In special conditions the stability may be considerably worse than indicated by the formulae. The most common examples are:-

- Heads of breakwaters
- Breaking waves
- Crests of submerged breakwaters

For the head of a breakwater, it can be assumed that the K_D value is reduced to 75 - 50% of its original value.

5. DESIGN OF CROSS SECTIONS

5.1 Introduction

Since natural rock is obtained by blasting, the designer must expect that various sizes of rock are produced at the same time. Anticipating the percentage of various sizes of rocks is one of the most difficult aspects of designing a rubble mound from a newly opened quarry, since in principle all material must be used!

With the mixture from the quarry one can act as follows:

5.1.1 Use only the heaviest blocks from the quarry. The breakwater will be built up from a rather uniform size of stone. As long as the weight meets the values calculated with the Hudson formula, the stability will be ensured.

One must realize, however, that:-

- the permeability will be great; (wave and sand penetration)
- the large blocks are placed directly on the bottom and may sink into it.

In general this method is not accepted.

5.1.2 Use the mixture of stone as it comes from the quarry. The permeability and foundation will cause no problems. The expensive work of classifying stone is not required.

Disadvantages are:-

- the fines will be washed out and deposited in an uncontrolled way;
- the fines may work as "grease" in the skeleton of larger blocks.

In general this method is used where the cost of classification is high and the cost of the extra quantities required low due to short transport distance.

5.1.3 Construct the breakwater in layers of classified stone. This most common method will be discussed in the next paragraphs.

Much information on design and a great number of examples can be found in [~~2~~ 1 and ~~29~~ 6]

7,3

5.2 Construction in layers

The general ideas of this construction method are illustrated in Figures ~~5~~.1 and ~~5~~.2

5 6

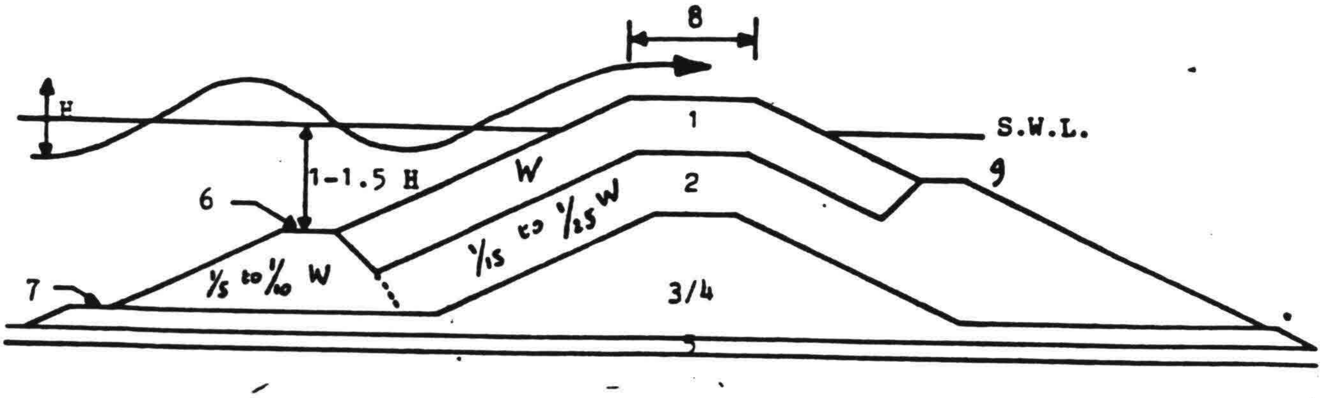

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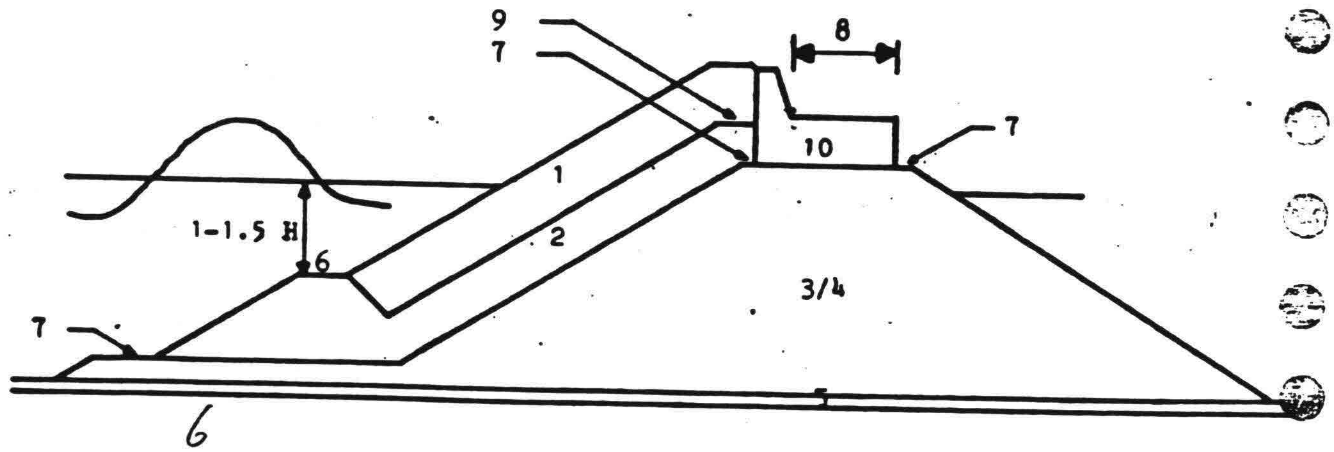
Fig. 5.1 Overtopping breakwater.

1. armour layer
2. second class stone
3. small stones (3rd class)
4. quarry run
5. filter layer(s)
6. berm
7. extra width for flexibility
8. crest (width min. 3 armour units)
9. inner berm below S.W.L.



6

Fig. 5.2 Non-overtopping breakwater.

1. armour layer
2. second class stone
3. small stones (3rd class)
4. quarry run
5. filter layer(s)
6. berm
7. shoulder for flexibility
8. crest (width = road width)
9. support for 1 armour unit
10. cap construction

3.3.1

~~5.2.1~~ General rules:-

- Each layer should prevent material from sublying layers to pass through its voids.
- The outer layer(s) should withstand the design wave.
- All layers should show a reasonable stability during construction.
- Restrict the number of layers (cost).
- The minimum layer thickness is 2 stones for safety.

5.2.2 Outer or armour layer. (sea side):

The outer layer should be designed in such a way that it withstands the design wave. It shall extend from a level of 1 to 1.5 H below the minimum S.W.L. to the crest in case of a non-overtopping breakwater. Proper attention should be paid to support the armour layer at toe and crest. ~~When calculating the block weight, H_s can be used instead of H in the Hudson formula.~~

3.3.3

~~5.2.3~~ Underlying layers:

The underlying layers may consist of stone with a weight of 1/15 tot 1/25 of the block weight in the next layer provided the stability during construction permits it.
This applies when the same type of materials is used. In special cases (quarry under "dolosses" etc.) the ratio should be kept more conservative!

3.3.4

~~5.2.4~~ Core:

The core of the breakwater usually exists of quarry run. This material has the advantage that it is almost impermeable for sand, which is important when the breakwater also has to stop the longshore transport.

Hans dooren

./38

5.2.5 Foundation:

When the breakwater is built on a sandbed, special precautions have to be taken to prevent erosion of the sand from underneath the breakwater. The wave action is responsible for pressure fluctuations inside the mass of the breakwater. Under the influence of these pressure fluctuations (and the accompanying water movement) particles from the subsoil can be whirled up and washed out, which leads to undesirable settlement of the structure.

To counteract these effects, a filter layer is necessary between the actual breakwater and the subsoil. Such a filter derives it's protective power from two facts: -

- it may damp the pressure fluctuations at the critical interface;
- it may form a physical barrier against washing out of fine particles.

The design of a filter layer is not simple. Model investigations are complicated since scale rules for a proper representation of the pressure fluctuations inside the breakwater do not conform the scale rules for the movement of sand particles nor for the physical transport of sand grains through the voids of succesive layers of material.
Therefore, a series of model investigations is required, whereby the results of a first model form the boundary calculations for a second experiment.

To arrive at a proper estimate of the pressure fluctuations inside the breakwater, attention should be paid to the head losses inside the mass of rubble at low Reynolds numbers (model scale!) 21 and 21A.. Tests to actually investigate the sand-tightness of filter material are then carried out on a large scale, applying measured or calculated hydraulic gradients.

Filters can consist of: -

- several layers of gravel or
- fascine mattresses or
- filter cloth (woven or non-woven).

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When filter cloth is used, generally a woven or non-woven synthetic material is selected which is sand-tight but not water-tight. When the material would be impermeable, water over-pressures might develop underneath the foil and create a quick-sand condition. On top of the filter cloth a protective layer is applied to prevent large angular stones to puncture the structure. Manufacturers of filter cloth can provede data on sand tightness and permeability of their products.

A layered gravel filter consists of a number of layers. The consecutive layers prevent the material of the underlying finer grains from geing washed out. To be sure that the proper filter composition is achieved in all places, the minimum thickness of each layer constructed under water is 0.5 m.

5

Recently, extensive investigations have been performed on the filter properties of gravel beds. ~~[30]~~ and ~~[31]~~ These investigations have resulted in design rules, stating the critical hydraulic gradient as a function of the ratio between the grain size of the base material D_b and the grain size of the filter material D_f. A distinction is made between flow parallel and perpendicular to the interface and between steady flow and unsteady flow conditions. (Figures ~~5.3~~, ~~5.4~~ and ~~5.5~~)

6, 7, 8

7 8 9

Note: The subscript 15 indicates that 15% of the material is finer that the grain size indicated.
The index p (D_{90}/D_{10}) is a measure indicating the gradation of the material.
The void ratio of a certain material is indicated by the letter n.

Similar filters shall be applied ~~when~~ along the slopes when the breakwater serves as retaining bund for a reclamation.

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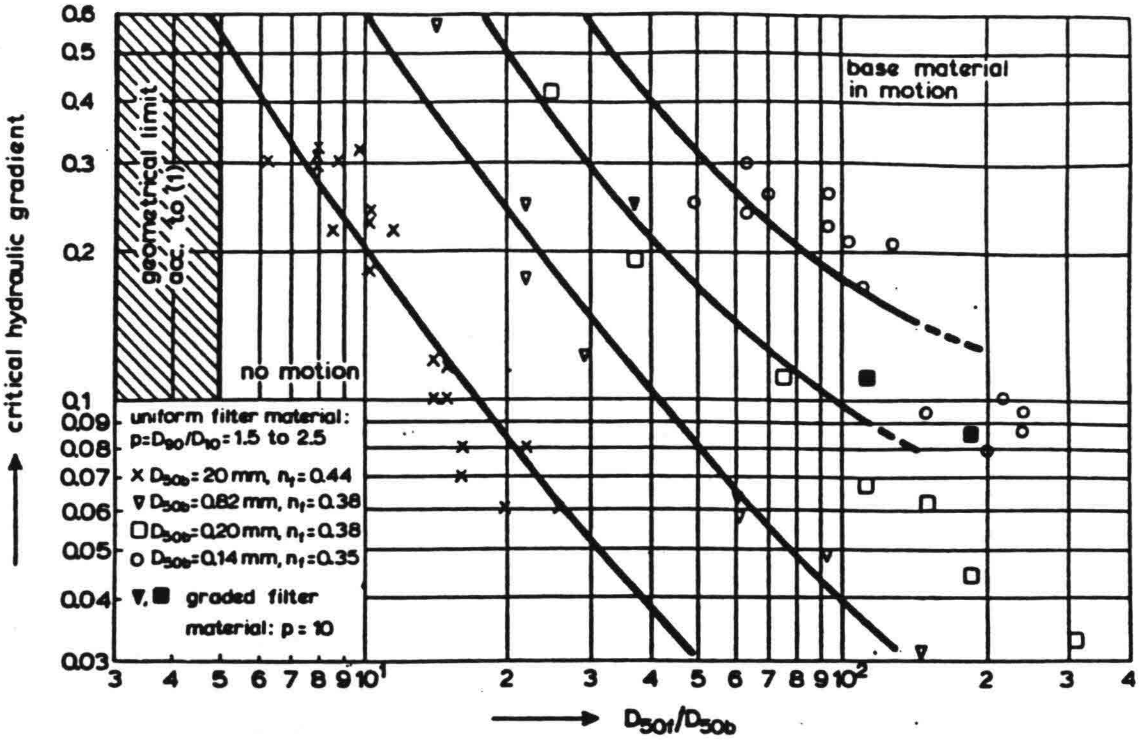

Fig. 5.3 Critical Hydraulic Gradient with Steady Flow Parallel to Interface.

7

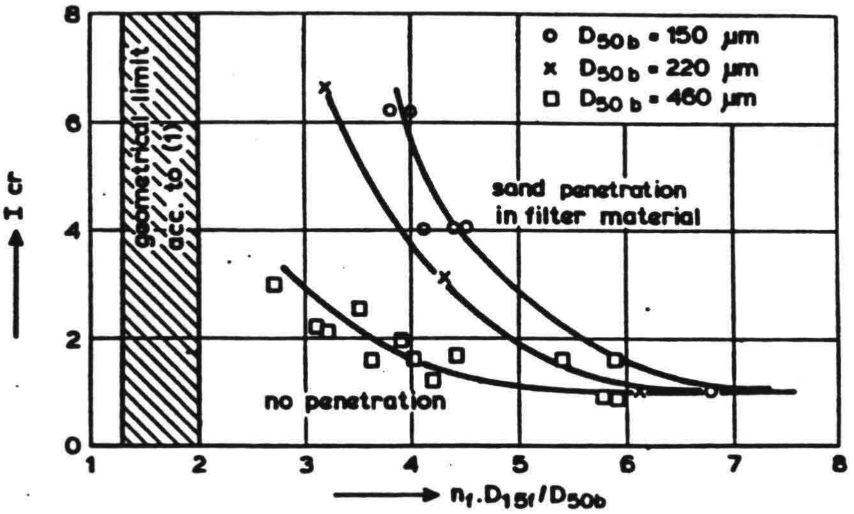

Fig. 5.4 Critical Hydraulic Gradients with Steady Flow Perpendicular to Interface.

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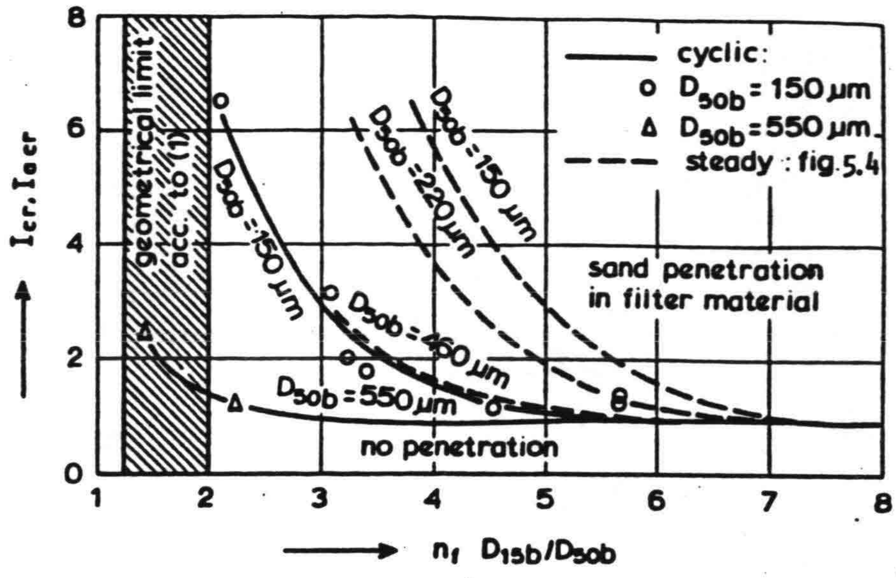

Fig. 5.5 Critical Hydraulic Gradients with Cyclic Flow Perpendicular to Interface.

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[redacted]

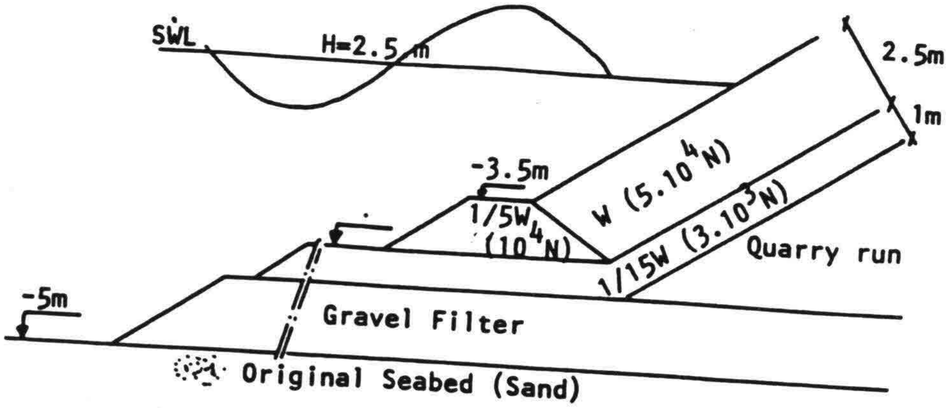

Fig. 5.6a Standard design (water depth 8m minimum)

10a

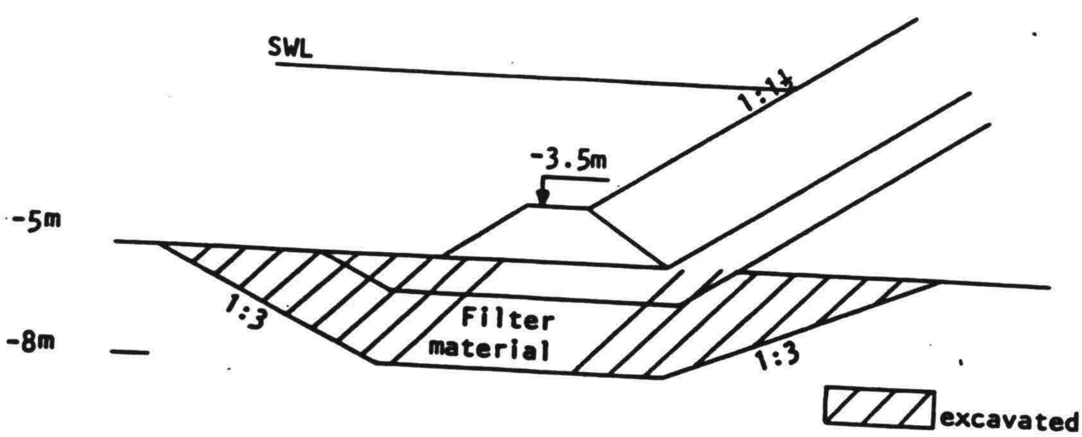

Fig. 5.6b Shallow water (5m) dredged trench gravel filter.

10b

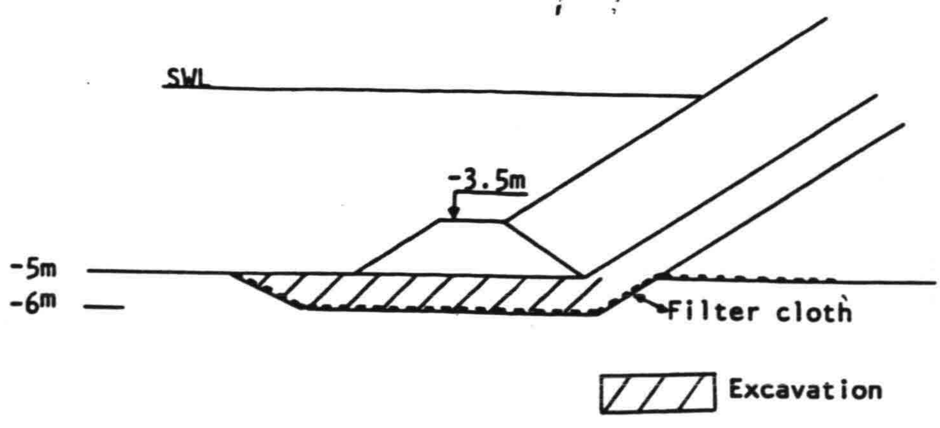

Fig. 5.6c Shallow water, dredged trench filter cloth.

10 c

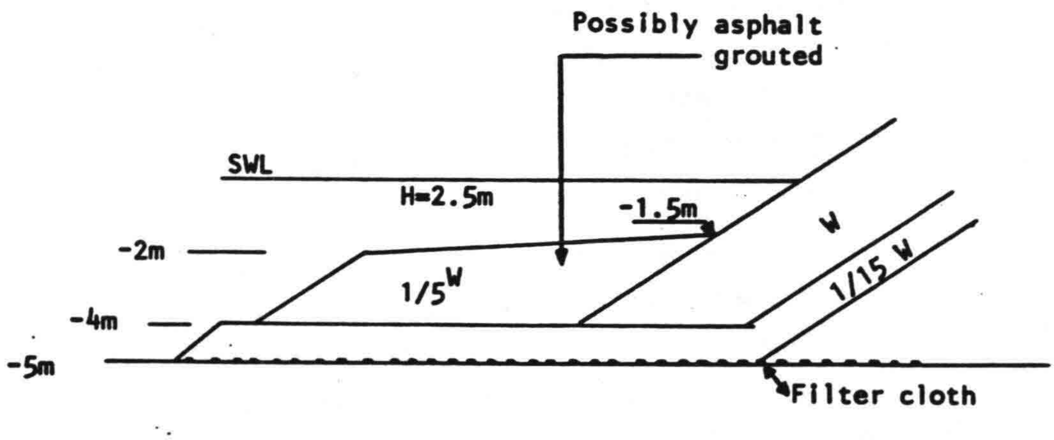

Fig. 5.6d Shallow water

no excavation
filter cloth
increased berm

10 d

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~~5.2.6~~ Scour protection:

The filter layer is normally extended beyond the toe of the breakwater to avoid scouring. In order to protect the filter layer itself (beyond the toe acting as a bottom protection layer or revetment) from the wave action in shallow water, a layer of heavier stones should be placed on top of this filter layer outside the toe of the breakwater.

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5.2.7 Toe:

The toe of the breakwater should be designed in such a way that it gives support to the armour layer. It should allow for some inaccuracy in the placing of both toe units and armour units. The unit weight of toe blocks shall always be determined by model investigation. The weight of the units is 1/5 to 1/10 of the armour weight, depending on the level of the berm.

In shallow water, (thus for the breakwater sections close to the shore) it is sometimes difficult to find space for all required layers on top of each other.

In those cases, it is necessary to find compromise solutions. Common construction methods are then :

a) dredging a trench to provide space for all filter layers;

b) use filter cloth instead of gravel layers to reduce the height of the structure;

c) disregard the design rules and accept toe material at a level much higher than theoretically possible.

In this case loss of stones should be accepted and provided for by placing an additional quantity. Extra safety can be obtained by applying asphalt grout.

The basic principle of these solutions is illustrated in Fig. ~~5/6~~ 10 (a through d)

3 3 8

5.~~2~~.8 Crest:

The crest elevation is selected on the basis of acceptable overtopping. If no overtopping is allowed, often a concrete cap is designed with a parapet supporting the armour layer and reducing the volume of core and inner slope. It is also used as a road for maintenance work. The core should be equipped with a shoulder to prevent large voids between units and parapet when the structure is settling (Figure ~~5.2~~). 6 The concrete cap can be precast or cast in situ. In the latter case attention should be paid to the filling of voids.

The stability of the cap block will be subject to a (model) study.

When overtopping is allowed, the armour layer is often continued over the crest. In that case the crest width should be at least 3 blocks. The blocks on the crest line are relarively vulnerable; interlocking blocks improve the stability to a lower extent than on a slope!

5.~~2~~.9 Inner slope:

The inner slope will withstand the waves in the inside of the harbour. Often, however, the overtopping is decisive for the block dimensions. Data are not available in literature, tests have to be carried out. In general when serious overtopping is expected, the armour layer is continued to 1 or 2 m. below minimum S.W.L. (Figure ~~5.1~~)
5

3 9

~~5.3~~ <u>Head of the breakwater</u>

The head of a breakwater is relatively vulnerable since the curvature causes the armour units to be less supported and/or interlocking.

In general damage occurs in one of the inner quadrants. (Fig. ~~5.7~~)
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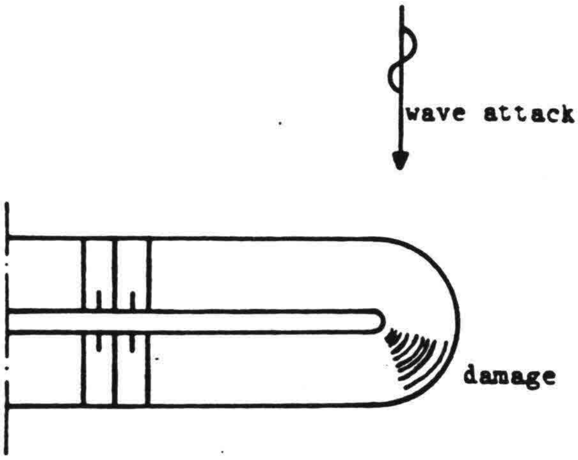

Fig. ~~5.7~~ 11 Head damage.

Therefore, the head of a breakwater is often reinforced by using heavier blocks or by reducing the slope.

Increasing the block weight involves the following disadvantages:-
~~Construction:~~

- Crane shall be able to place heavier blocks.
  This is necessary for a small part of the structure.
- In case of quarry stone in the outer layer,
  it is questionable whether the quarry can produce heavier blocks.
- In the case of concrete armour units, it is not very economical to produce a small number of different blocks
  (increase of ρ_r may be possible for a small number of blocks, by selecting heavier aggregates such as iron ore)

Reducing the slope has also disadvantages:-

- The harbour entrance is narrower and the width of the entrance is less defined.
- If this method is adopted, proper attention must be paid to the navigational aspects! ~~(see Figure 5.8)~~
- The crane shall be able to place blocks at a greater distance.

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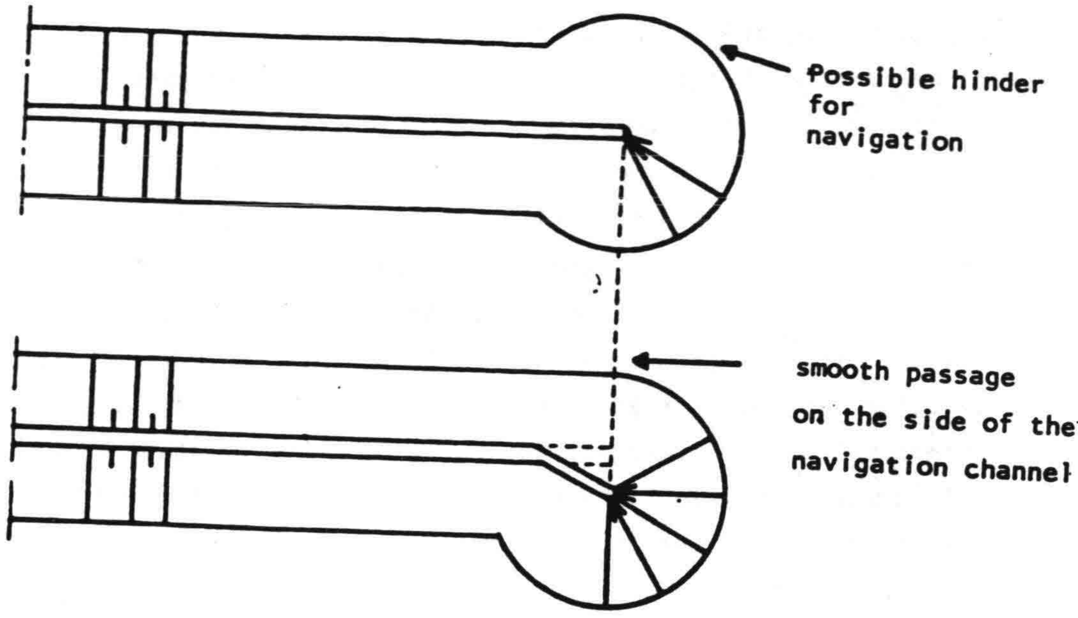

Fig. 5.8 Typical head design.

6. CONSTRUCTION METHODS

6.1 Introduction

It is out of the question to construct a breakwater in the dry, using cofferdams and pumps. Further, a large majority of the material has to be placed under water, which puts strong limitations on construction methods, accuracy and possibility of inspection. The design of a rubble mound breakwater should include sufficient safety margins to tackle these problems. In general, it can be stated that positioning errors must be expected of 1 to 2 m. in horizontal directions and of 0.3 to 0.5 m. in the vertical direction. In spite of such inaccuracies, the sequence of successive filter layers may not be lost. It is evident that in such conditions there is a strong mutual influence between design and construction method. It is virtually impossible to produce a sound design of a rubble mound breakwater without considering the construction method.

This situation is aggravated by the need to provide a reasonable degree of safety during subsequent construction phases.

The construction of a rubble mound breakwater starts at the quarry. Therefore, special attention will be paid to reconnaisance, testing and operation of a quarry. Further, transport of material from the quarry to the construction site and methods for bringing the material in position will be discussed.

6.2 Quarry

6.2.1 Reconnaisance:

Basically, two types of quarries can be discerned:

a) Producing aggregates for concrete etc.
   A fine fragmentation is required. It is achieved by special drilling and blasting techniques. Classification is done by sieving.

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b) Producing blockstones.

The aim of the quarry operation is here to produce the largest possible blocks by sawing and cutting or by drilling and blasting.

Classification takes place by picking up individual blocks.

For the construction of rubble mound breakwaters, quarries of the b-type are indispensable.
The size of the blocks obtained from the quarry is limited by the geological properties of the stone massif. Whatever is the origin of the geological formation, there will be discontinuities restricting the block size. To a certain extent, the size of the blocks can be influenced by the drilling and blasting pattern, but the size of a block will never exceed the distance between the natural cracks in the material.

When assessing suitable locations for a quarry a geological survey should be carried out, paying attention to the following points:- [32]

- Joints (see Fig. 6.1 and 6.2)

  A break of geological origin in the continuity of a body of rock along which there has been no visible displacement. A group of parallel joints is called a set and joint sets intersect to form a joint system. Joints can be open, filled or healed.

  Joints frequently form parallel to the bedding-planes, foliation and cleavage and may be termed bidding-joints, foliation joints and cleavage-joints accordingly.

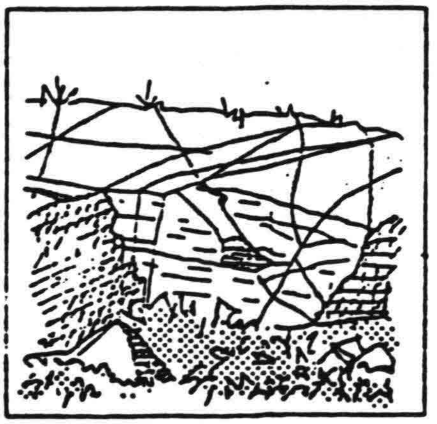

Fig. 6.1

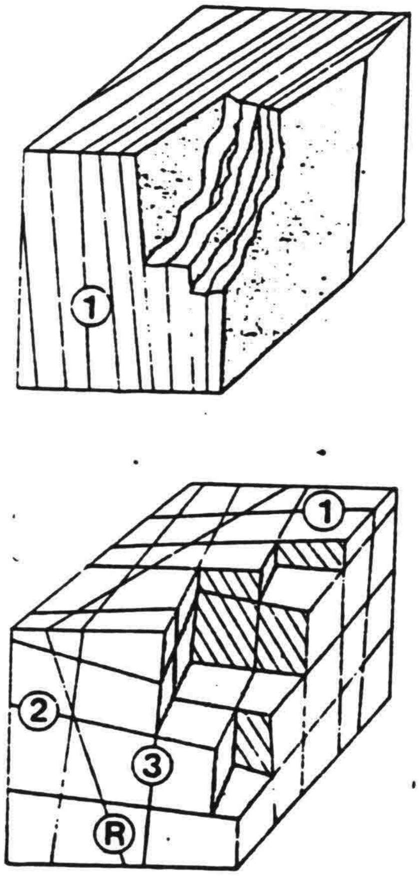

Fig. 6.2

- Faults (see Fig. 6.3 and 6.4)

  A fracture or fracture zone along which there has been recognisable displacement from a few centimeters to a few kilometers in scale. The walls are often striated and polished (slickensided) resulting from the shear-displacement.
  Frequently rock on both sides of a fault is shattered and altered or weathered, resulting in fillings such as breccia and gouge. Fault width may vary from millimeters to hundreds of meters.

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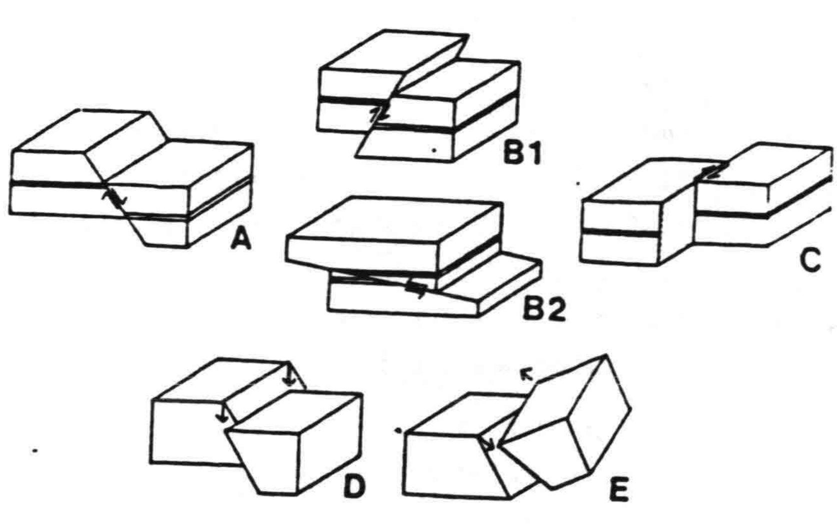

Fig. 6.3

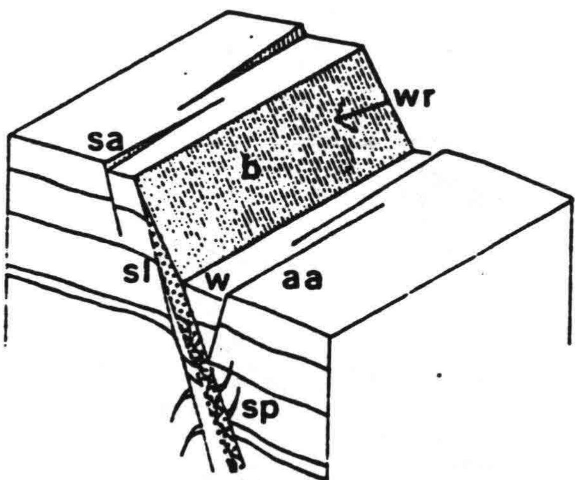

Fig. 6.4

- Discontinuities:

  The general term for any mechanical discontinuity in a rock mass having zero or low tensile strength.
  It is the collective term for most types of joints, weak bedding planes, weak schistocity-planes, weakness zones and faults.
  The ten parameters selected to describe discontinuities and rockmasses are as follows:

  1) Orientation:
     Attitude of discontinuity in space (see Fig. 6.5)

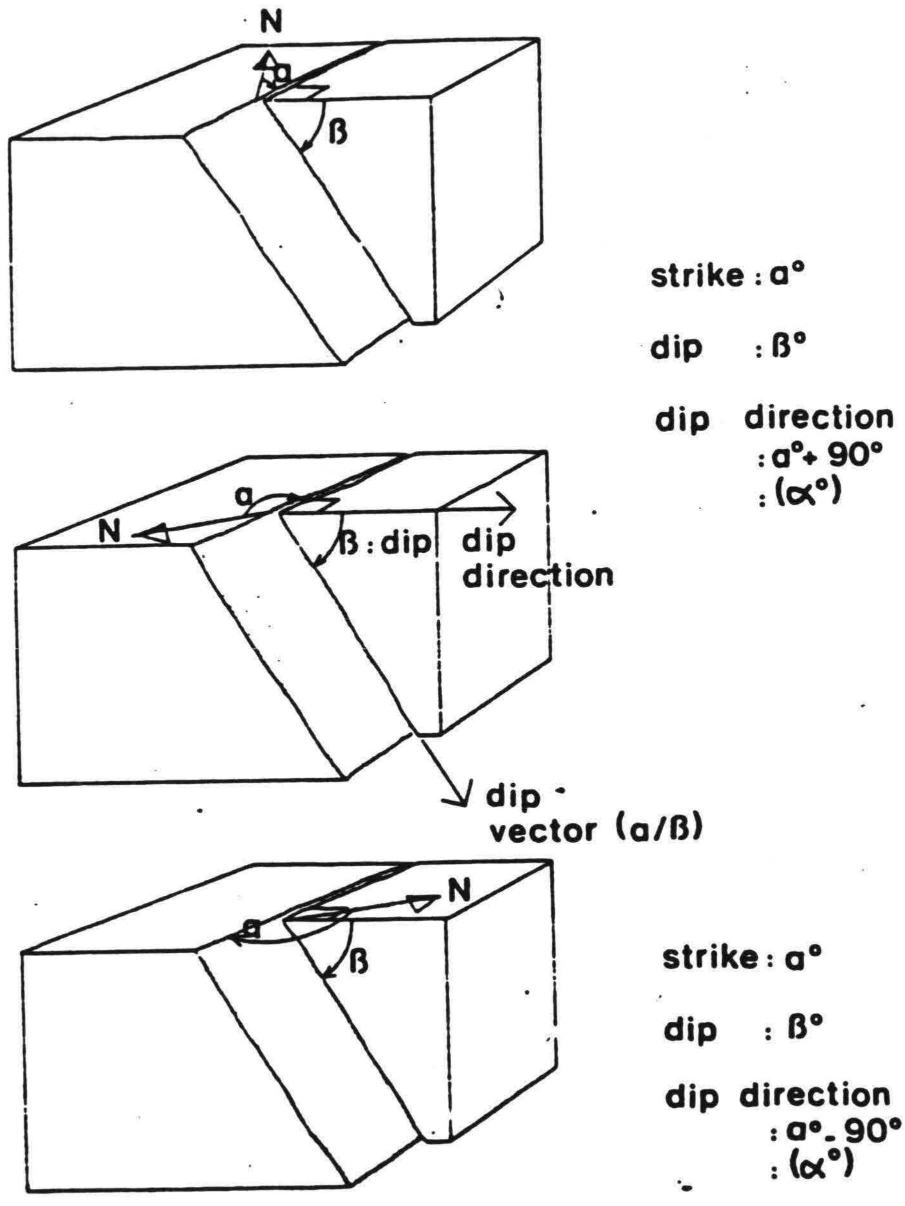

Fig. 6.5

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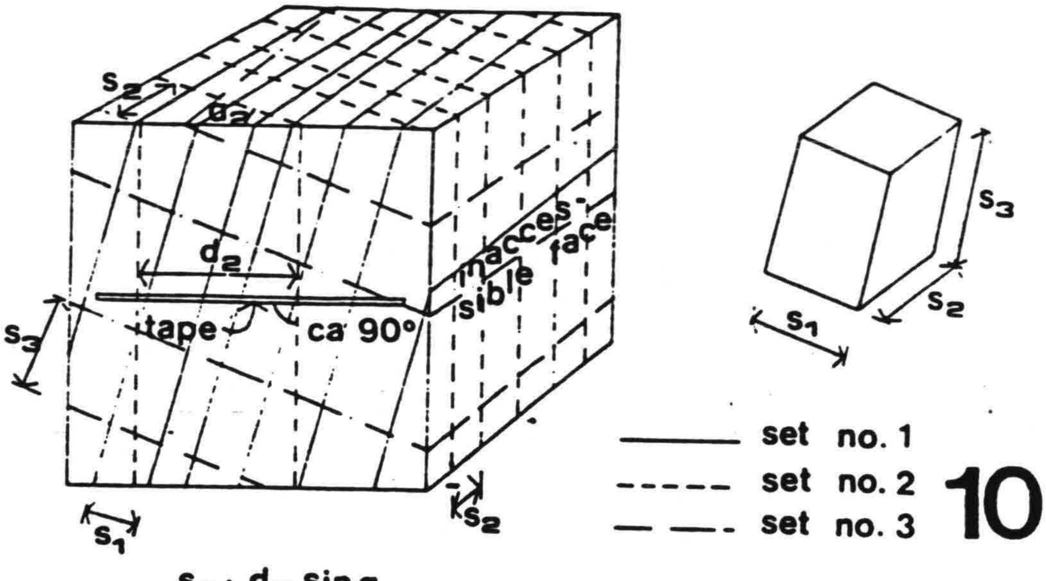
2) Spacing:

Perpendicular distance between adjacent discontinuities. (see Fig. 6.6 and 6.7)



$s_2 : d_2 \sin a_2$

Fig. 6.6

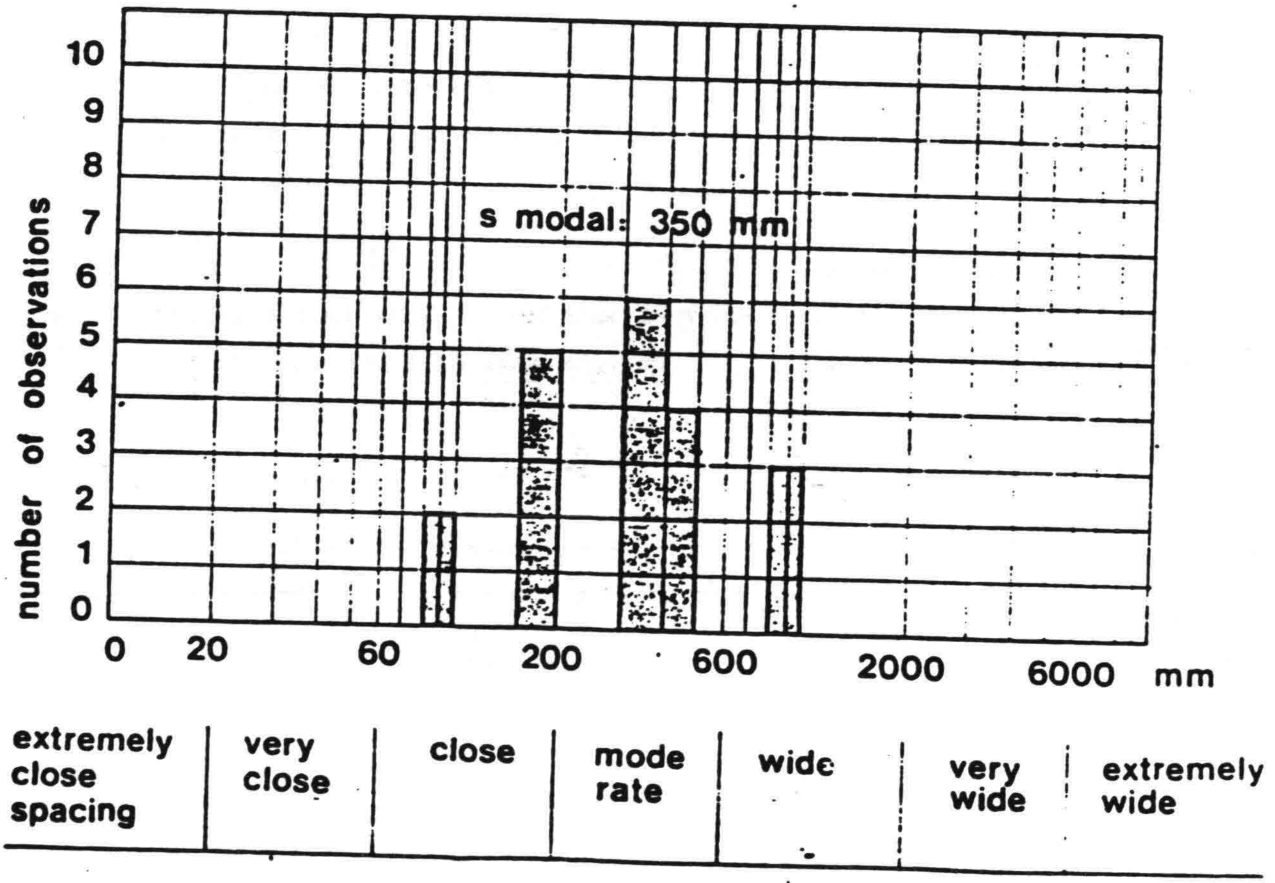


Fig. 6.7

3) Persistence:
Discontinuity trace length as observed in an exposure.

4) Roughness:
Inherant surface roughness and waviness relative to the mean plane of a discontinuity.

5) Wall strength:
Equivalent compression strength of the adjacent rockwalls of a discontinuity.
Maybe lower than rock block strength due to weathering or alteration of the walls.

6) Aperture:
Perpendicular distance between rock-walls of a discontinuity in the intervening space is air or waterfilled.

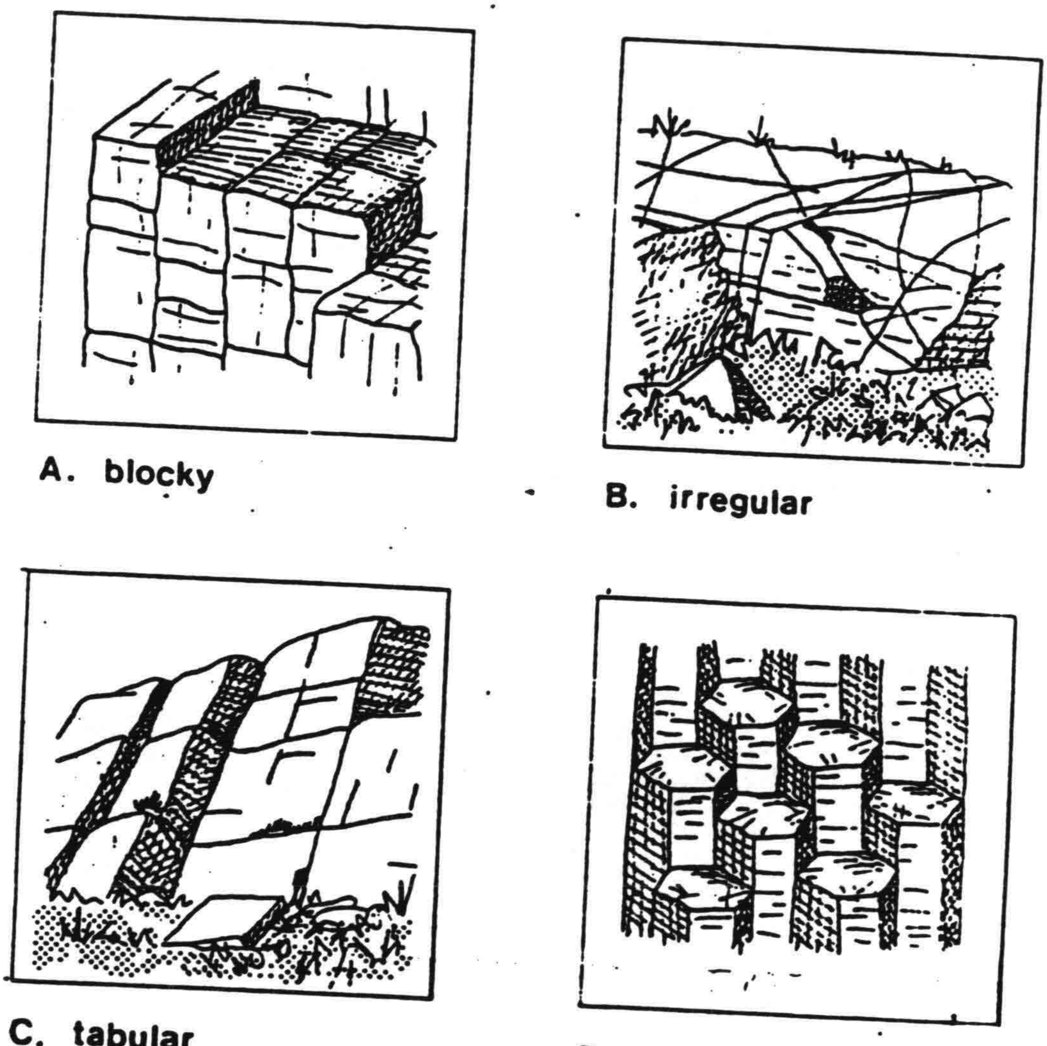
7) Filling:
Material that separates the adjacent rock-walls of a discontinuity and that is usually eeaker than the parent-rock.

8) Seepage:
Water-flow and free-moisture visible in individual discontinuities or in the rock mass as a whole.

9) Number of sets:
The number of joint sets comprising the intersecting joint system the rock mass may be divided by individual discontinuities.

./55

10) Block-size (see Fig. 6.8)
Rock-block dimensions resulting from the mutual orientation of intersecting joint sets and resulting from the spacing of the individual sets. Individual discontinuities may further influence the block and the shape.
Block-size can be described either by means of the average dimension of typical blocks (block-size index I_b) or by the total number of joints intersecting a unit volume of the rockmass (Volumetric Joint Count J_v).
(See Table 6.1)



A. blocky

B. irregular

C. tabular

D. columnar

Fig. 6.8

Table 6.1

The following descriptive terms give an impression of the corresponding block size:

Description	J_v (Joints/m^3)
Very large blocks	< 1.0
Large blocks	1 - 3
Medium-sized blocks	3 - 10
Small blocks	10 - 30
Very small blocks	> 30

Values of J_v > 60 would represent crushed rock, typical of a clay-free crushed zone.

On the basis of this information an experienced geologist is able to provide an expected fragmentation curve. See Fig. 6.9 as example.

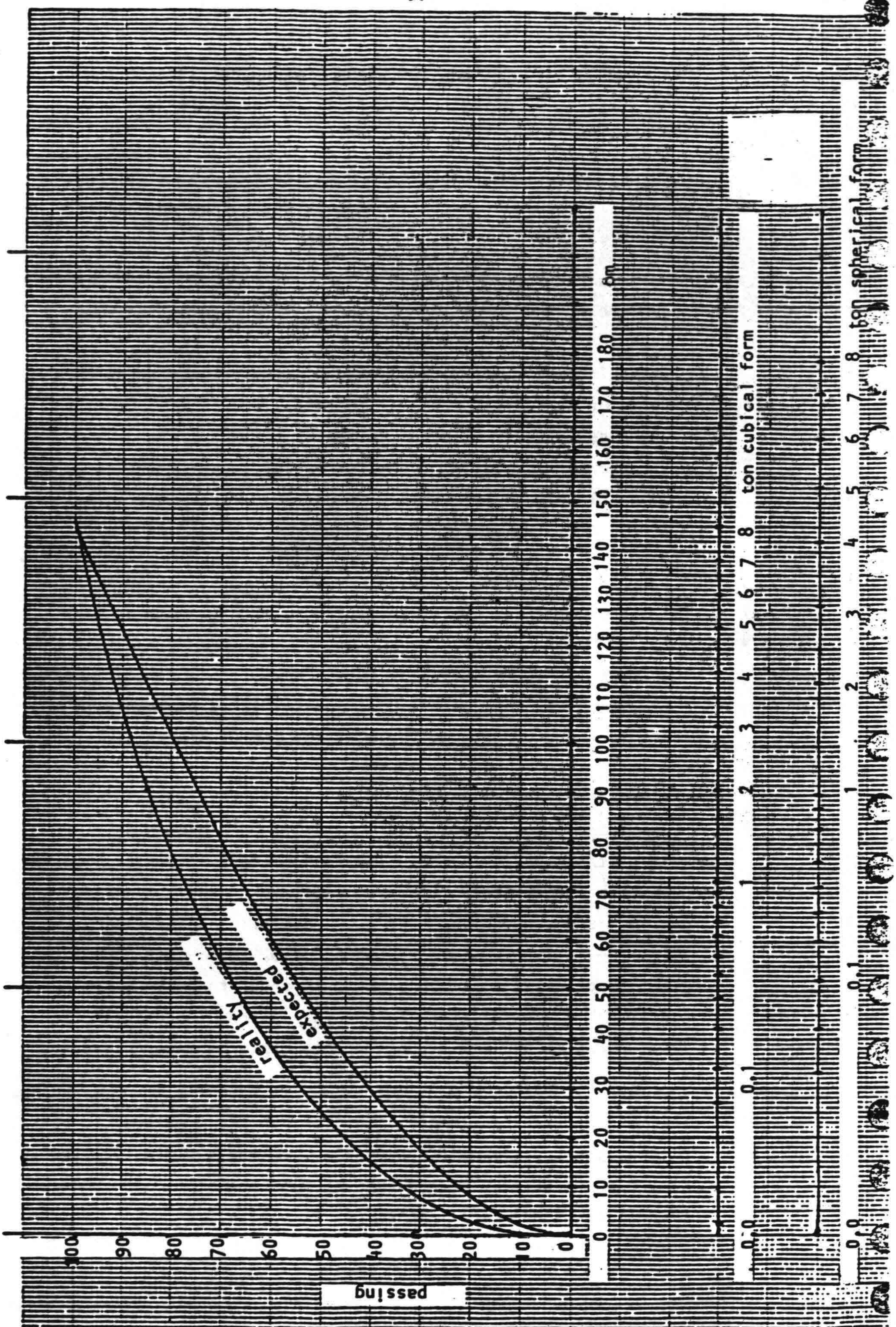

passing
0
10
20
30
40
50
60
70
80
90
100
reality
expected
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180
0.0 0.1 1 2 3 4 5 6 7 8 ton cubical form
0.0 0.1 1 2 3 4 5 6 7 8 ton spherical form

Fig. 6.9

Apart from these data, information must be obtained on the density, the mechanical strength, the abrasive resistance and the chemical durability (in sea water!).

Before a prospective location can be selected to establish the quarry, it should be ascertained that the following requirements are met:-

- easy accessibility;
- volume of the formation must be enough to serve the whole job!
- blasting must be possible without excessive damage to human life or the environment in general;
- concessions must be made available;
- in the near vicinity of the quarry sufficient space should be available to open work yards, depots etc.

6.2.2 Operation of the quarry:

The planning of the quarry operation is mainly based on the expected fragmentation curve.
According to Fig. 6.9, 10% of each blast will be in blocks of 5 ton (5.10^4 N) and larger. Consequently it is necessary to blast 10 X ton of material to obtain X ton of blocks of 5 ton and larger.
The other 90% of the material must, however, also be classified, transported, stored and eventually be disposed of. In view of the cost involved it is often necessary to search for productive use of this finer material.
Fig. 6.9 shows at the same time the dramatic consequence of a slight deviation from the expected curve. This would double the quantity of material to be blasted in order to deliver the required X tonnes of stone larger than 5 ton. Therefore a test blast of up to 100.000 ton of material is a necessary investment in the pretender stage.

The mining operation itself should be done in a systematic way, following a pre-designed mining plan. During the blasting, bench-floors are created. The sequence of blasting depends on the overall pit slope (soil mechanical stability!).

The width on each bench floor should be sufficient to create working space for classification, loading and transport. (Fig. 6.10 and 6.11) See also [33]

mining plan 15 m benches
with topbenchfloor at 145+

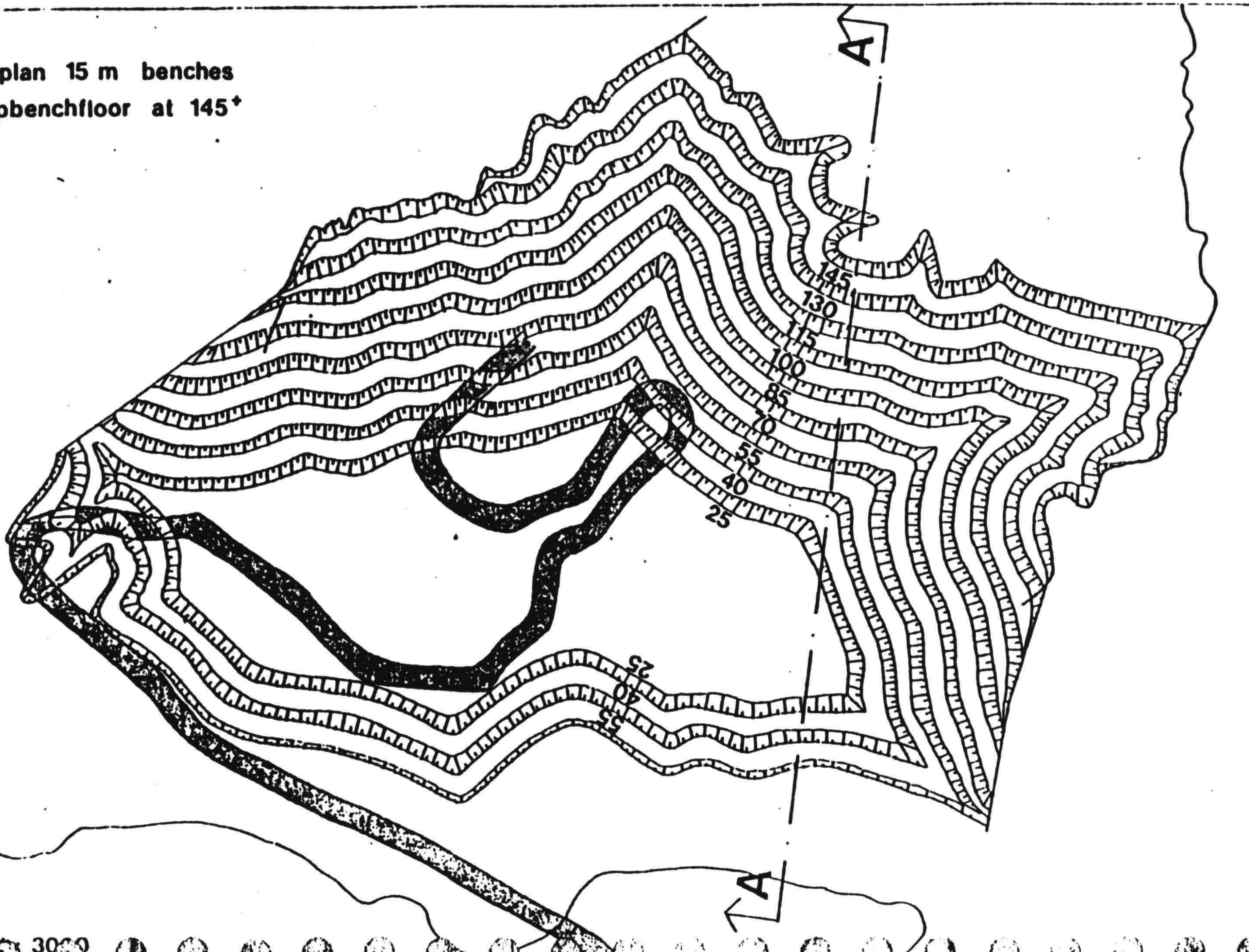


Fig. 6.10

scale 3000

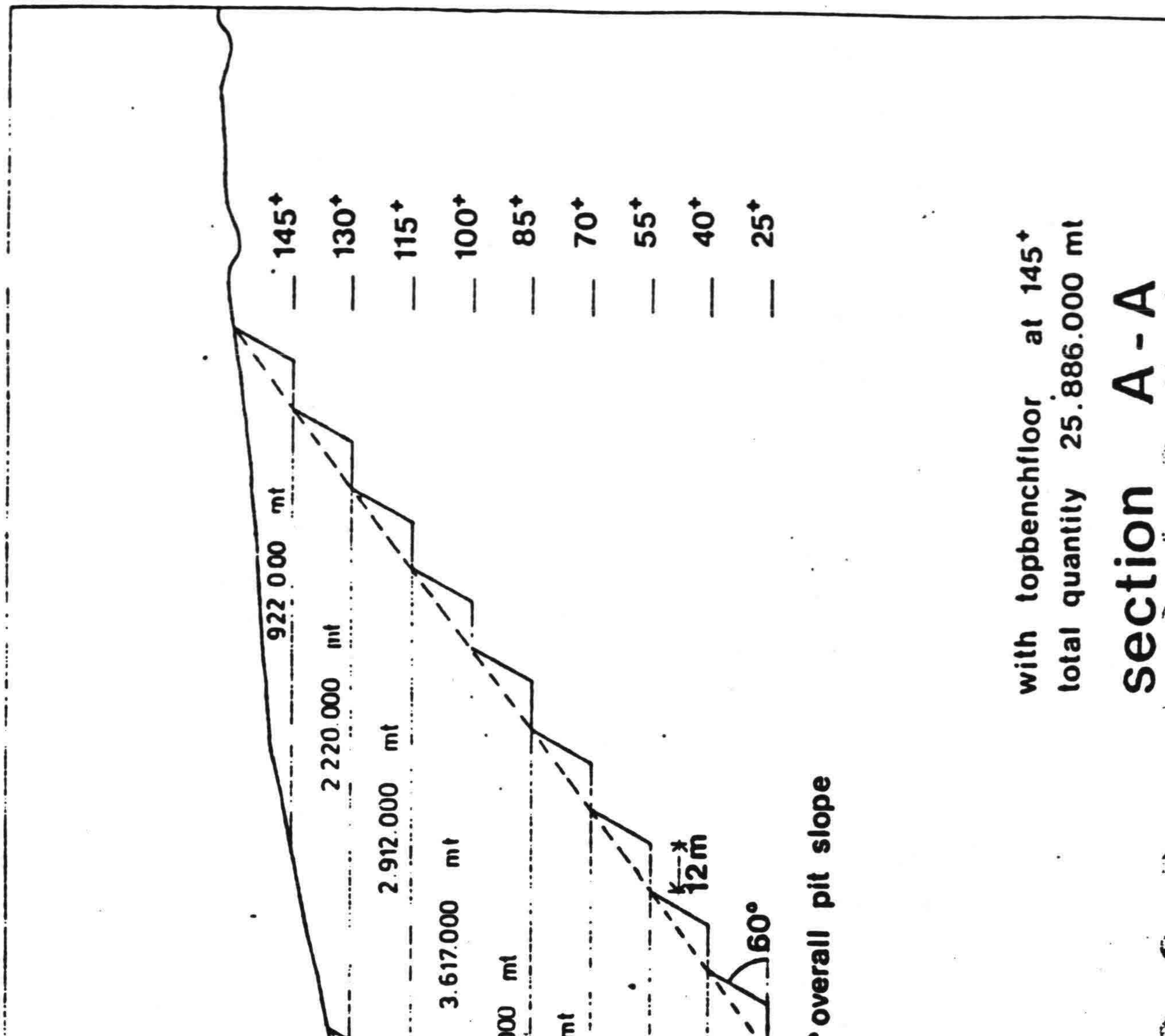

145+
130+
115+
100+
85+
70+
55+
40+
25+
135+
120+
105+
90+
75+
60+
922 000 mt
2 220 000 mt
2 912 000 mt
3 617 000 mt
3 627 000 mt
3 755 000 mt
3 848 000 mt
2 973 000 mt
2 012 000 mt
697 000
1 197 000
1 244 000
1 429 000
1 770 000
12 m
60°
36° overall pit slope
36° overall pit slope
elphine stone
mining plan 15 m benches
with topbenchfloor at 120+
total quantity approx. 6.337.000 mt
with topbenchfloor at 145+
total quantity 25.886.000 mt
section A - A
scale 1 : 1000

Fig. 6.11

./62

6.2.3 Transport:

For any major project -and the construction of a rubble mound breakwater certainly is one - it is necessary to create storage areas at the quarry and at the construction site if the distance between the two exceeds 20 to 30 km.
In this way, the three production units, i.e. the quarry, the transport and the actual construction can go on independent of each other.
The capacity of the storage areas should equal at least one month's production each.

Storage is generally done on land, even at the construction site. Only when absolutely no storage area on land can be found, one can consider storage under water. The cost of handling, however, is much higher in the latter case.

For the transport proper, one has a choice between:-

- transport by road;
- transport by rail;
- transport by water (sea or inland water);

or a combination thereof.

It is impossible to indicate a preferable transport system, because much depends on local conditions, available facilities and required extra investments.
In general, transport over water is far cheaper (4 to 5 times) per ton kilometer than transport by road or rail.

Again, it is emphasized that a calibrated weigh bridge is requisited either at the quarry or at the construction site. The purpose is dual:-

- base for payment between client and contractor;
- means for quality control.

6.3 Actual breakwater construction

In principle there are three methods to breng the material into the profile of the breakwater:-

- by floating equipment;
- by rolling equipment;
- by a combination.

6.3.1 Floating equipment:

The transport of the large quantities of materials can be done economically by floating equipment. Several types of barges can be used, differing only by the method of unloading:-

- split barges; (Fig. 6.12)
- bottom door barges; (Fig. 6.13)
- tilting barges; (Fig. 6.14)
- (hydraulic) side unloading barges (Fig. 6.15)

The major problems of floating transport are:-

- weather conditions (waves, fog);
- positioning;
- draught.

In general, due to their draught, barges can be used only for the parts of the breakwater which are more than 3 m. under water. The upper part can be constructed with the aid of crane barges. (Combined crane and transport barge, or transport barges and crane pontoon). (Fig. 6.16) Often te upper part of the breakwater is constructed with rolling equipment (see 6.3.2). Special attention is drawn to the possibility of jack-up crane platforms.
For easy and accurate manoeuvering, special drive systems for barges and tugboats are available such as Schottel and Voith-Schneider Propeller. (Fig. 6.17 and 6.18)

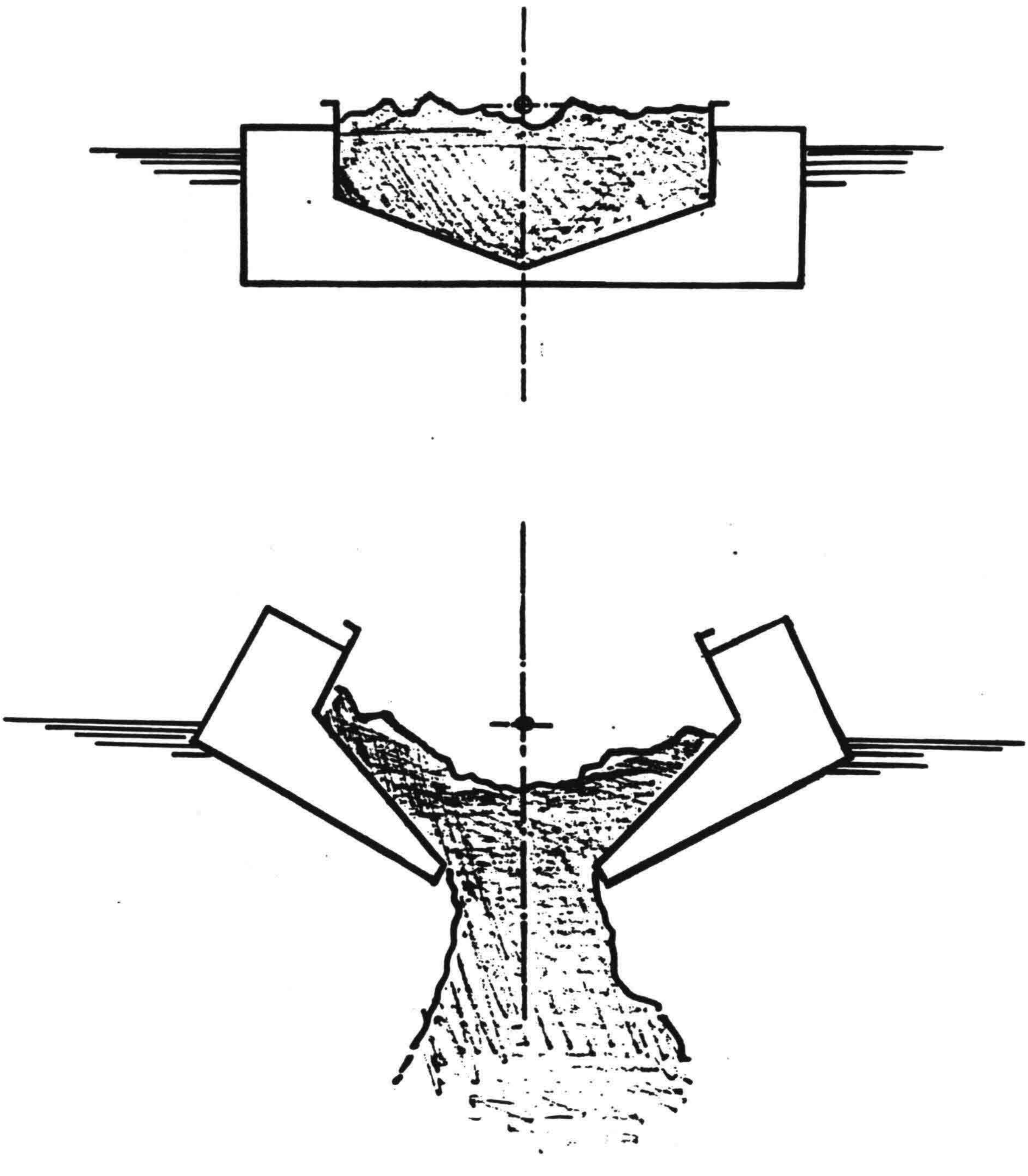


Fig. 6.12 Split barge.

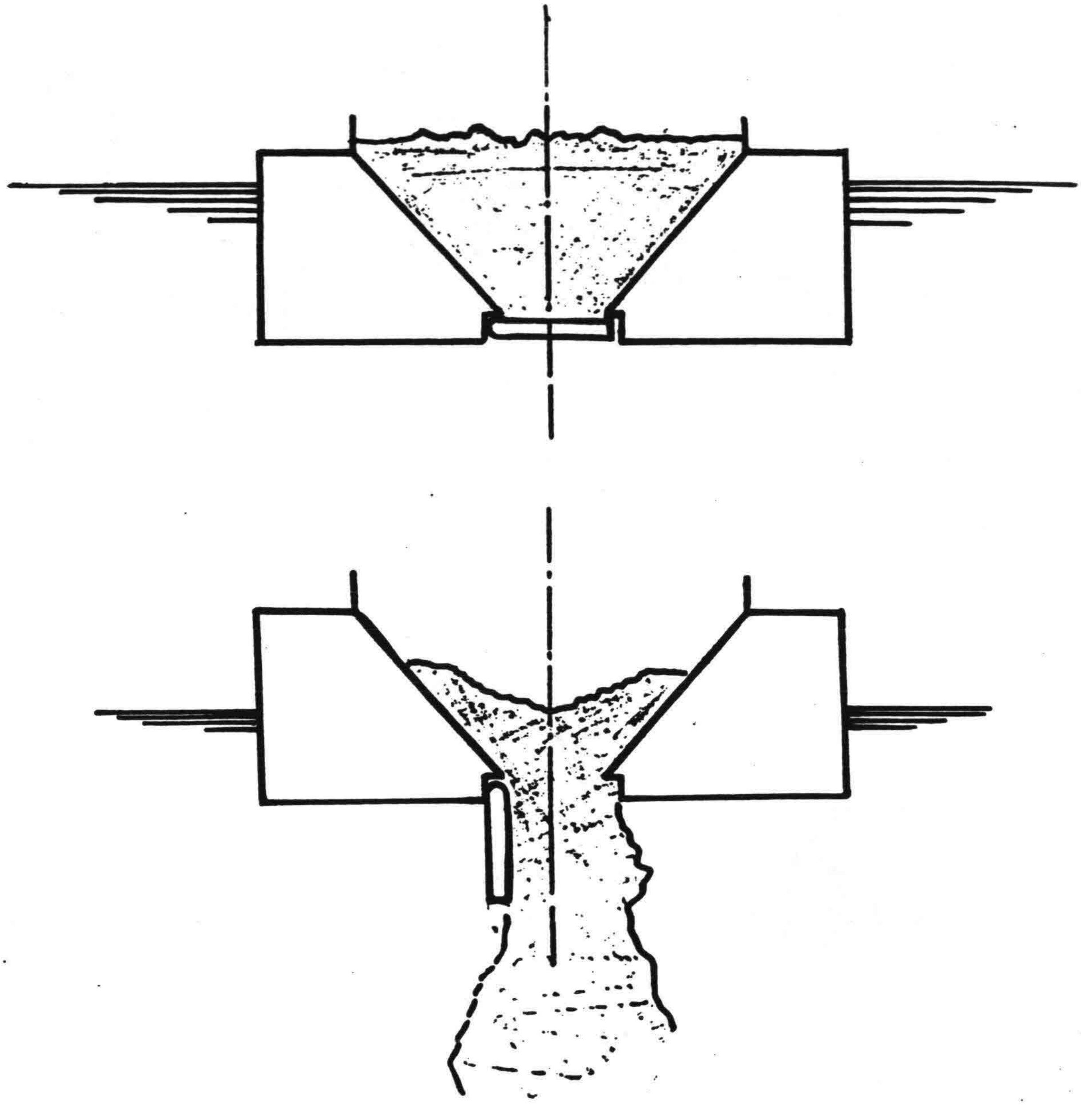


Fig. 6.13 Bottom door barge

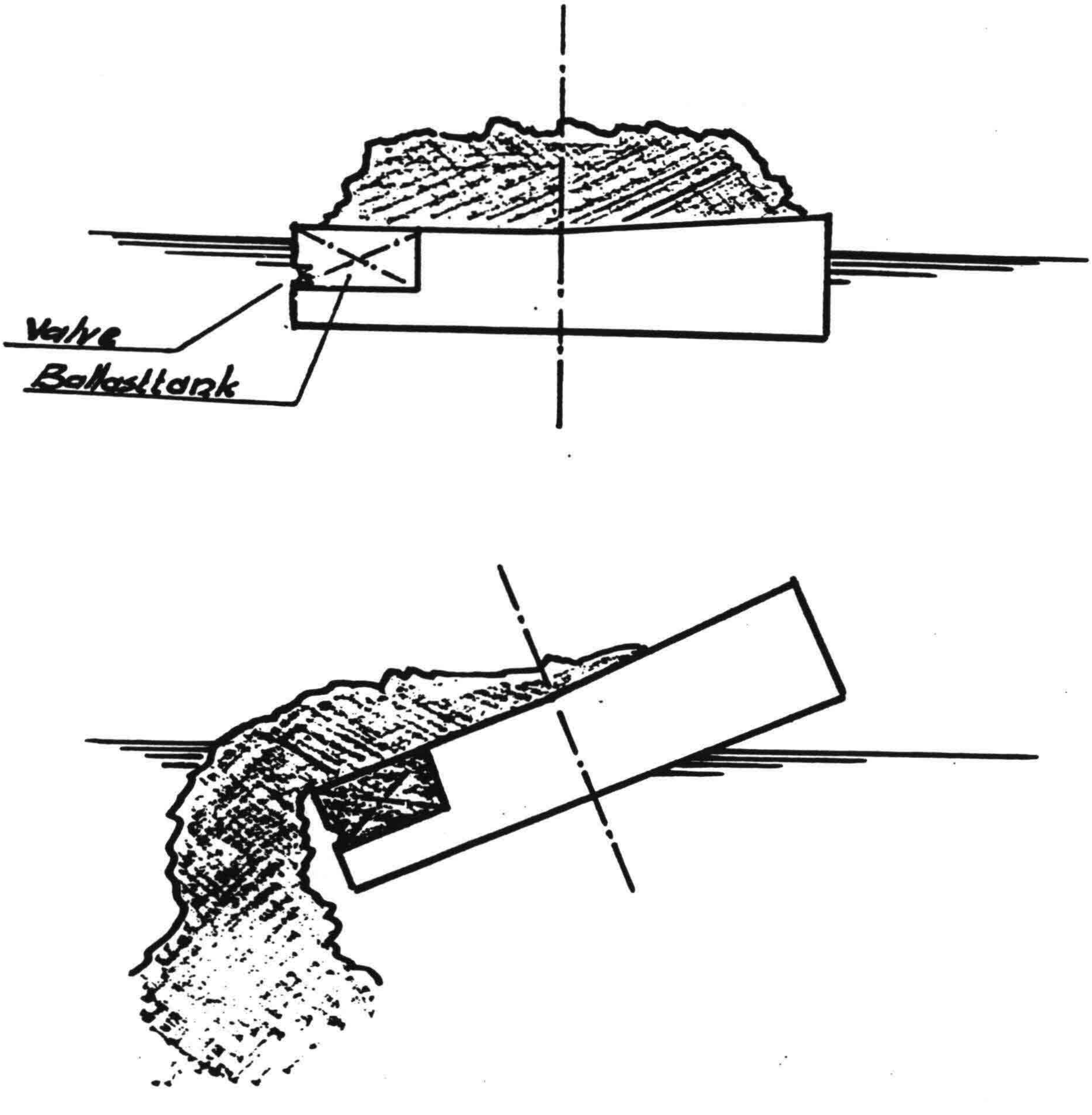


Fig. 6.14 Tilt barge

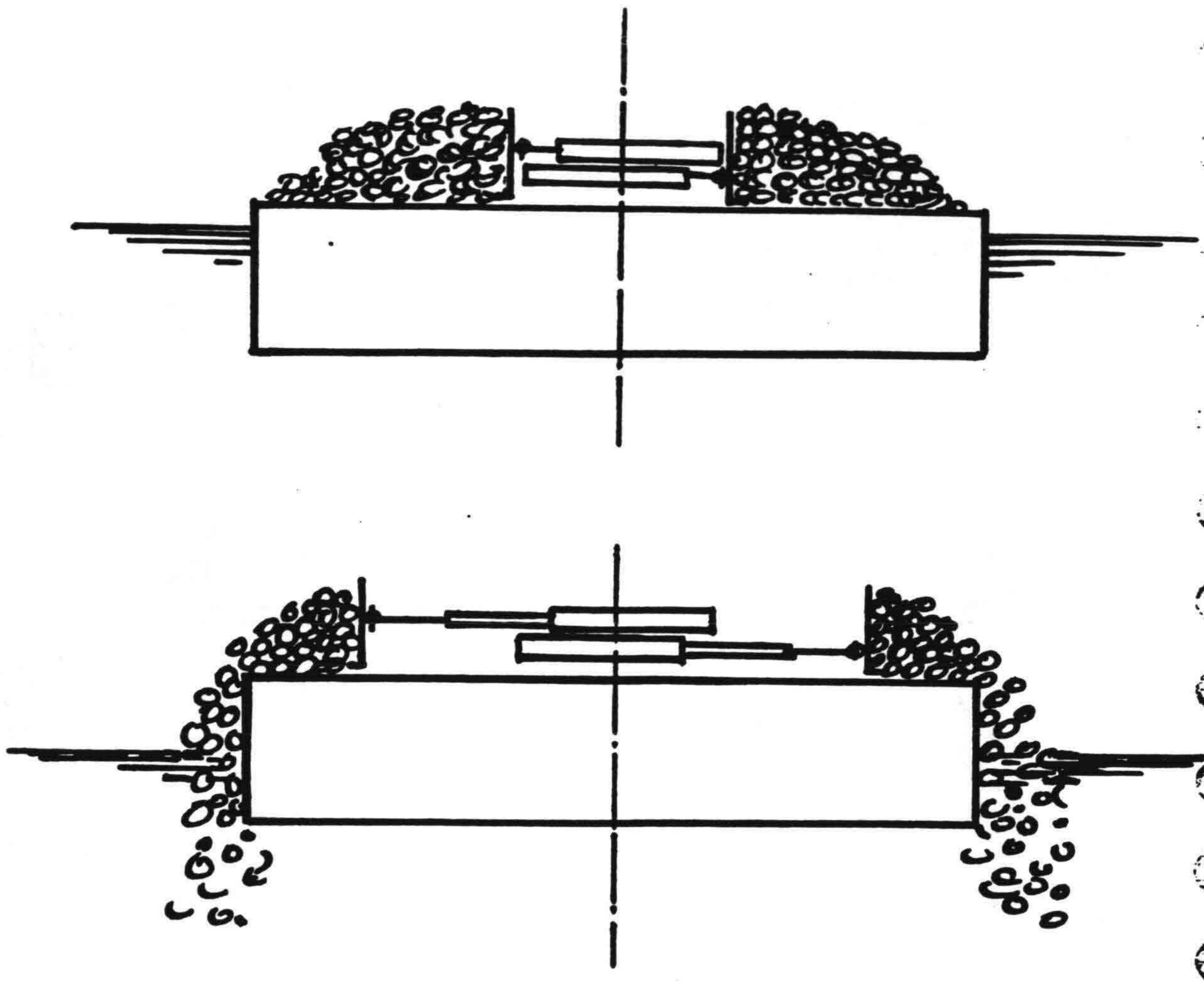
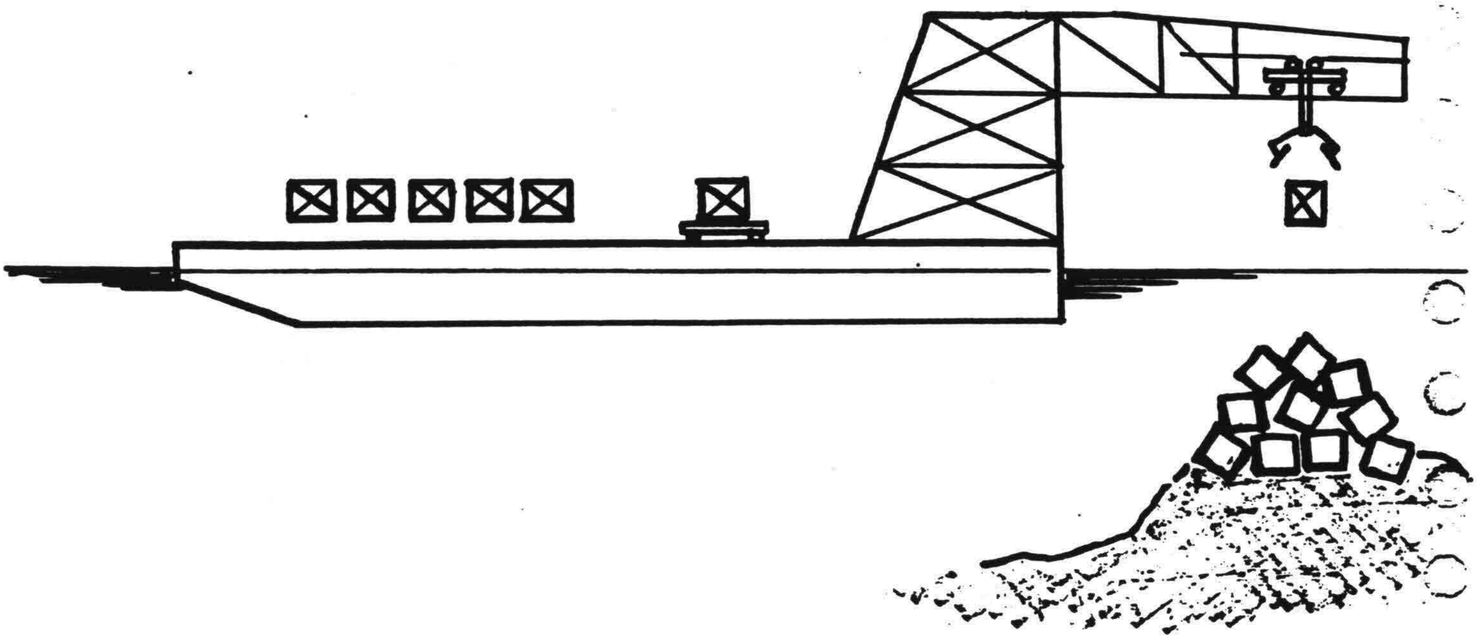
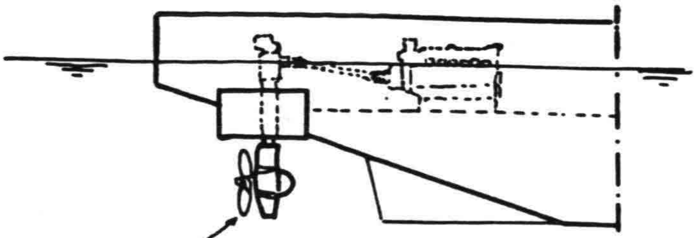


Fig. 6.15 Side Unloading barge



Fit. 6.16 Crane and transport barge



SCHOTTEL SYSTEM

Fig. 6.17

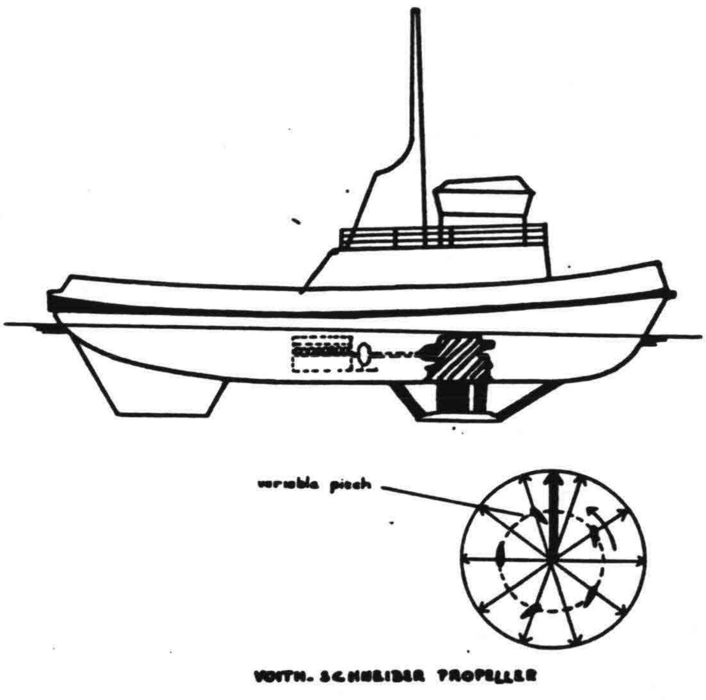


Fig. 3.18

6.3.2 Rolling equipment:

All material is transported over the crest of the breakwater by truck or train. (The crest is not submerged!) Material is dumped directly (core) or by means of a crane (armour units) at the front of the breakwater. This front moves slowly ahead. If the length of the breakwater is great, this often leads to organisational problems and a relatively long construction time.

If a crane is placed on the crest of the breakwater, its safety has to be assured, also during sudden storms in the calm season.
Sometimes the core and the lower part of the armour layer is constructed first because transport is easier over the relarively fine core material. Finally, the cap is placed when the crane is removed at the end of the works (Fig. 6.19 and 6.20)

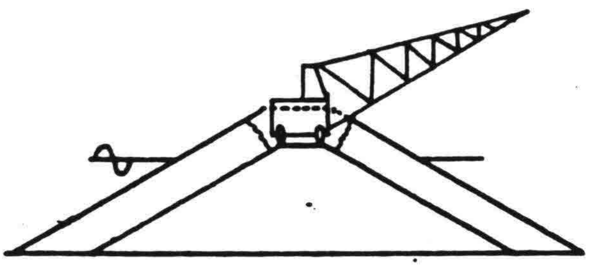


Fig. 6.19
Working ahead over core.

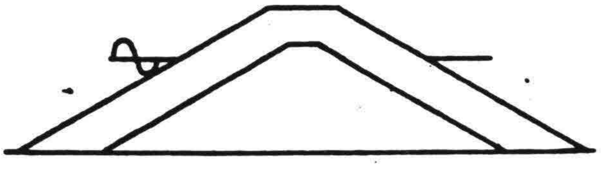


Fig. 6.20
Completed cross section after removing of crane

6.3.3 Combining rolling and floating equipment:

In the case rolling and floating equipment are both used, generally the filter layers and the lower parts of the core are constructed by floating equipment.
For the upper part one can combine:

floating transport	+	crane on the breakwater
rolling transport	+	floating crane or jack-up crane
rolling transport	+	crane on the breakwater

When the core of the breakwater is submerged, sometimes prefabricated concrete cap elements are used to create a safe roadway. The armour units are than transported over these cap elements.

6.4 Stability during construction

As a result of the continuous wave action and current it is possible (depending on the water depth) that the finer material of the breakwater (mass varying from 10 to 200 kg.) will scour of not protected. In this case the breakwater cannot simply be built layer by layer, starting with the core.

One way to do this is shown in the following figures. The material is put in place in the numbered sequence.
1, 3 and 5 are containing heavier stone, 2 and 4 are fines.

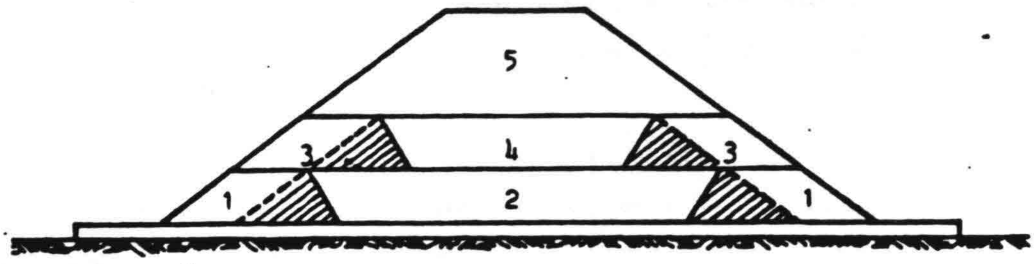


Fig. 6.21 Possible construction sequence of mound breakwater.

The disadvantage of this system is that the cross hatched areas contain the heavier stone which is more expensive. However, this is acceptable when the cost difference is not too great.
Another possibility, which provides a saving on the heavier stone but not on the labour, is the following:

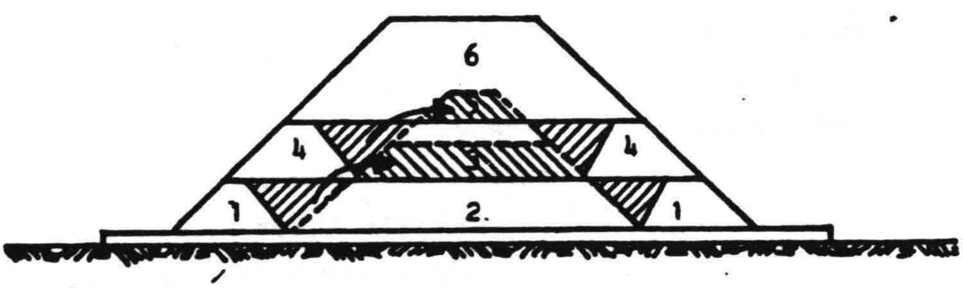


Fig. 6.22 Possible construction sequence of mound breakwater.

First the dams (1) are constructed. Between these two the finer material such as quarry run is dumped. Then the cross-hatched sections are dug out and backfilled with heavier material as in (1) after which dams (4) are constructed consisting of the same, or heavier, units than in (1). The material that has been dug out is used for the core in section (3) etc. The excavation has to be done during calm weather with a crane on a jack-up platform or on the completed section of the breakwater to enable the crane operator to determine the exact position of the bucket.

6.5 Examples

6.5.1 Europoort:

The recently built breakwaters for the harbour entrance of Hook of Holland consist of:

a) Noorderdam, which is basically an extension of the existing Noorderhoofd.
b) Zuiderdam, which is connected with the closure dam of the Brielse gat.
c) Separation dam between the Rotterdamse Waterweg (Rotterdam Waterway) and the new Caland Canal which is the entrance to Europoort.

To meet the demands of shipping regarding guidance of current, reduction of wave height and marking of entrance channels, it was necessary to build the Zuider- and Noorderdam up to a level of at least MSL + 2 m. The southern part of the Zuiderdam could be made out of sand because the gentle slope of the sea bottom forces the higher waves to break before they reach the breakwater.

The northern part of the dam is built in deeper water and the harbour complex behind it is protected by a second defense system between the breakwater and the complex.
This implies that mass overtopping will occur and that, consequently, the crest and the inner side of the breakwater will be subject to severe wave attack. To reduce the overtopping substantially, the crest would have to have been raised to a level of at least MSL + 7 meters. This appeared to be an uneconomical solution in view of the increase in cross-sectional area and the relatively expensive core material. Therefore the crest height was kept at a level of MSL + 2 m. (Fig.6.23)
Offshore conditions and economical reasons dictated the adoption of the steepest possible slope which turned out to be 1.5 : 1.
To reduce the resulting high rate of wave reflection, it was recommended to make use of aprons and a cover layer with a high degree of porosity.

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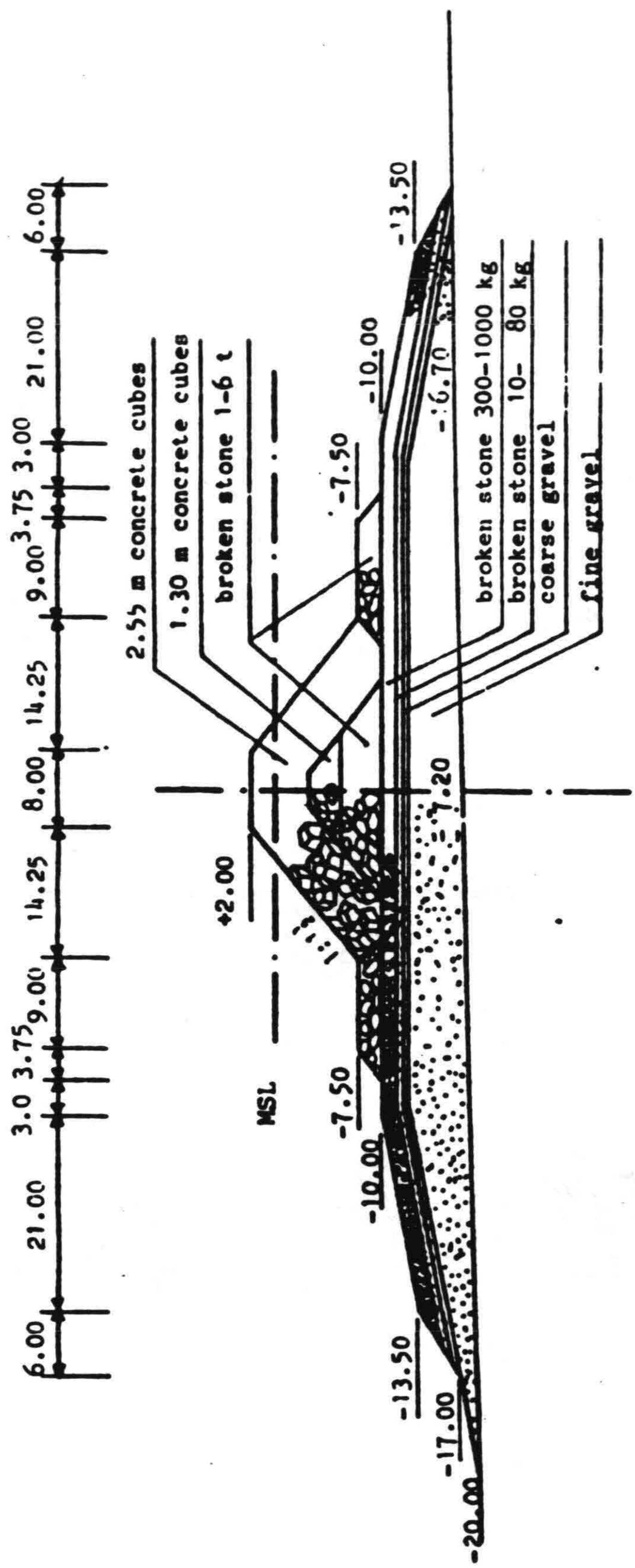

Cross section Noorderdam
Hook of Holland
Scale 1:600

Fig. 6.23

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The following types of breakwaters have been considered and tested in the laboratory:

1) Caissondam (vertical breakwater)
2) Composite breakwater
3) Mound breakwater with a cover layer of pell-mell placed concrete blocks.

Type 3 was finally selected because this type required the least complicated construction method. This method would also suffer the least amount of down-time because of weather conditions.
The construction phases of the Noorderdam (northern-breakwater) in Hook of Holland was executed as shown in figure 6.24.

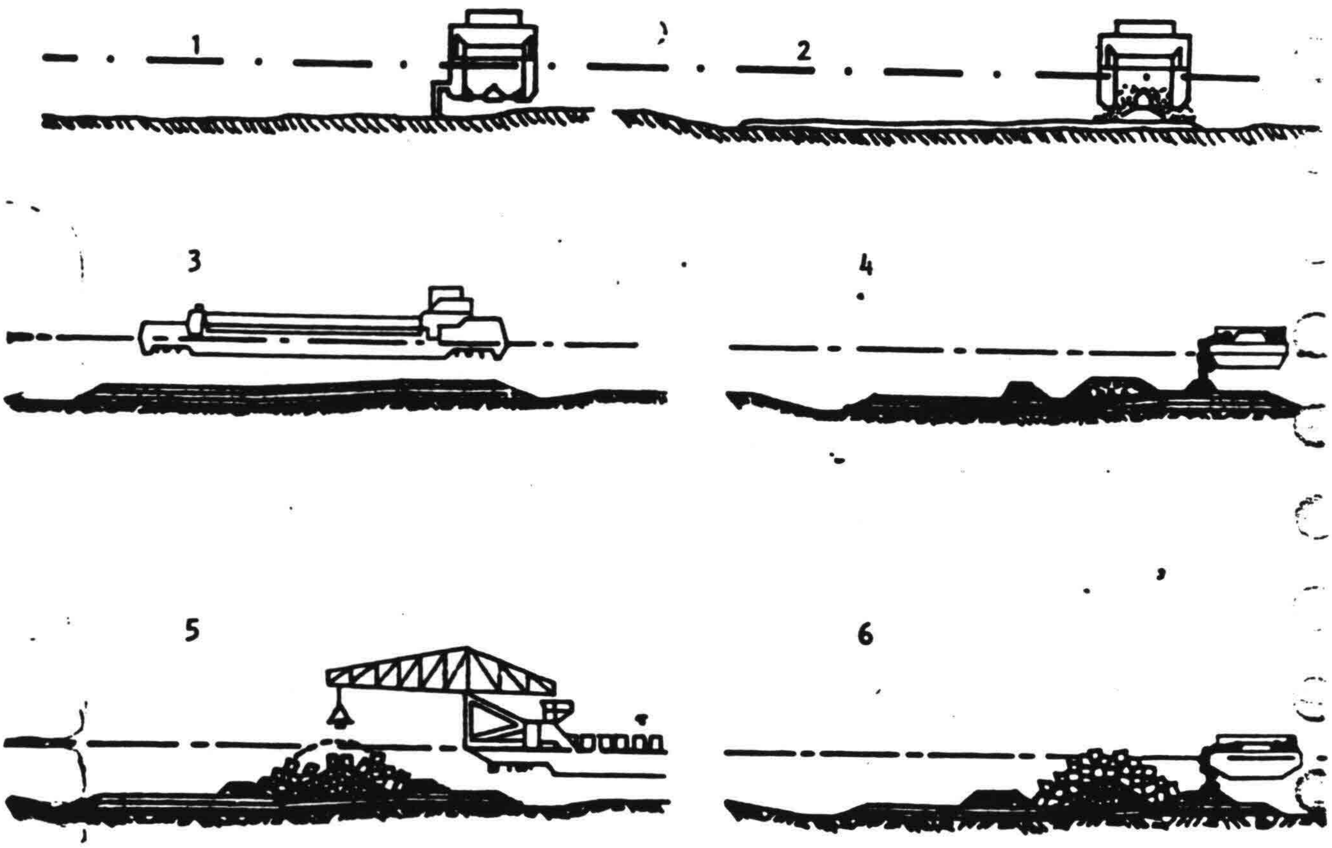

Fig. 6.24 Construction phases Noorderdam

1. Dredging to remove silty material
2. Construction of 1st. filter layer (sea gravel)
3. Construction of 2nd. filter layer (alluvial gravel) and small rock
4. Placing boulders 1-6 tons
5. Placing armour units
6. Placing apron consisting of boulders 1-6 tons

./75

6.5.2 IJmuiden:

As another example the development - and history - of the design of the breakwater of IJmuiden will be described. The various cross sections which were considered are given in figure 6.25, 1) through 9):

1) The original cross section. Failure occurs due to damage on the harbour side of the crest (inner slope).
2) In order to avoid this, these armour units have been removed. In order to protect the much lighter rock blocks of one to five tons under the first cover layer, these rock blocks have been penetrated with asphalt.
   Due to the typical lay-out of the moles, waves will reach the inner slope of the mole almost parallel to the breakwater, with the result that the armour units on the inner slope just below the water surface are attacked.

3) In order to avoid the necessity of penetration, the cap construction of rock asphalt has been extended below water level. When the armour units on the inner slope move (due to the oblique waves) the stability is endangered.
4) In order to overcome these difficulties, the entire inner slope is made from rock asphalt. The disadvantage of this solution is, however, that the layer can be lifted due to pressure differences across this layer. This layer, therefore, has to be of sufficient thickness and weight.

   Note 1: When a decision was taken on the cross-section, the model technique had not yet progressed to the extent that the plastic mass of (impermeable) stone asphalt could be reproduced in the model. Therefore, the initial decisions on the design in stone asphalt were based upon calculations only.

5) For this reason the inner slope is not covered completely with rock asphalt, but only in spots. These spots increase the stability sufficiently without the danger of uplifting.
6) In order to avoid or to decrease the uplift forces, the cover of rock asphalt has been extended to the inner and outer slope of the breakwater.

7) Since this breakwater does not suffer from overtopping it can also be lowered.

Note 1: The length of the scouring protection greatly influences the breakwater stability [24]

8) In a later stage of this design development, the crest has again been made higher in order to enable the transport of construction materials over this crest to the cranes standing at the construction area at both sides of the breakwater.

9) This figure shows the savings in material (by double hatching).

Note 2: In the meantime, the design proved to be of insufficient stability, which was, afterwards, confirmed by model tests. To increase the stability, the asphalt cover layer was covered with concrete cubes, approximately according to the double hatched area of 9). See also [26]

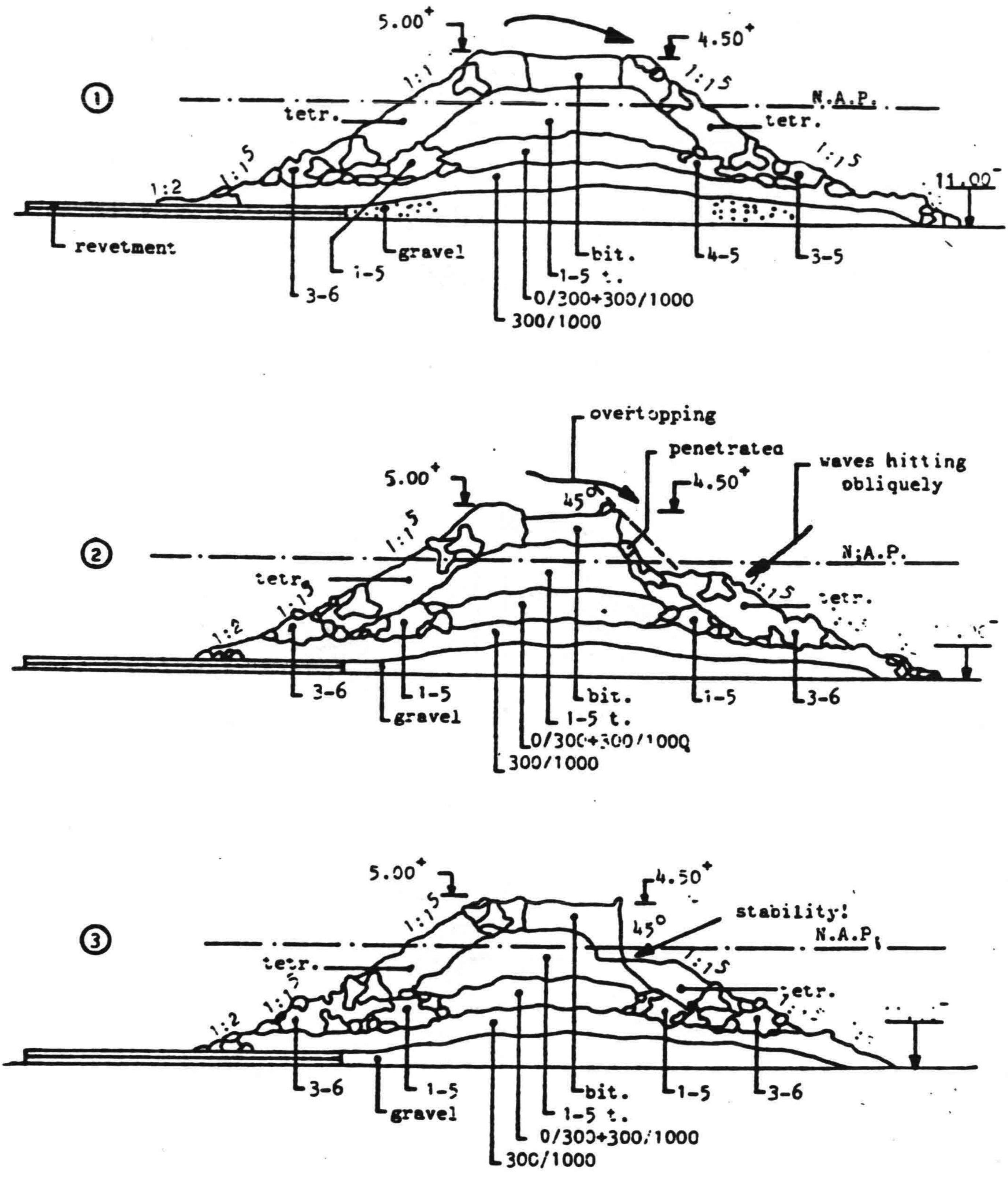
①
5.00+
4.50+
1:1
1:1.5
N.A.P.
tetr.
tetr.
1:1.5
1:2
1:1.5
11.00-
revetment
gravel
bit.
4-5
3-5
1-5
1-5 t.
3-6
0/300+300/1000
300/1000
②
overtopping
penetrated
waves hitting obliquely
5.00+
4.50+
45°
1:1.5
N.A.P.
tetr.
tetr.
1:1.5
1:2
1:1.5
3-6
1-5
bit.
1-5
3-6
gravel
1-5 t.
0/300+300/1000
300/1000
③
5.00+
4.50+
1:1.5
stability!
45°
N.A.P.
tetr.
1:1.5
tetr.
1:1.5
1:2
3-6
1-5
bit.
1-5
3-6
gravel
1-5 t.
0/300+300/1000
300/1000

Fig. 6.25

./78

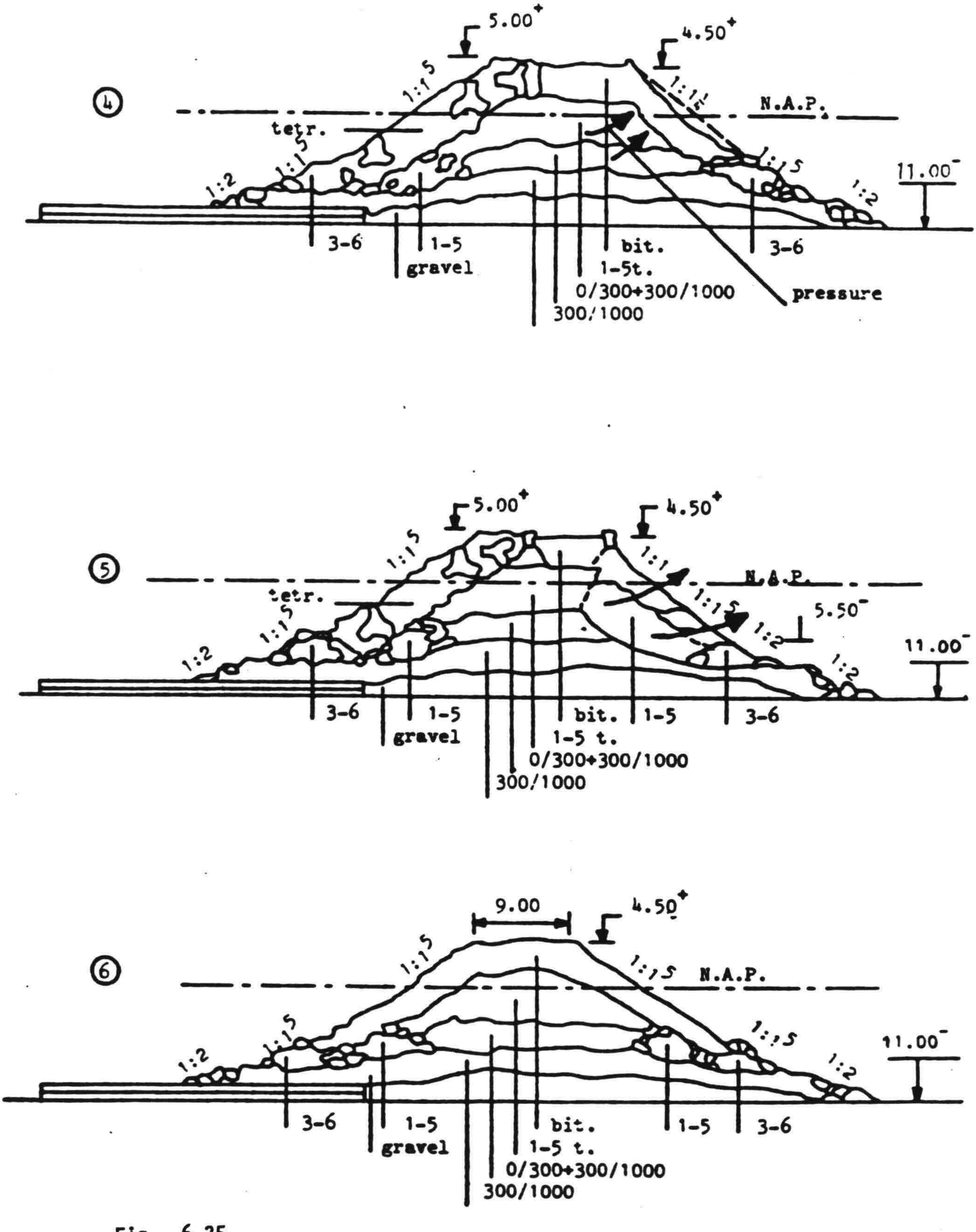
4
5.00+
4.50+
1:1,5
1:1,5
N.A.P.
tetr.
1:1,5
1:2
1:1,5
11.00-
1:2
3-6
1-5
gravel
bit.
1-5t.
0/300+300/1000
300/1000
3-6
pressure
5
5.00+
4.50+
1:1,5
1:1
N.A.P.
tetr.
1:1,5
1:1,5
5.50-
1:2
1:2
11.00-
1:2
3-6
1-5
gravel
bit.
1-5
3-6
1-5 t.
0/300+300/1000
300/1000
6
9.00
4.50+
1:1,5
1:1,5
N.A.P.
1:1,5
1:2
1:1,5
11.00-
1:2
3-6
1-5
gravel
bit.
1-5 t.
0/300+300/1000
300/1000
1-5
3-6

Fig. 6.25

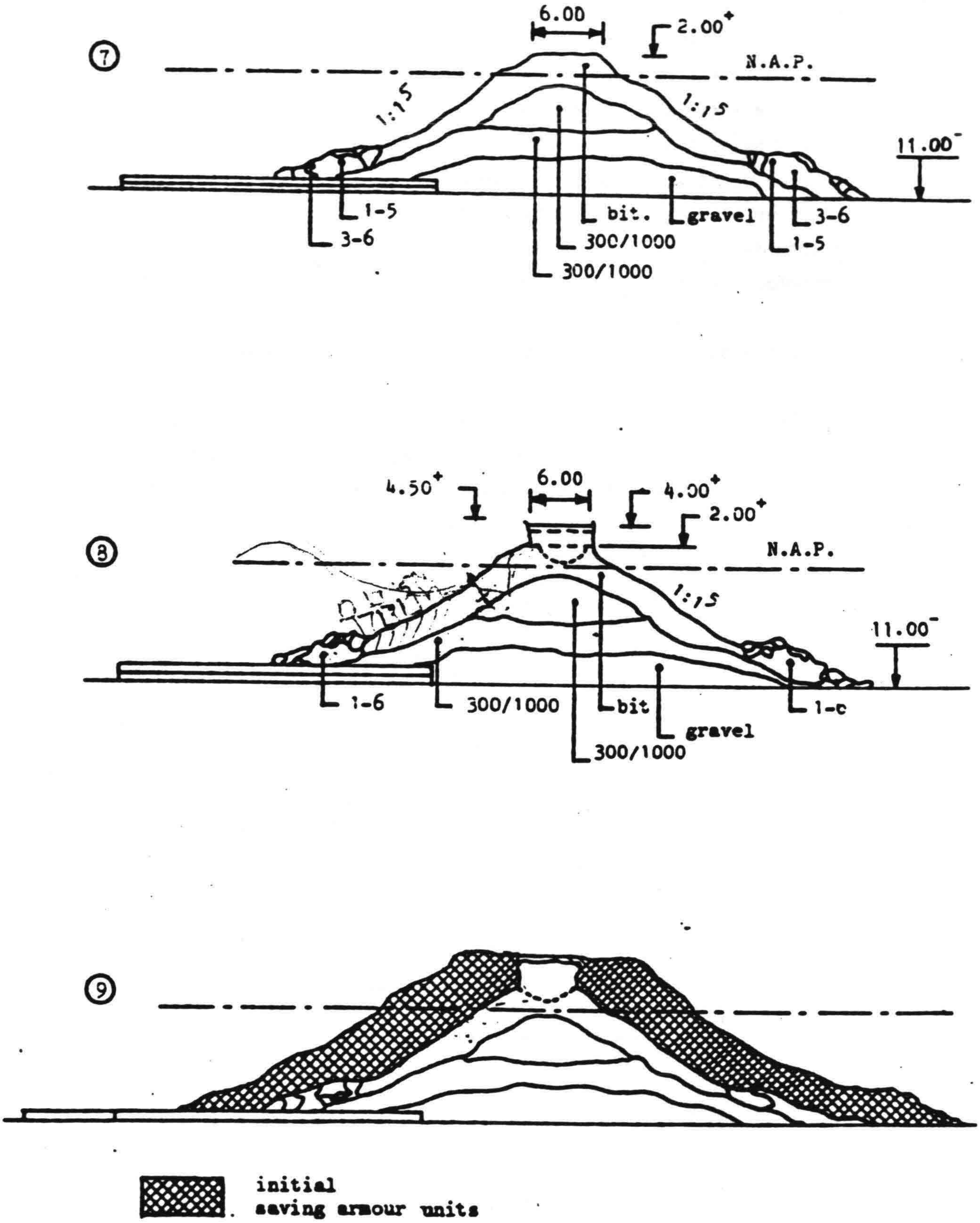
7
6.00
2.00+
N.A.P.
1:15
1:15
11.00-
1-5
3-6
bit.
gravel
300/1000
300/1000
3-6
1-5
8
4.50+
6.00
4.00+
2.00+
N.A.P.
1:15
11.00-
1-6
300/1000
bit
gravel
300/1000
1-c
9
initial
saving armour units

Fig. 6.25

6.5.3 Scheveningen (Fig. 6.26):

Originally it was proposed to construct the breakwaters for the fishing harbour of Scheveningen following a modified IJmuiden design.

1) The disadvantages of the IJmuiden design, i.e. the lifting up of the slabs was solved by constructing a fully asphalt grouted superstructure above the level of -1 m., i.e. in the area where grouting could be done dry above water.

Model tests, however, showed that the toe at -1 m. would not be stable. It was proposed to grout also the toe 2). This was rejected because grouting below the level of L.W. was not accepted.

Therefor, the toe was then protected with cubes. 3) This design proved to be relatively uneconomic because both the grouted superstructure and the cubes were designed to withstand the wave forces.

Finally design 4) was selected and constructed. For the deeper parts, the design was adapted to the more severe wave attack by heavier armour units and by a strong berm.

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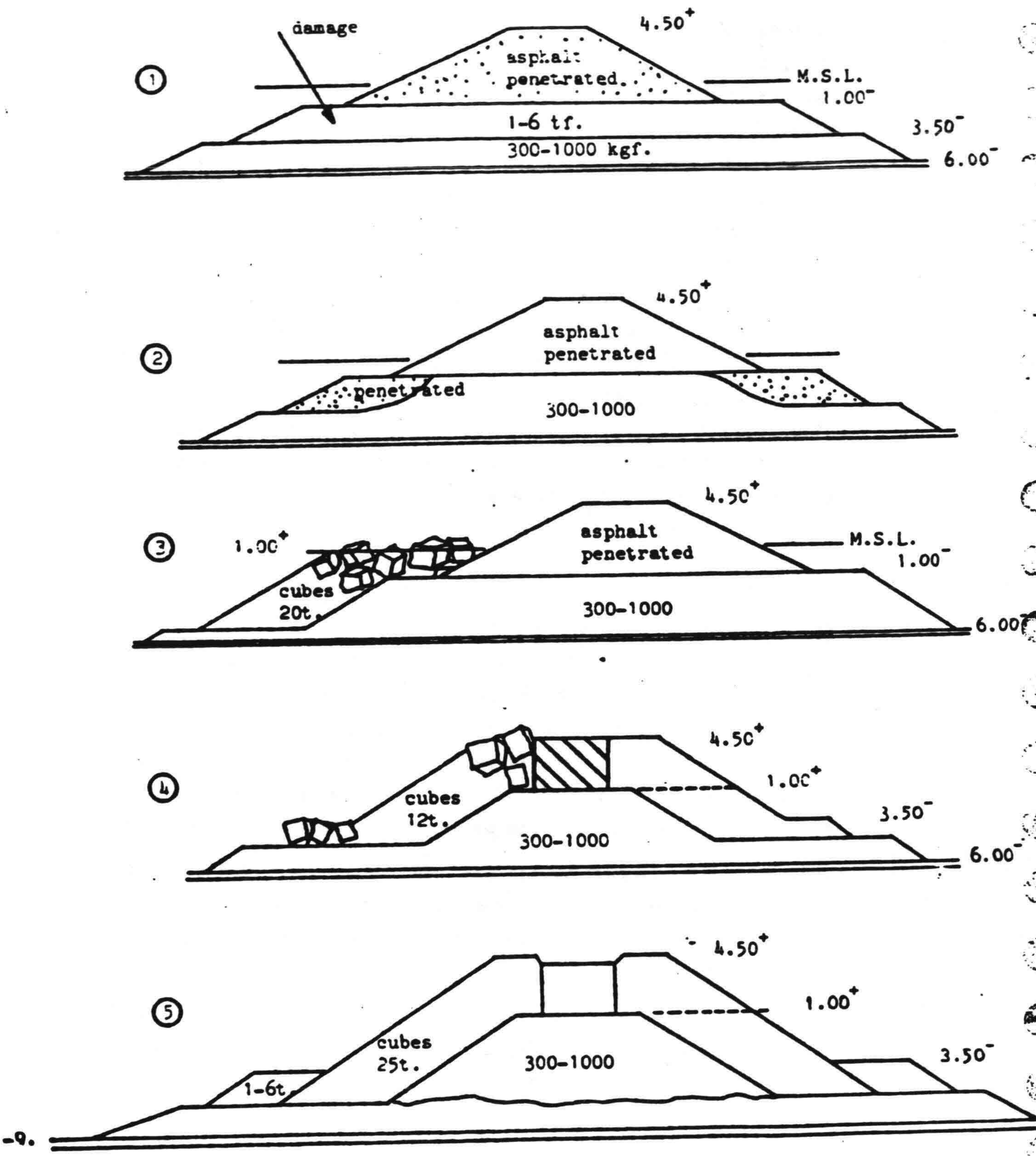
1
damage
4.50+
asphalt
penetrated
M.S.L.
1.00-
1-6 tf.
3.50-
300-1000 kgf.
6.00-
2
4.50+
asphalt
penetrated
penetrated
300-1000
3
4.50+
1.00+
asphalt
penetrated
M.S.L.
1.00-
cubes
20t.
300-1000
6.00-
4
4.50+
1.00+
cubes
12t.
3.50-
300-1000
6.00-
5
4.50+
1.00+
cubes
25t.
300-1000
3.50-
1-6t
-9.

Fig. 6.26

7. OPTIMUM DESIGN

7.1 Balanced design

In order to obtain a design which leads to the minimum overall cost, it is necessary that every piece of material in the breakwater fulfills its function.
This can be compared with a normal structure, such as a frame, which is designed in such a way that all members are loaded to the same rate and failing under the same (over)load.
In the same way it can be attempted that all materials in the breakwater lead to a failure for the same wave height. In this respect it is not the no-damage wave height H_{so} which is important, because a certain exceedance of H_{so} may lead for some elements to a slight damage only and for others to a complete failure, eventually causing failure of the overall stability. It is clear that all materials are used economically if they complete failure at the same wave height H_j, which is of course greater than the no damage (design) wave height H_{so}.

7.2 Selection of design wave height

It is very complicated to select the design wave height, i.e. the wave height for which the structure should have no damage. Often this design wave height is selected in a subjective way. Although the method described in the following includes a great number of question marks, it is given because it is the only rational approach towards the selection of the design wave height.

This method of optimum design has been discussed in principle in a previous section. The four most important factors for determining the minimum total breakwater cost will now be discussed and illustrated:

a The frequency of occurrence of different off-shore conditions. To describe the wave attack, use is made of a probability curve giving significant wave heights to be expected.

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This curve can be constructed after a programme of wave measurements has been carried out. As an alternative, it can be constructed from meteorological data using wave forecasting techniques.

b The relationship between off-shore condition and structure performance. Because wave forces are difficult to determine, the significant wave height, H_s, is taken as a characteristic wave attacking the structure.

This still implies that the known distributions of wave heights and periods are applied. This enables us to define a H_{so} for the structure. H_{so} is the maximum wave height which can satisfy the no-damage criterium. When the design wave height H_{so} is exceeded, a number of armour units will be moved from their placed position. The point of this work is to establish the relationship between the percentage of armour units moved (percent damage), the design wave height (H_{so}), and the actual incident wave (H_s). One complication to this, however, is that time is also an important factor. Until recently model tests could only be carried out with regular waves which made it difficult to relate the model to the prototype.

Several investigations indicated that the H_s of a series of irregular waves is well represented by a H_r of the same height of a series of regular waves.
For some years now several laboratories have a so-called wind wave flume with a programmed wave generator at their disposal which produces irregular waves.
The relationship between the damage caused by a certain H_s can, thus via the tests, be related to the design wave H_{so}. H_{so} is directly related to the shape and dimensions of the breakwater.

c Relationship between construction cost and H_{so}.
The initial costs will, to a large extent, depend upon the quantities of the different kinds of rock, and also on the size of the material used, especially the armour units. For the purpose of simplification, the influence of size is neglected for the moment; we can say for the construction cost I: $I = f(H_{so})$.

d The relationship between expected damage, off-shore conditions and H_{so}.
A breakwater designed for a given H_{so} is damaged whenever the H_{so} is exceeded. The damage to be expected depends upon the probability of occurrence of waves higher than the waves of the storm characterized by the H_{so}.
To determine the damage to be expected it is assumed that an insurance company is willing to insure the structure against damage. When the company covers a large amount of constant risks, which are unrelated, then the premium is s, where s is the probability of damage multiplied by the repair costs.
In this case constant risk means that all damage is repaired immediately. This premium s which would have to be paid for a single structure and which would have to function indefinitely, is equal to the average amount that would have to be paid each year to repair the damage.
When the H_{so} is exceeded by the amount $\Delta H_{s(i)}$, this is accompanied by an amount of damage $\Delta W_{(i)}$. The probability of this happening each year is $\Delta p_{(i)}$. Then the average yearly total damage is $s = \sum^{n} \Delta p_{(i)} \times \Delta W_{(i)}$ when other factors such as inflation are neglected. $\Delta W_{(i)}$ also contains economic losses due to for instance non availability of the harbour after extensive damage.
The amount $s_{o(b)}$, which has to be reserved now to pay the damages b years from now, can be calculated with the method of compounded interest.

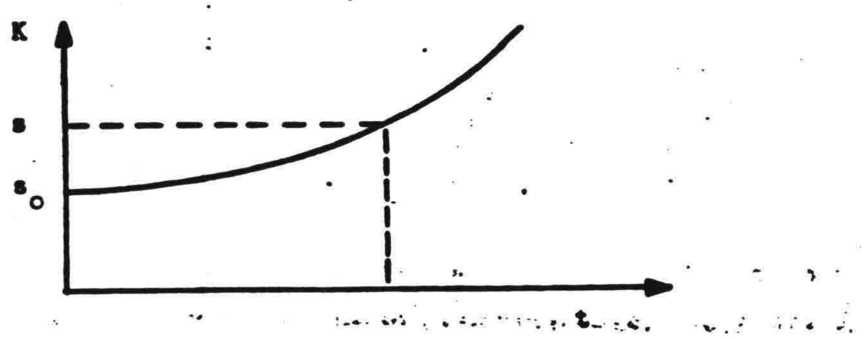

Fig. 7.1 Growth of capital with compound interest.

$S = \sum \Delta p_{(i)} \times \Delta W_{(i)}$

$$\frac{dK}{dt} = \frac{\delta}{100} K \qquad \frac{dK}{K} = \frac{\delta}{100} dt$$

$$\int_{s_{o(b)}}^{s} \frac{dK}{K} = \int_{o}^{b} \frac{\delta}{100} dt$$

$$\ln K \Big|_{s_{o(b)}}^{s} = \frac{\delta}{100} t \Big|_{o}^{b} \qquad \ln s - \ln s_{o(b)} = \frac{\delta}{100} * b$$

$$\frac{s}{s_{o(b)}} = e^{\frac{\delta}{100} * b}$$

$$s_{o(b)} = s\, e^{-\frac{\delta}{100} * b}$$

The amount $s_{o}(20)$ (the amount we have to reserve now to pay the amount s twenty years from now) is, of course, less than the amount $s_{o(10)}$.

The total amount S we have to reserve now to pay the damage during the lifetime, T, of the construction is:

$$S = s_{o(1)} + s_{o(2)} + s_{o(3)} + \ldots\ldots + s_{o(t)} + \ldots\ldots + s_{o(T)}$$

$$= \int_{o}^{T} s_{o(t)}\, dt = s \int_{o}^{T} e^{-\frac{\delta}{100} * t}\, dt.$$

$$= s \frac{100}{\delta} \left(1 - e^{-\frac{\delta T}{100}}\right).$$

$$T = 100 \text{ years: } S = s \frac{100}{\delta} \left(1 - e^{-\delta}\right) \approx s \frac{100}{\delta}$$

$$T = 10 \text{ years: } S = s \frac{100}{\delta} \left(1 - e^{-\frac{\delta}{10}}\right) \approx 0.35 \frac{100}{\delta} \cdot s$$

$\delta = 3.5\%$

So the total cost Fl of the breakwater is the initial construction cost I + the capitalized value of the total damage ~~100~~ S

$$
\begin{aligned}
Fl &= I + S \\
&= f(H_{so}) + \frac{100}{\delta} \, . \Sigma \Delta p_i \, . \Delta W_i
\end{aligned}
$$

if the expected lifetime of the structure is 100 years or more.
This computation has to be executed for all cross sections (based on varying design waves). In principle, all cross sections are exposed to the same wave programme, i.e., waves occurring in the prototype. For every cross section, however, waves lower than the design wave are of no importance, and for waves higher than about 1,5 H_{so} the breakwater is completely destroyed. So the various breakwater cross sections to be tested are exposed to a certain, and to the design wave related part of the total probability spectrum of the wave climate.
The method described is suitable only to draw a comparison between different breakwaters as far as the monetary consequences are concerned. It should be noted that governmental agencies do not reserve money to repair future damage to their projects: they raise the necessary money at a certain time in a different way (as taxpayers know).
It is not realistic to suppose that all damage is repaired immediately. To a certain extent the breakwater shows a degree of flexibility depending upon the shape of the armour units. When one armour unit has moved away, the surrounding units tend to move in order to fill the created gap. It is up to the engineer to determine when repair is necessary. It is better not to repair too often, as mobilization for maintenance work can be more costly than the repair job itself.

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7.3 Numerical examples

The following factors are neglected in this example:

a Whether there is enough natural rock of the desired size available or whether it is desirable to use some kind of artificial concrete block (cube, Akmon or Tetrapod). The principle of optimal design is not changed this way even if it has an influence on the construction cost.

b Whether overtopping is allowed or not. If harbour activities permit no overtopping then the crest will have to be at a high level. In this case the inside berm is defended against wave attack. When a high waterlevel (wind set up + high tide) coincides with high waves, damage can be extensive to the inside berm.
Low crest level will result in a cover layer almost as strong on the inside as it will be on the outside berm. Consequently, high water levels will not be that important.

c Construction methods. Developments in this field are occurring so rapidly that to include this cost in these examples would not be realistic.

This example discusses a possible breakwater construction for use in Europoort. The next figure shows a cross section of one breakwater design of a long series that was tested in the laboratory.

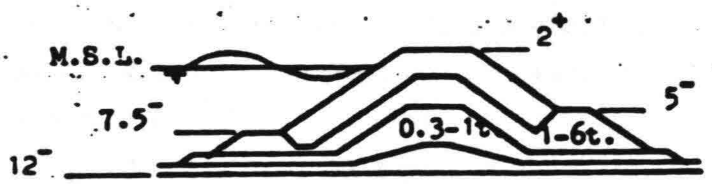

Fig. 7.2 Structure with cover layer of concrete blocks.

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The principles involved in the design of this cross section have been discussed previously. The cover layer exists of a concrete Akmons having a mass density of 2800 kg/m3.
The data of the off-shore conditions in the planned area of the prototype were obtained from wave recording stations in the North Sea. A probability distribution curve of H_s was derived from data that described wave conditions in terms of "the number of storms in which a certain H_s was exceeded". (see below).

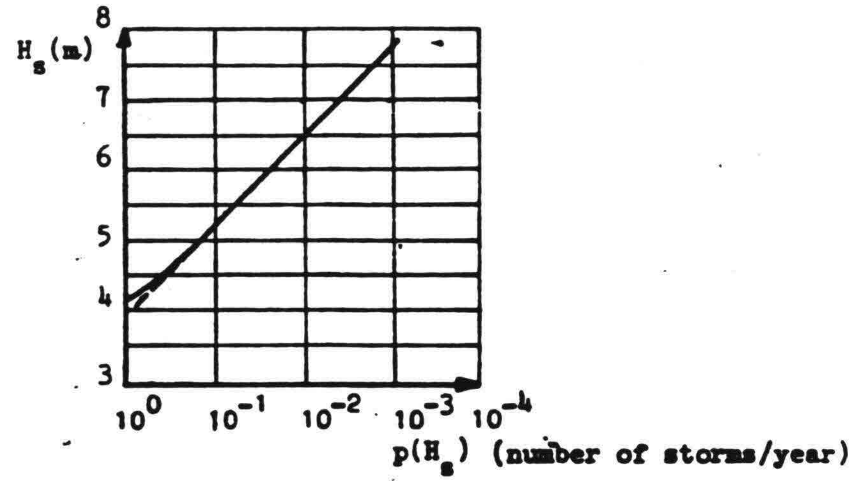

Fig. 7.3 Probability of excess of H_s.

Relationship between off-shore conditions and behaviour of the structure:

According to the theory explained earlier, the amount of damage to the breakwater occurs as shown in the figure below in case H_s exceeds H_{so}. This relationship was established in the laboratory. Here, the percentage of damage refers to the whole concrete cover layer.

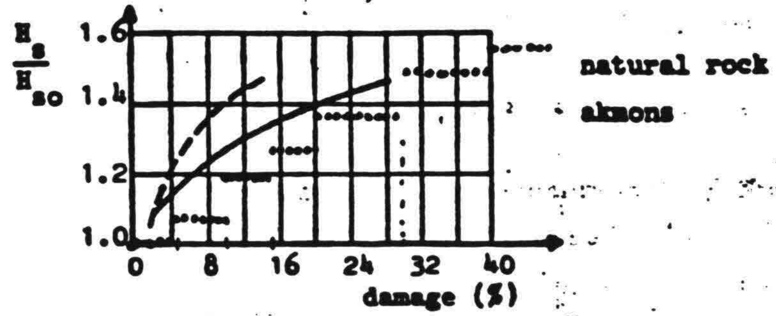

Fig. 7.4 Damage as a function of $\frac{H_s}{H_{so}}$ (experimentally determined)

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Arbitrarily, it was assumed that the structure will collapse when the Akmons suffered 10% damage; hence, from the figure 7.4, when $H_s/H_{so} = 1.45$. In earlier tests, it was established that when the slope of the face is 1.5 : 1 with Akmons of $\rho = 2800$ kg/m3, the required mass of the units was:

$$W = \frac{2{,}8\ (H_{so})^3}{50}$$

Relation between cost of construction and H_{so} ($Fl = f\ (H_{so})$):

The cost of construction can be divided between the cost of the cover layer and the cost of the secondary layer and core.
The latter two are independent, to a large extent, of the design wave height. These latter two were estimated to cost f. 8,620.-- per running meter.
The cost of the cover layer was estimated to be equal to $1320 \times H_{so}$. (For a detailed report on this see publication 31 of the Delft Hydraulics Laboratory by Van der Kreeke and Paape) [18]
Consequently, the construction cost of the structure per meter is:

$$Fl = 1320\ H_{so} + 8620$$

Relationship between anticipated damage, off-shore conditions and design wave:

$$S = \frac{100}{\delta}\ \Sigma\ \Delta p.\Delta W$$

In this particular case three intervals of $\frac{H_s}{H_{so}}$ were tested in the laboratory, namely:

$1 < H_s/H_{so} < 1.3$, $1.3 \leq H_s/H_{so} < 1.45$ and $\frac{H_s}{H_{so}} > 1.45$

The corresponding damage percentages and the probability of occurrence of these damage have been discussed above.

The amount of damage W is assumed to be: percentage of damage cost of construction of cover layer 2 . The factor 2 has been adopted arbitrarily in view of the fact that the placing of a limited number of blocks is more expensive. than new building. In case of collapse ($H_s/H_{so} > 1.45$) ΔW is assumed to be equal to the total initial cost of the construction.

The following table shows the relationship between the various parameters for four breakwaters with different H_{so} 's.

Table 7.1

H_{so} (m)	Initial cost total	Initial cost cover layer	$1<H_s/H_{so}<1.3$, n=4%			$1.3<H_s/H_{so}<1.45$, n=8%			$H_s/H_{so}>1.45$;collapse		
			Δp	ΔW	$\Delta p\ \Delta W$	Δp	ΔW	$\Delta p\ \Delta W$	Δp	ΔW	$\Delta p\ \Delta W$
4	13900	5280	1.01	420	430	$5.2.10^{-2}$	860	40	$3.8.10^{-2}$	13900	530
5	15220	6600	$1.6.10^{-1}$	530	80	$4.7.10^{-3}$	1060	5	$2.8.10^{-}$	15220	40
5.5	15900	7280	$6.3.10^{-2}$	580	40	$1.6.10^{-3}$	1160	-	7.10^{-4}	15900	10
6	16540	7920	$2.5.10^{-2}$	630	15	$5.2.10^{-4}$	1260	-	$1.8.10^{-4}$	16540	3

As it is not always advantageous to repair all partial damage immediately, two cases will be considered:-

- the total amount of damage when all partial damage is repaired;
- the total amount of damage when all partial damage is not repaired.

For $\frac{100}{\delta} = 30$ values of s and S are given in table 7.2

Table 7.2

H_{so} (m)	With repairing partial damage		Without repairing partial damage	
	$s = \Sigma \Delta p . \Delta W$	$S = \frac{100}{\delta} s$	s	S
4	1000	30000	530	15900
5	125	3750	40	1200
5.5	50	1500	10	300
6	18	540	3	90

The total cost of the structure Fl = I + S

The total cost of the structure for various values of H_{so} is given in table 7.3 and plotted in the following graph.

Table 7.3

H_{so} (m)	With repairing partial damage			Without repairing partial damage	
	I	S	Fl	S	Fl
4	13900	30000	43900	15900	29800
5	15220	3750	18970	1200	16420
5.5	15900	1500	17400	300	16200
6	16540	540	17080	90	16630
6.5	17200	100	17300	20	17220

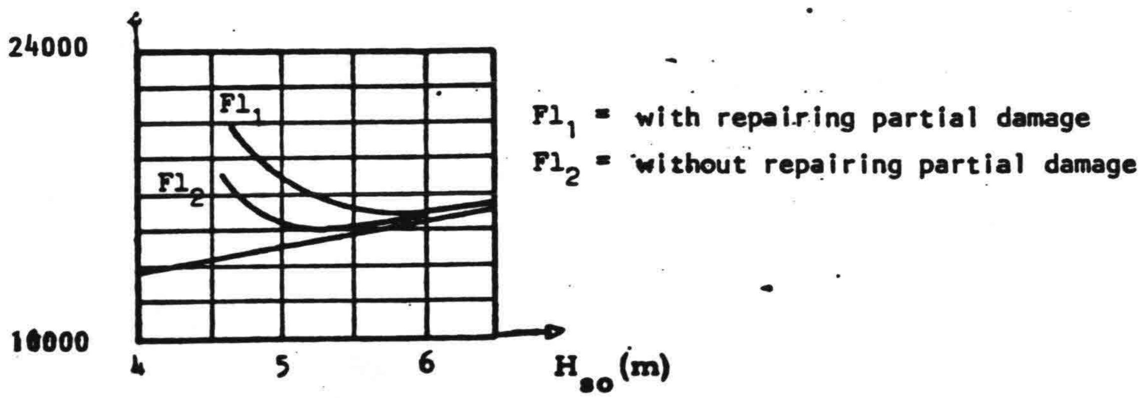

Fig. 7.5 Cost as a function of H_{so}.

7.4 Conclusion

According to the minimum cost criterion, the optimal choice of design wave is:

H_{so} = 6 m. if partial damage is repaired

H_{so} = 5.5 m. if partial damage is not repaired

This leads to block weights of 12 and 9 tons respectively.

It is also clear, however, that the coice of H_{so} = 7 m is less catastrophic than the choice of 5 m., both in case partial damage is repaired. In view of the uncertainties in the absolute quantitative value of some assumptions it is therefore advisable to tend towards the selection of a design wave height which is slightly on the high side.

PART THREE

Vertical Wall Breakwaters

8. HYDRAULIC ASPECTS

8.1 Standing wave or clapotis

When a wave is progressing to an infinitely high, vertical wall, its energy will be reflected for 100% when no energy dissipation takes place. This means that the wave will be reflected and that it will travel in the opposite direction. The height of the reflected wave can be equal to or smaller than the incident wave height, depending on the energy dissipation at the wall.
Mathematically, this can be expressed as follows (see also Fig. 8.1 and 8.2).
All values referring to the incident wave have a subscript i, and those of the reflected wave have a subscript r.

Height of the incident wave : H_i
Height of the reflected wave : H_r
Reflection coefficent : $r = \frac{H_r}{H_i}$

$$\eta_i = \frac{H_i}{2} \cdot \cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \qquad (1)$$

$$\eta_r = \frac{H_r}{2} \cdot \cos\left(\frac{2\pi x}{L} + \frac{2\pi t}{T}\right) = r \cdot \frac{H_i}{2} \cdot \cos\left(\frac{2\pi x}{L} + \frac{2\pi t}{T}\right) \qquad (2)$$

$$\eta_{tot} = \eta_i + \eta_r$$

$$\boxed{\eta_{tot} = (1+r)\,\frac{H_i}{2} \cdot \cos\frac{2\pi x}{L} \cdot \cos\frac{2\pi t}{T} + (1-r)\,\frac{H_i}{2}\,\sin\frac{2\pi x}{L} \cdot \sin\frac{2\pi t}{T}} \qquad (1)+(2)$$

Special points occur when:

$$x = o \pm n.\frac{L}{2} \quad \leftrightarrow \quad \eta_{tot} = (1+r)\,\frac{H_i}{2} \cdot \cos\frac{2\pi t}{T} \qquad \text{(antinodes)}$$

and

$$x = 1/4\,L \pm n.\frac{L}{2} \quad \leftrightarrow \quad \eta_{tot} = (1-r)\,\frac{H_i}{2}\,\cos\frac{2\pi t}{T} \qquad \text{(nodes)}$$

It can easily be seen that the amplitude in the anti-nodes is maximum and the amplitude in the nodes is minimum.

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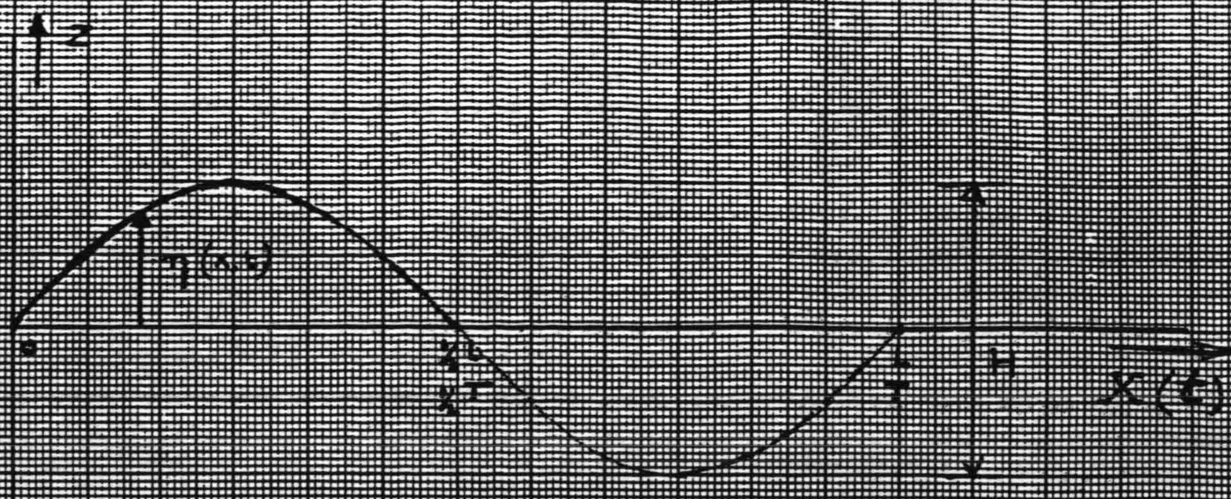


Fig. 8.1

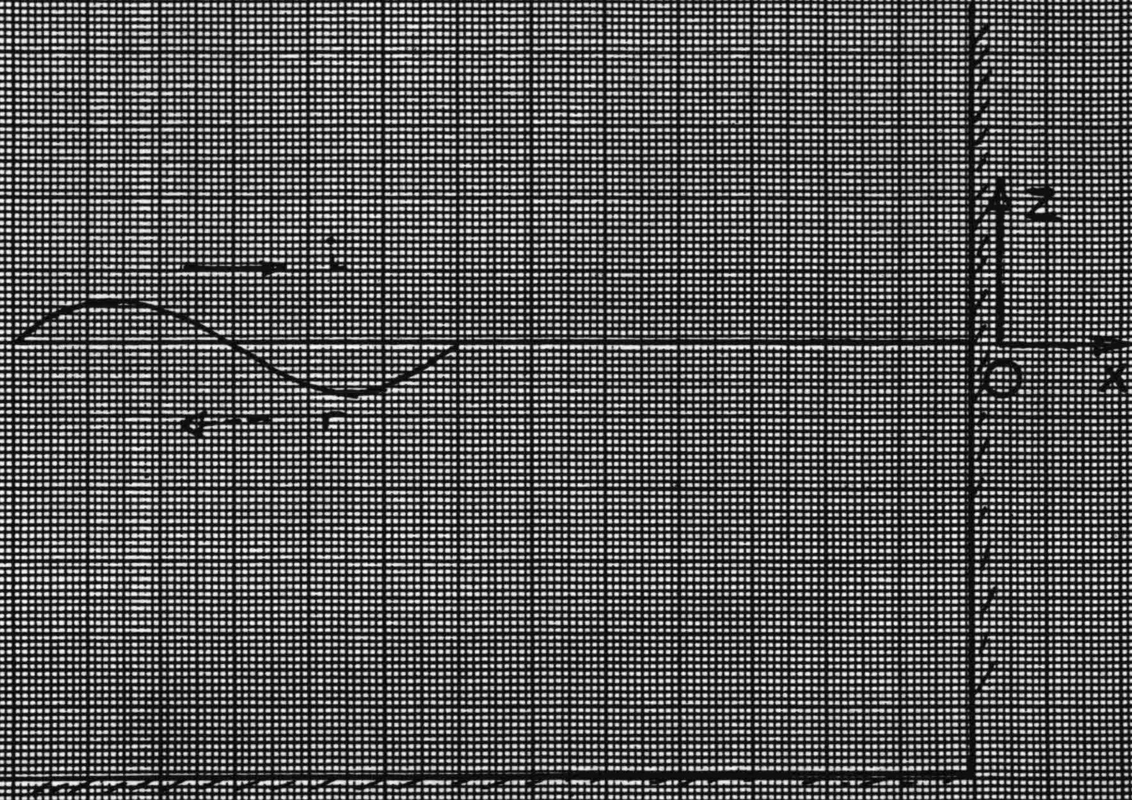


Fig. 8.2

For complete reflection ($r=1$), these values change into:

η_{node} = o , irrespective of t

$\eta_{antinode} = H. \cos \frac{2\pi t}{T}$ i.e. double the value of η_i

Another interisting exercise is substitution

of $t = 1/4\, T \pm n.T/_2$

Then, $\cos \frac{2\pi t}{T} = o$ and $\sin \frac{2\pi t}{T} = \pm 1$, irrespective of x

If at the same time $r = 1$, it means that the sea level is completely horizontal. (Fig. 8.3.)

The wave pattern thus created in front of a vertical wall is called standing wave or clapotis.

In practice, it is not possible to discern η_i and η_r. Instead hereof, H_{node} and $H_{antinode}$ can be measured.

A simple mathematical deduction learns then that:

$$H_i = \frac{H_{antinode} + H_{node}}{2}$$

$$H_r = \frac{H_{antinode} - H_{node}}{2}$$

$$r = \frac{H_{antinode} - H_{node}}{H_{antinode} + H_{node}}$$

In a similar way, horizontal and vertical components of the orbital velocity can be calculated.
It appears that horizontal velocities are doubled in the nodes and nullified in the antinodes. Vertical velocities are minimum in the nodes. Theoretically, water particles remain in their original section, moving up and down in the same block, which extends over a half wave length.
The complete image is given in Fig. 8.3

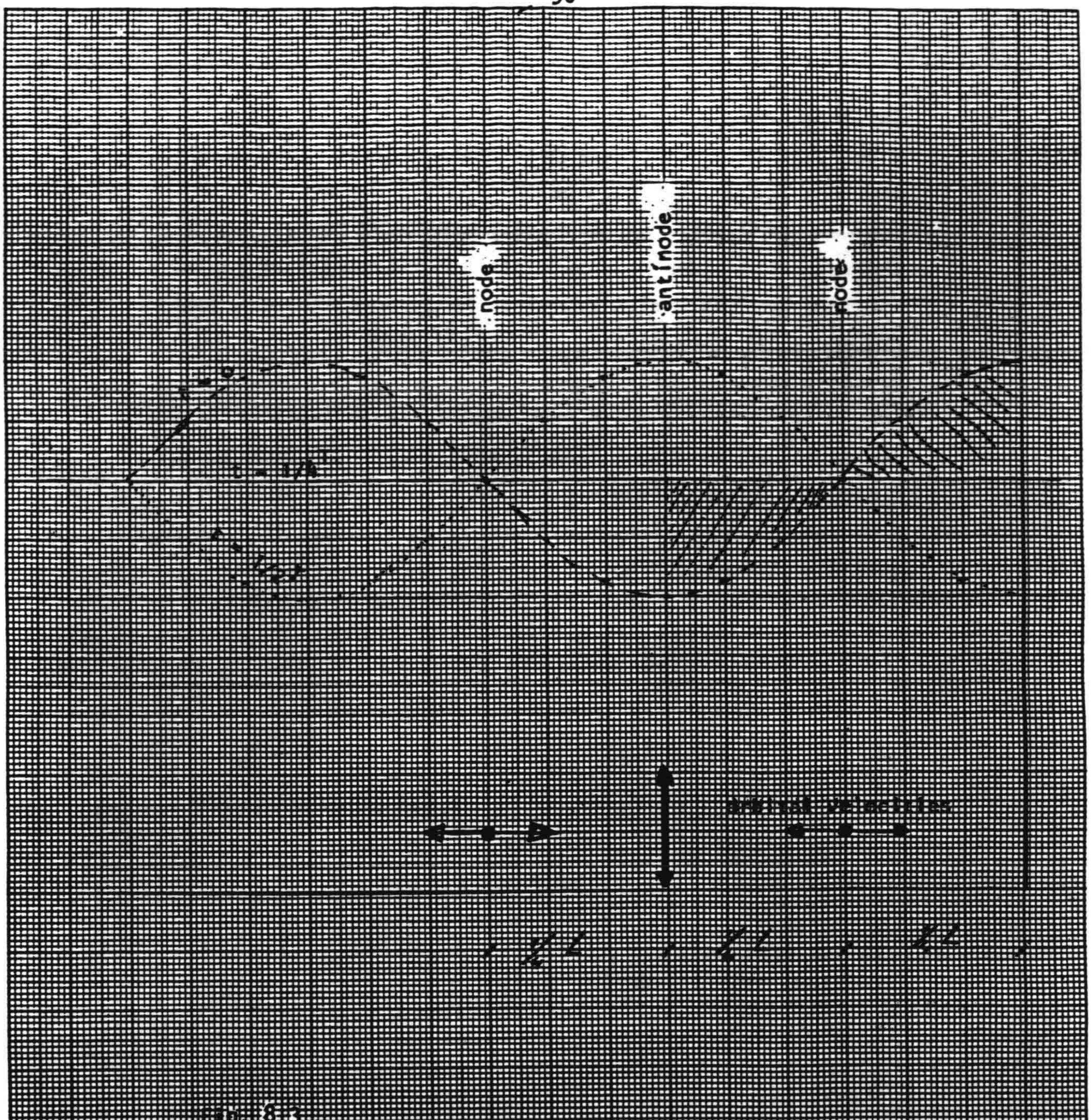
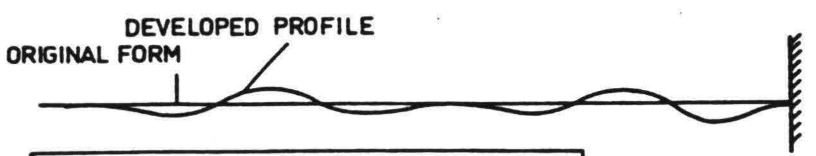


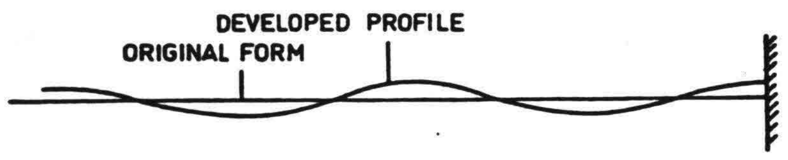
Fig. 8.3

8.2 Sediment Transport

Under the influence of the horizontal orbital velocities over the seabed, sand can be transported by waves.
Sediment transport due to the standing wave may also occur. The current pattern in a pure standing wave will of course not lead to progressive sand transport. Nevertheless, sand transport may take place within the various cells of the standing wave.
The scour pattern depends very much on the diameter of the sediment, and more specifically on the question whether the sediment will come into suspension. Model investigations have shown two distinctly different patterns for both cases. (see Fig. 8.4a and 8.4b)



a. BOTTOM PROFILE FOR COARSE SAND



b.. BOTTOM PROFILE FOR FINE SAND

Fig. 8.4a and 8.4b.

In case there is a current superimposed on the wave action, sediment can be merely whirled up by the waves and finally transported by the current. It is even possible, that neither the wave nor the current individually exceeds the threshold velocity and that their combined action leads to considerable sediment transport.
This situation can certainly develop in the region of the node of a standing wave since the horizontal orbital velocities are double the value of those caused by a progressive wave. Thus, the region at 1/4 wavelength distance from a vertical wall is susceptible to scouring.
Because of the uncertainty of the scour pattern to be expected, it is advisable to apply a scour protection in front of a vertical wall breakwater. The width of such scour protection should be sufficient to guarantee the soil mechanical stability of the structure, even if a steep and deep scour hole develops at the end of the protection.
Watching the development of the scour hole and applying a bottom protection in a later stage may lead to disappointments since the development of a scour hole may take place in a time too short to take adequate countermeasures.

8.3 Quasi Static Forces

As long as the conditions are such that no breaking waves occur, a standing wave will develop in front of a vertical wall breakwater.
The pressure distribution on this wall is a superposition of the hydrostatic pressure and the wave induced pressure.
The wave induced pressure can be calculated following the first order theory.

This leads to the follwing result:

$$p\ (z) = -\rho g z + \rho g \theta \ . \ \frac{\cosh \frac{2\pi}{L}(z+D)}{\cosh \frac{2\pi D}{L}} \quad (\eta_i + \eta_r)$$

or, for $r = 1$

$$p\ (z) = -\rho g z + \rho g H \ \frac{\cosh \frac{2\pi}{L}(z+D)}{\cosh \frac{2\pi D}{L}} \ . \ \cos\omega t$$

In wich:

p = pressure in N/m2
ρ = density in kg/m3
g = acceleration of gravety in m/sec^2
z = vertical coordinate (positive from water surface)
D = water depth in m.
ω = angular frequency $\frac{2\pi}{T}$ in sec^{-1}
L = wave length in m.
t = time in sec.
H = wave height of incident wave in m.

Basically, Sainflou [32] followed this theory in the method he published in 1929. The only difference with the linear theory is the fact that Sainflou takes into account a virtual rise of the mean sea level due to asymmetry of wave crest and wave trough.

The same effect can be introduced in a theoretically more sound way by applying a higher order theory. MicherRundgren and [33] discribed a second order theory.
Although modern calculatory aids have greatly facilitated the application of even higher order theories, it is not advisable to apply these, since their use will not lead to more reliable results.
In some cases, the crest of the standing wave will be higher than the breakwater. It is evident that the uppermost part of the pressure diagram should be omitted then. (See Fig. 8.5)

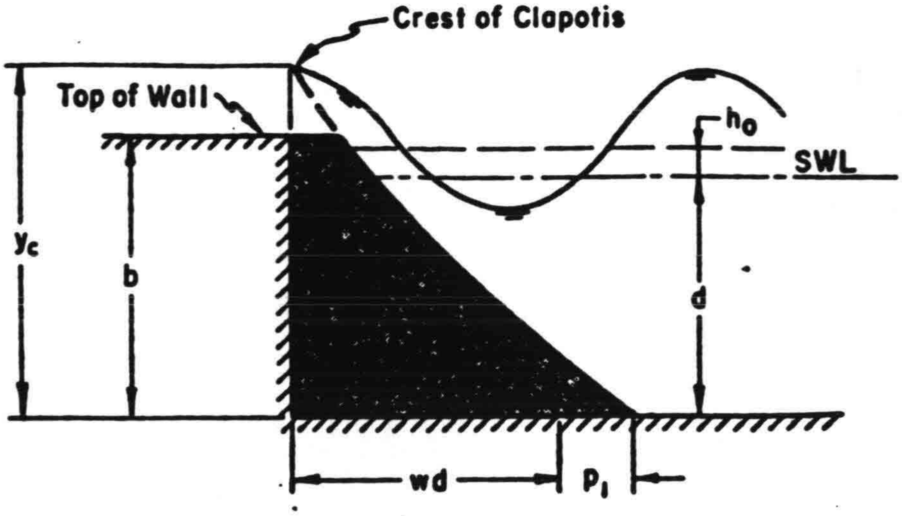


Fig. 8.5 Wall of Low Height - Pressure Distribution.

When considering the rear wall of the breakwater, it is clear that here the hydrostatic pressure is also present. The presence of wave pressures depends on the local circumstances. Attention should be paid to a possible systematic phase difference between the wave pressure on the front and rear wall respectively.

The wave pressure occuring at the toe of the front wall also penetrates underneath the breakwater and creates an additional uplift.

8.4 Dynamic Forces

Waves breaking against a structure can cause extremely high pressures. The duration, however, is short and the area of application relatively small.
The pressures are caused apparently by the deceleration of the mass of water hitting the structure. Such a decelerating mass of water is generally found in the crest of breaking waves, and impacts thus occur usually near the still water level. (See example in Fig. 8.6)

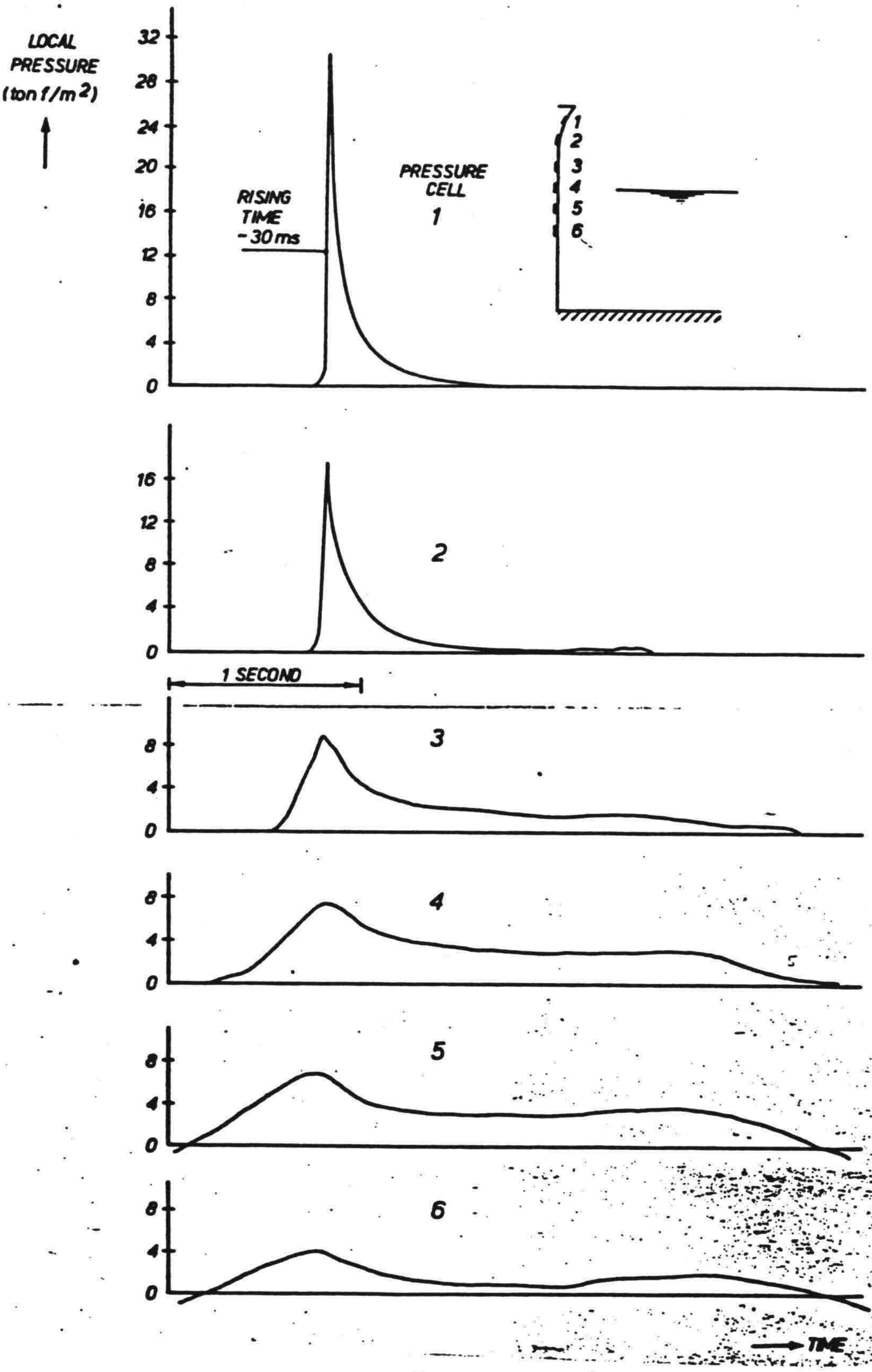


Fig. 8.6 Distribution of wave impact forces.

Two models can be used to describe and calculate the physical phenomena:-

- the continuous water jet yielding a pressure
  $p = 1/2 \; \rho \; v^2$ (v = the water velocity in the jet)
- The water hammer, resulting in
  $p = \rho vc$

  in which:

  v = the water velocity

  c = the velocity of sound in water (1543 m/sec)

  the duration of the shock in this case is

  $\Delta t = \frac{L}{c}$, if L is the length of the block of water.

The water velocity in the crest of a breaking wave is equal to the wave celerity (in shallow water $\sqrt{gd}$).
Substitution of a reasonable water velocity of v = 10 m/sec. leads to the following maximum impact pressures:

continuous jet $p = 1/2 \; . \; 1030.10^2 = 5.5 \times 10^4$ N/m2

water hammer $p = 1030. \; 1593.10 = 1589 \times 10^4$ N/m2

Experiments, carried out by Bagnold as early as 1939 and many others thereafter have shown that in reality, wave impact pressures may attain values of 50 to 150 N/m2. [34]

Both, measurement and calculation of wave impact forces are extremely difficult. Measurement, because of the short duration of the impact (10 to 100 msec.) and calculation because of the unknown effect of airbubbles in the wave crest. (This greatly influences the propagation velocity of sound in water, and creates a sort of shock absorber).

A simple calculation method for wave impact forces was developed by Minikin [35] and is described in [2]

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Minikin method:

The maximum pressure assumed to act at S.W.L. is given by:

$$p_m = 101.\rho g. \frac{H_b}{L} \quad \frac{d}{D} \quad (D+d)$$

$$R_m = \frac{p_m. \; H_b}{3}$$

in which:

p_m	=	maximum impact pressure	in N/m2
ρ	=	density	in kg/m3
g	=	acceleration of gravity	in m/sec2
H_b	=	breaking wave height	in m.
d	=	water depth at the toe of the structure	in m.
D	=	waterdepth at one wave length in front of the wall	in m.
L	=	wave length in a water depth D	
R_m	=	maximum resulting force Note: this force acts over a limited length of the structure only!	in N/m.

See also definition sketch (Fig. 8.7)

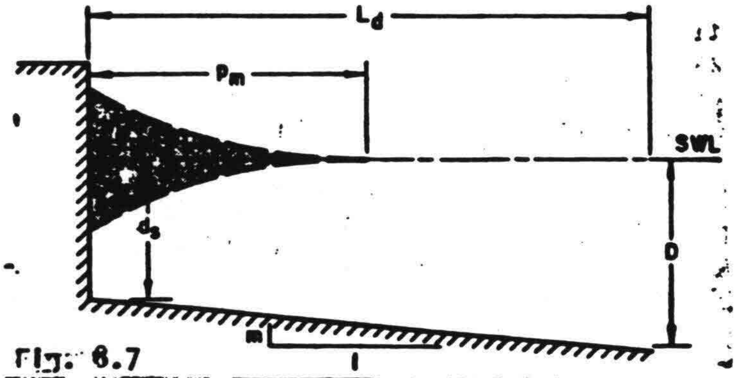

Fig. 8.7

The results of the above calculation cannot be considered to be more than a first assessment of the magnitude of the impact forces. When a monolithic breakwater will be subject to breaking wave forces, extensive model investigations will be required. In such further studies attention must be paid in particular to those elements of the structure where water can be entrapped. (Fig. 8.8 and 8.9) These locations have appeared to be extremely vulnerable.

After the magnitude, duration and area of application of the wave impacts have been determined, the dynamic response of the structure as a whole, separate elements and the foundation have to be taken into account.

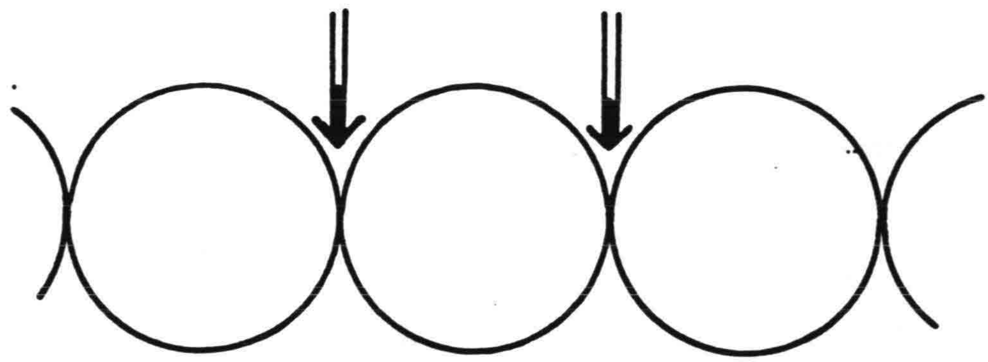

Fig. 8.8 Concentration of impact forces - cellular structure.

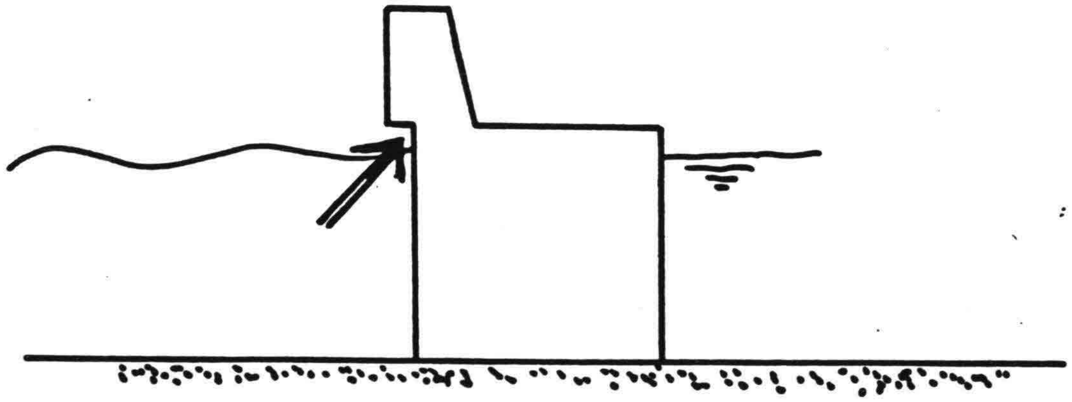

Fig. 8.9 Concentration of impact forces.

8.5 Overtopping

Overtopping of vertical front breakwaters by solid masses of water can easily be assessed on the basis of the standing wave formulae.

In case of breaking waves, large quantities of water will escape in upward direction and be blown over the breakwater by the wind. Neither the quantity of water nor the forces exerted by these masses can be neglected.

9. STRUCTURAL RESPONSE

The pressure fluctuations caused by a standing wave can usually be treated as static loads on the structure. This is the reason that these forces are often referred to as quasi static forces. In the case of wave impact forces, however, the duration of the external force is that short and close to the natural period of vibration of the structure, that inertia effects caused by the movement of the breakwater must be included.

It must be recognized that a monolithic breakwater acts as a mass, supported by a compressible soil (spring) and surrounded by an energy absorbing medium (damping).

9.1 Mass spring system

In its most elementary form, the mass spring system is represented by the sketch of Fig. 9.1

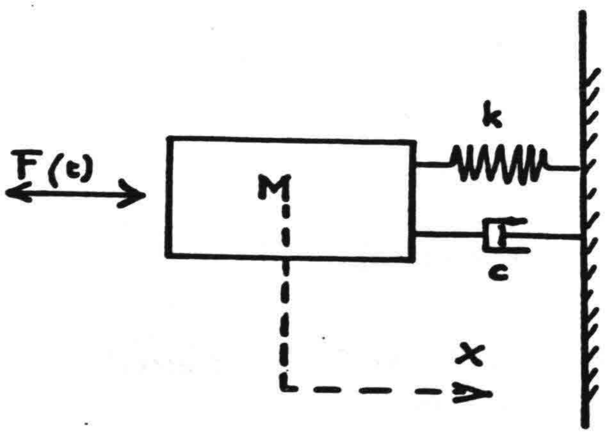

Fig. 9.1 Linear Mass-spring System.

When a time-dependent external force F (t), is applied, the equation of motion is:

$$F(t) = M.\ddot{x} + c.\dot{x} + kx$$

in which

M = the mass

x = the displacement of the mass from its neutral position

c = the damping

k = the spring constant

The above differential ~~calculation~~ equation can be solved for a harmonic external force:

$$F(t) = f \sin \omega t$$

The solution is:

$$x = \frac{f}{k} \cdot \frac{1}{\sqrt{\left(1-\frac{\omega^2}{\omega_o^2}\right)^2 + \frac{4.\omega^2 c^2}{\omega_o^2 . c_o^2}}} \cdot \sin(\omega t - \cancel{\phi}\varphi)$$

ω = angular frequency of the external force

φ $\cancel{\phi}$ = phase angle

$\omega_o = \sqrt{\frac{k}{M}}$ (resonance frequency)

$c_o = 2\sqrt{k.M.}$ (critical damping)

$\frac{f}{k}$ is often referred to as the "static" displacement, (x_{static}) i.e. the displacement for infinitely small ω.

The structural response is represented in Fig. 9.2

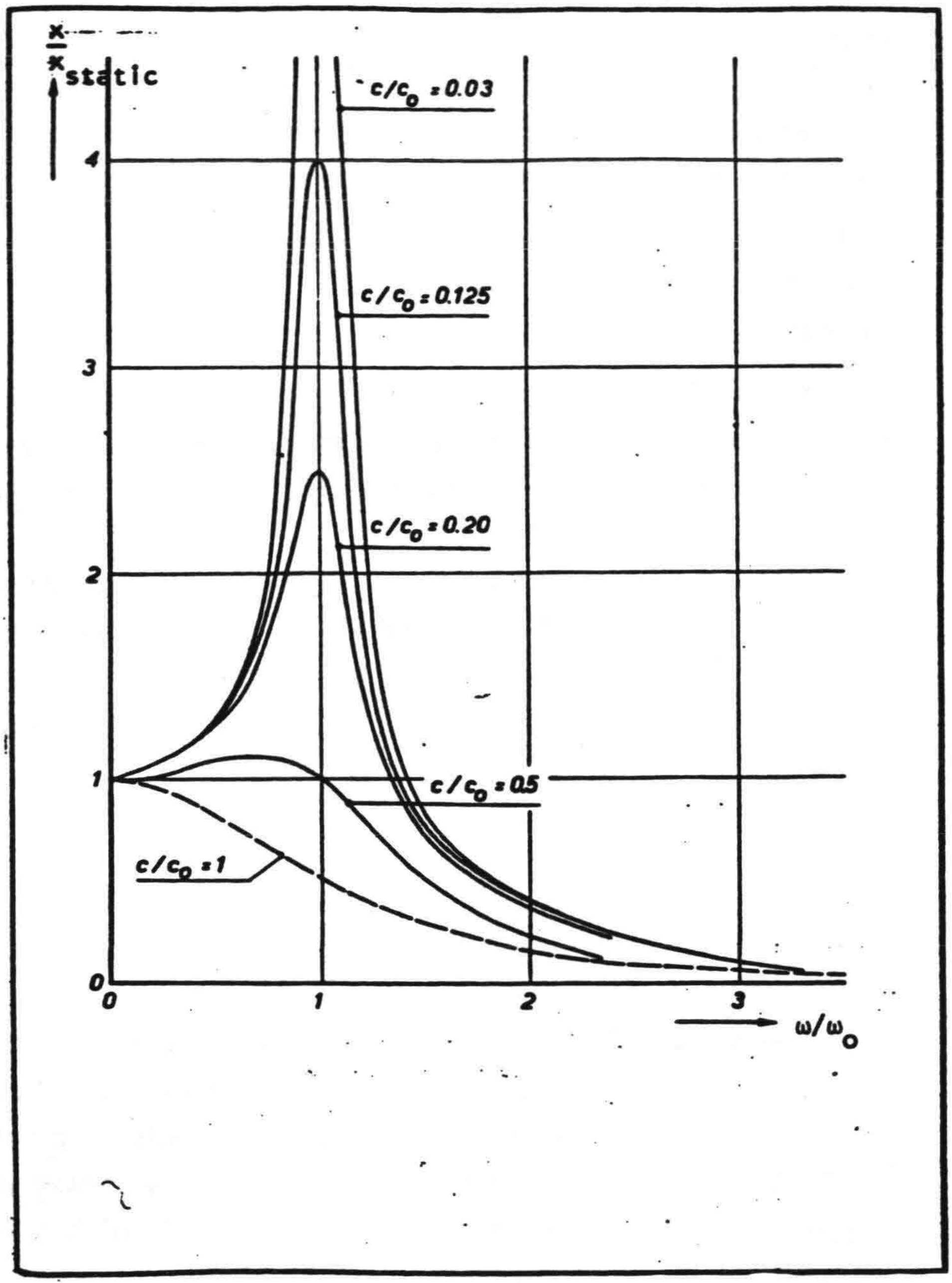

Fig. 9.2 Response curves for linear mass-spring system.

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From the above, it can be concluded that a flexibly supported structure has a resonance frequency ω_o , determined by the mass and the spring constant only. If this structure is loaded by a harmonic force with a frequency $\omega = \omega_o$, the displacements will be infinitely high in the complete absence of damping. In case energy is dissipated in the system there is damping and consequently the displacements will be less.

It is evident that a monolithic breakwater reacts more complicated that the elementary system as described above. The most essential differences are:

1) The mass to be taken into account is not only the mass of the breakwater, also water and soil are moving along with the breakwater, and an addition has to be made for their respective masses.
2) The spring is not only created by compression of the subsoil, also the bending of beams and walls is to be taken into account.
3) Finally, for an assessment of the damping, the generation of surface waves and compression waves into the ground should be taken into account.

9.2 Model experiments

When carrying out model experiments on wave impact forces, it should be kept in mind that the model also reacts as mass-spring system. To measure the external force accurately, the dynamic behaviour of the model should be eliminated. This is possible by using an extremely rigid model and electronic equipment (pressure gauges, amplifiers, recorders) with a relatively high resonance frequency.

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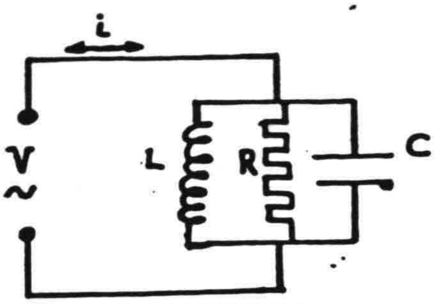
After assessing the external forces, the structural behaviour can be calculated with the aid of a computer, simulating the elastic behaviour of structure and subsoil, using the measured forces from the model as input signal.
A more simple way is the use of an analogue technique, based on the electric equivalent of the mass spring system. (See also Fig. 9.3)



$$\hat{i} = \hat{v}\sqrt{\frac{1}{R^2} + \left(\omega C - \frac{1}{\omega L}\right)^2} \quad \text{and} \quad \omega_0 = 1/\sqrt{LC}$$

Fig. 9.3 Electrical Analogon of mass-spring system.

Another but much more complicated method is to construct a model with the same dynamic characteristics as the prototype. (Elastic similitude). This technique eliminates the necessity of structural response calculations, since the structural response is directly measured (stresses, displacements, etc.)

9.3 Subsoil Response

The oscillatory loads exerted by waves on the structure are transferred to the foundation. The soil is alternately loaded and relieved. The consequential volumetric changes are not caused by compression of either grains or groundwater, but by a re-arrangement of the grains with respect to each other.
The pore volume increases or decreases or in other words: the packing changes.
Because the foundation soil underneath a breakwater is generally saturated with water, any change in the pore volume must cause in - or outflow of water.

In case of a low permeability (fine sand or clay) the flow of water due to a change of the external load takes time. Consequently the pressure of the groundwater rises.
This means at the same time that the contact pressures between the grains decrease:

$$\sigma_{total} = \sigma_{grain} + \sigma_{water}$$

The decrease of σ_{grain} leads to lower shear strength, since $\bar{\tau}$ is a function of σ_{grain}.

In case of a static load, the water over.pressure gradually decreases, and the shear strength is restored.
In case of an oscillatory load, the water over-pressure is also fluctuating harmonically. In case of a high compresibility and a low permeability (fine, loosely packed sand) the fluctuations of the water pressure can take place at a higher level after every oscillation. The water pressure σ_{water} may become that high that σ_{grain} and consequently the shear strength $\bar{\tau}$ are nullified.
This leads to liquefaction and loss of stability. (Fig. 9.4)

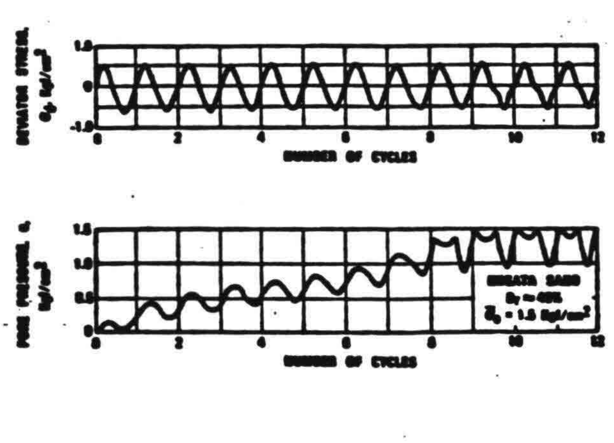


Fig. 9.4 Typical Pore Pressure Response.

The processes described above are the main threat to monolithic breakwaters.

The excess pressures of the groundwater, however, give also rise to other problems. A sketch of the calculated excess pore pressures under the bottom corner of a monolithic breakwater is given in Fig. 9.5

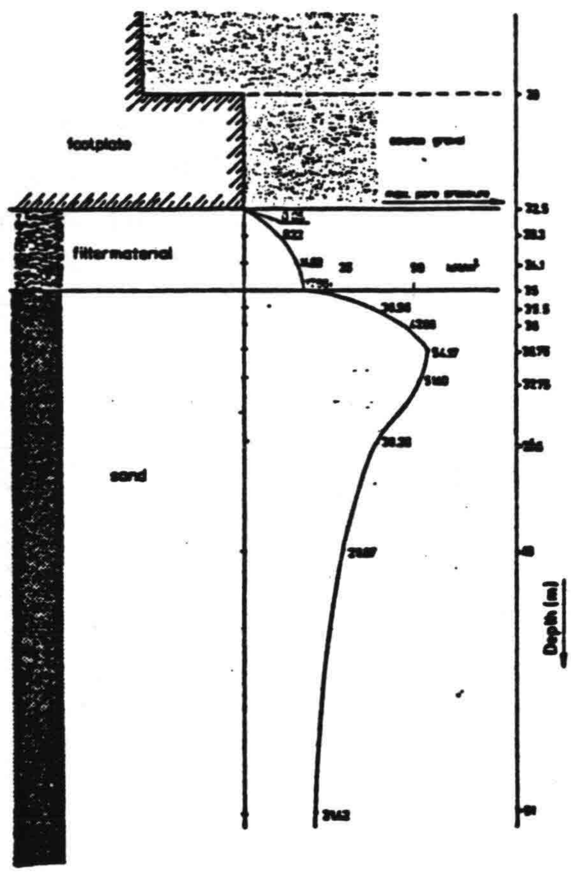


Fig. 9.5 Cyclic pore pressure amplitude.

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It appears that the (vertical) pressure gradient is very large, especially in the vicinity of the seabed.
As a result of this, individual grains may be lifted up because the upward pressure gradient exceeds the gravety force. Stability of the seabed can only be ensured by high quality.
The pressure gradients near the interface between subsequent layers of the filter are extremely high,(>100%), and exceed the usually encountered values.
Research on the resistance of filters towards gradients parallel and perpendicular to interfaces is being carried out in several soil mechanical and hydraulic laboratories.
The problem is closely connected with piping. Preliminary results of filter tests have already been shown in chapter 9.
Partial results have been published in [30] and [31]
Pending availability of final results of filter tests, it is recommended to apply conservative values as published by Terzaghi:

For uniform material:

$$\frac{d_{50}(\text{filter})}{d_{50}(\text{base})} < 5 \text{ to } 10$$

For graded material:

$$\frac{d_{50}(\text{filter})}{d_{50}(\text{base})} < 12 \text{ to } 58$$

and

$$\frac{d_{15}(\text{filter})}{d_{85}(\text{base})} < 5$$

10. DESIGN ASPECTS

10.1 Structural Design

Indication of structural calculation methods for monolithic breakwaters is beyond the scope of these lectures.
It is evident that concrete technology plays a major role. Especially avoiding of crack formation in the concrete is important to prevent corrosion of reinforcing and or tensioning steel. Recent litterature on this subject is available in connection with the construction of off-shore gravity platforms in the North Sea.

10.2 Conceptual Design

The quasi static and dynamic forces exerted by the waves on the front of a monolithic breakwater act perpendicular to the plane of application, i.e., their direction is horizontal.
As a result of this force, the structure may translate or rotate (slide or overturn). Both movements are counteracted by the weight of the structure, the friction over the bottom and the ground pressures. (Fig. 10.1)

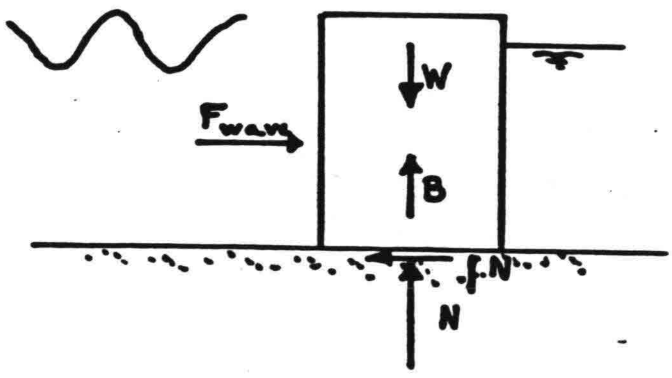

Fig. 10.1 Equilibrium of forces. vertical front.

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The stability can certainly be increased if the direction of the wave force is changed. This can be achieved by designing a sloping front wall. (Fig. 10.2 and 10.3).
Especially a sloping front in the vicinity of the still water level (Fig. 10.3) is effective in reducing the effect of wave impact forces on the over all stability.
A completely different concept was introduced by Jarlan [36] , who designed a caisson type of breakwater with a perfoated front wall (Fig. 10.4). As a result of this perforation: -

- the impact forces are reduced;
- the reflection coefficient is reduced;
- the quasi-static forces are reduced.

The principle of a perforated front wall and a stilling chamber behind it is applied in a great number of concrete off-shore structures in the North Sea.
The principle is patented.

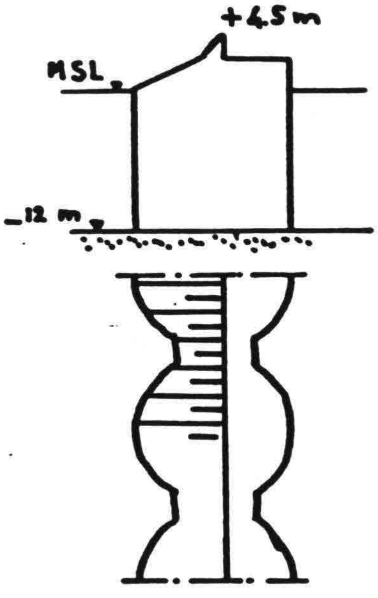

Fig. 10.2 Hanstholm breakwater with sloping front 1:3

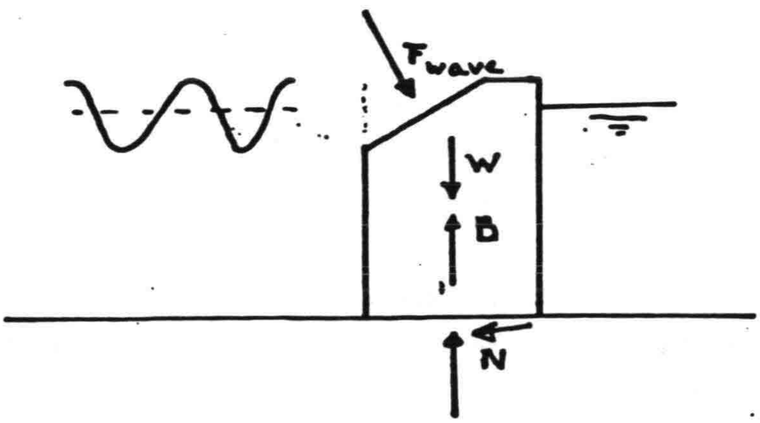

Fig. 10.3 Equilibrium of forces, sloping front.

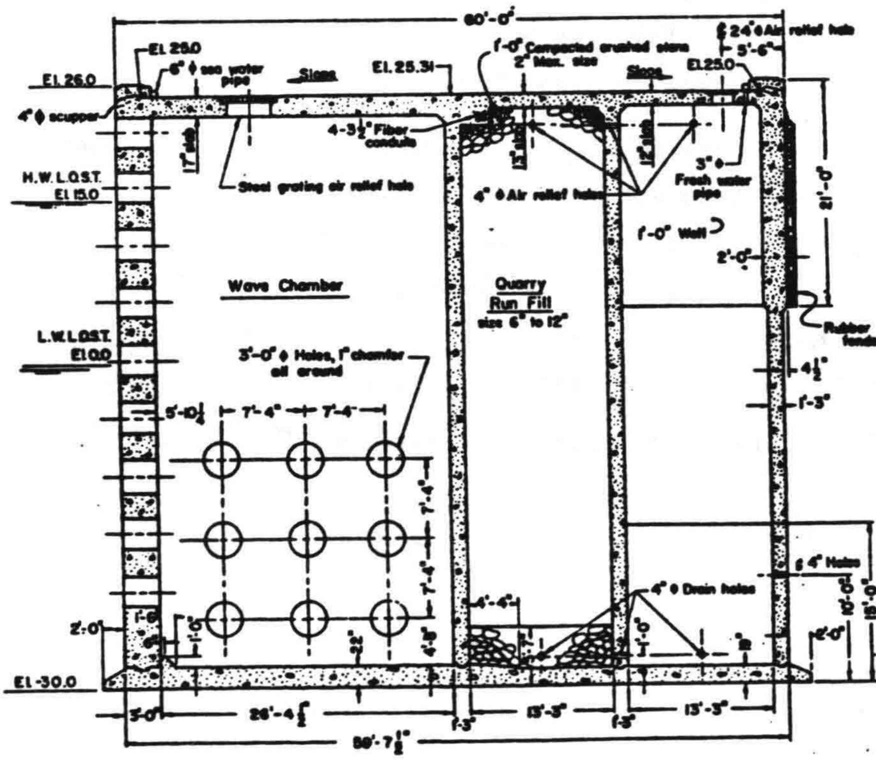

Fig. 10.4 Perforated Caisson Breakwater.

10.3 Design of cross sections

Despite the complexity of calculating the stability and the structural strength of vertical wall breakwaters, the international commission for the study of waves formed by PIANC has established a number of design recommendations. [37]

In doing so, two limit states are defined: -

- a limit state for use of the breakwater characterized by a wave height H_u with a reasonable return period;
- a limit state for rupture of the breakwater, characterized by a wave height H_r which is an extreme wave height.

For establishing a preliminary design of a vertical breakwater, it is possible to design the cross section of the structure, as a first approcimation, as follows: -

α

- wall presenting a free height of at least 1.5 H_r below low water (according to the XVIIIth International Navigation Congress' recommendations, Rome, 1953), a thickness at least equal to 0.80 times the free height and a toe protection against undermining, whose thickness would be at least equal to 0.15 times the free height (that puts the depth of the wall foundation at least 1.5 x 1.15 = 1.72 H_r below low water);
- crest mass rising up to the elevation 1.3 to 1.5 times H_u on the open sea side and up to 0.5 H_u on the harbour side (with a trapezoidal guard-wall or a graded wall, whose thickness averages about 0.75 H_u);
- riprap substructure base with a width of about 2.5 H_u at the level of the wall foundation.

This leads to a cross section as given in Fig. 10.5

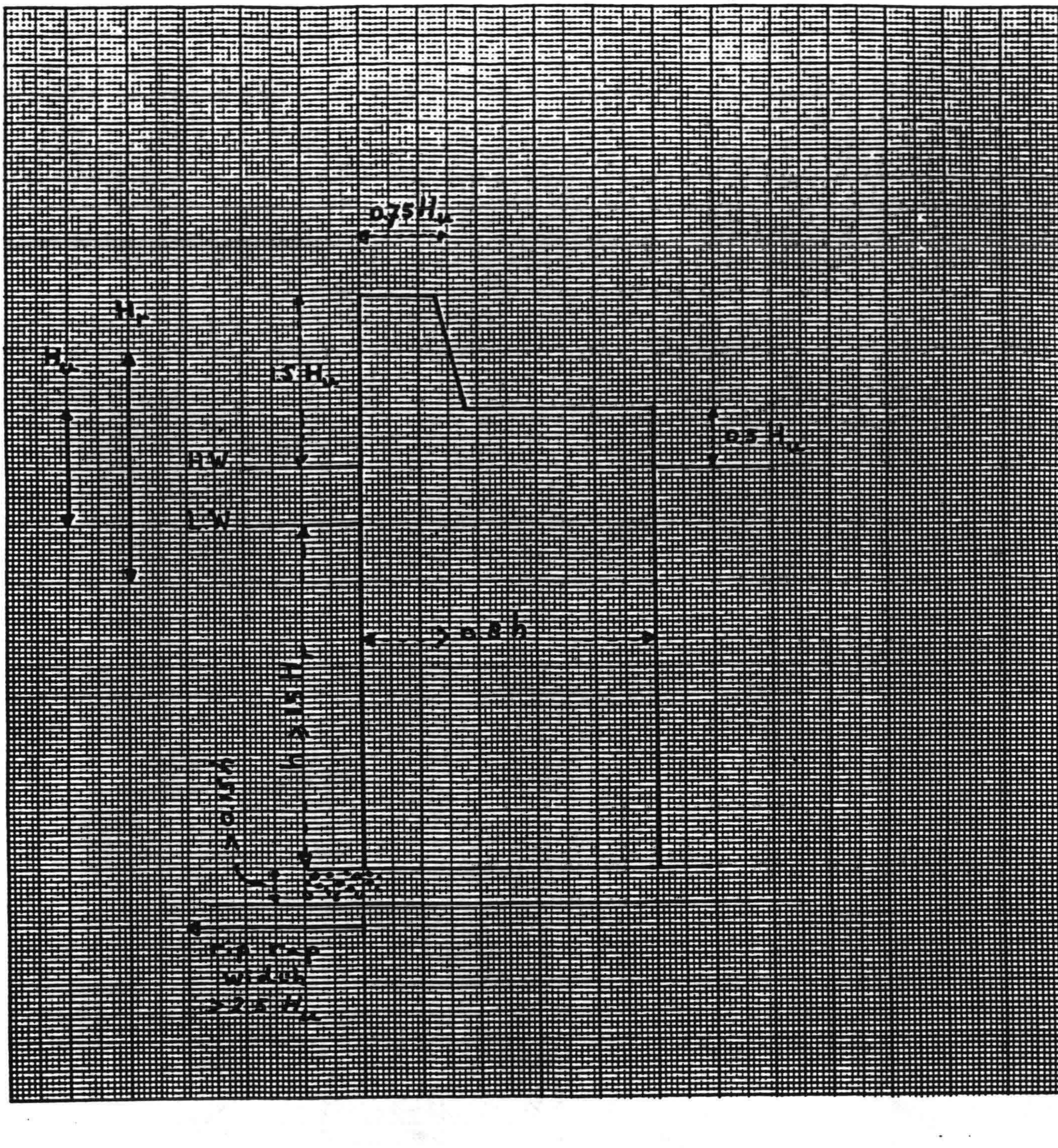

Fig. 10.5 Definition Sketch.

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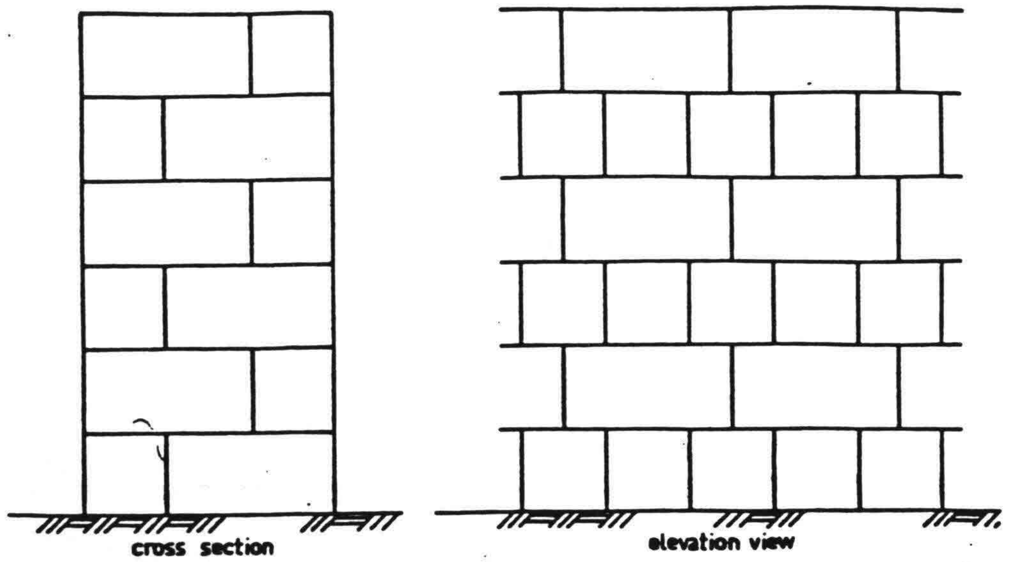

Fig. 11.1 Typical Monolithic Breakwater (scale 1:150)

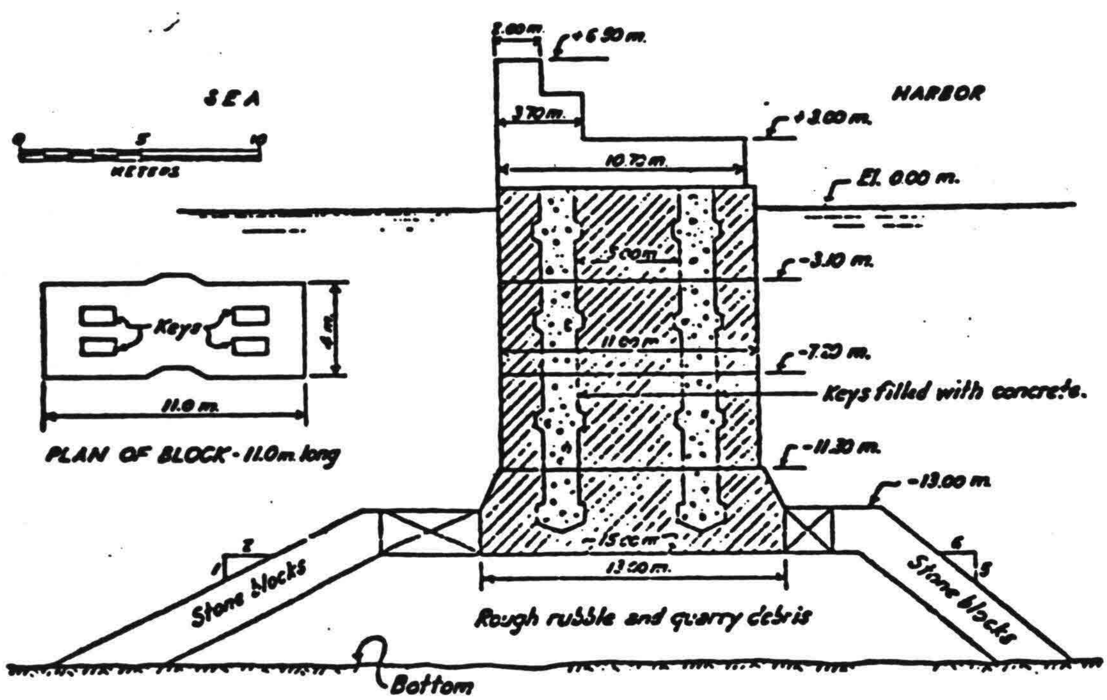

Fig. 11.2 Breakwater from Algiers, Morocco.

11.3 Construction of large units in situ

The most common example of in situ construction of large monolithic units is the construction of sheet pile cells. The main problem of this type of structure is the closure of the slots between the planks, all around the cell. Also workability during driving of the sheet piles may cause problems. When impact forces occur, the fill material inside the cells may be fluidized, which causes extremely high ground pressures on the sheet piling. The effects of spray are always underestimated in model experiments by the influence of surface tension. This means that for stability considerance the cells must be assumed to be saturated over the full heights.

11.3.1 Self floating:

Large self floating caissons are generally casted in concrete. Construction can take place on a slipway, in a dry dock or in a special purpose construction pit. To minimize the use of these expensive facilities, the structures are often floated as soon as the lower part is completed, the upper part of the walls is casted with the caissons floating along a pier or jetty.
After completion, the caissons are towed to the site of the breakwater (sufficient water depth should be available!) and by opening valves in the bottom, the caissons are sunk in position onto a foundation consisting of the proper filter material. The caisson is than ballasted with sand and the superstructure completed.

For accurate positioning of the caissons, a relatively large number of tugboats is required. (Fig. 11.3)
After the first caisson has been placed, this can be used as a fixed point during the positioning of subsequent units. The most difficult phase of sinking is encountered just before the caisson touches the bottom. It has a tendency to move sideward. (saucer effect)

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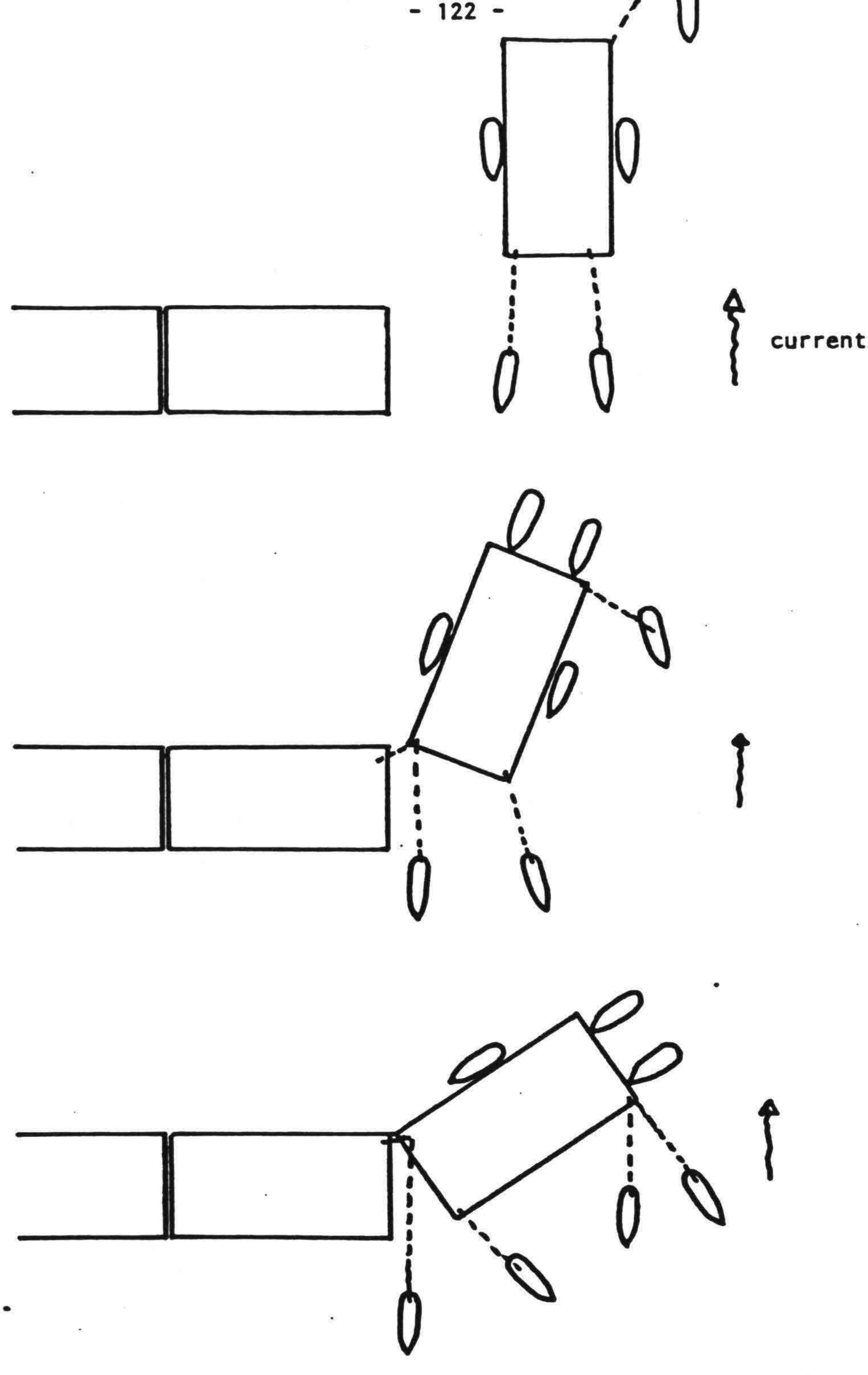

Fig. 11.3 Positioning of caisson

To avoid this effect, the sink velocity should be kept small, and also dowels can be applied, pinning the caisson in position before it approaches the seabed (Fig. 11.4)

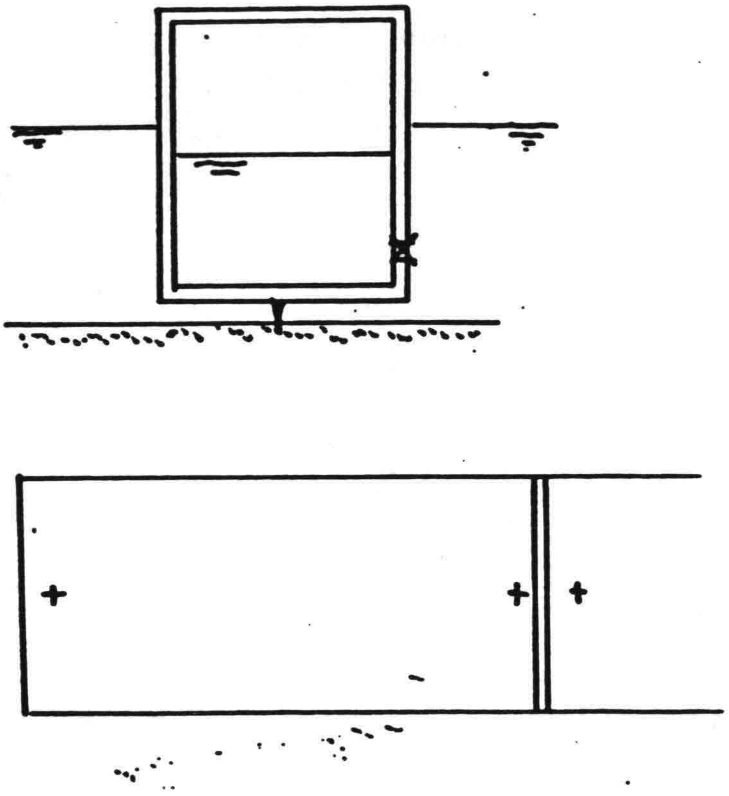

Fig. 11.4 Dowels

When the foundation at the seabed is not completely flat, the caisson will be supported by the three highest spots of the foundation. When these spots are distrubuted at random the bottom of the caisson may be supported unfavourably. This situation can lead to unwanted cracks. Therefore, the bottom of the caisson is sometimes shaped so that support is certainly concentrated in extra reinforced sections (buttocks).

Lateron, the empty space between the foundation and the bottom of the caisson can be filled by pressing grout (a sand cement mortar) into the void. In this case, a steel skirt is applied, which penetrates into the seabed and closes of the circumference of the caisson. (Fig. 11.5)

The skirt can also eliminate or reduce the saucer effect.

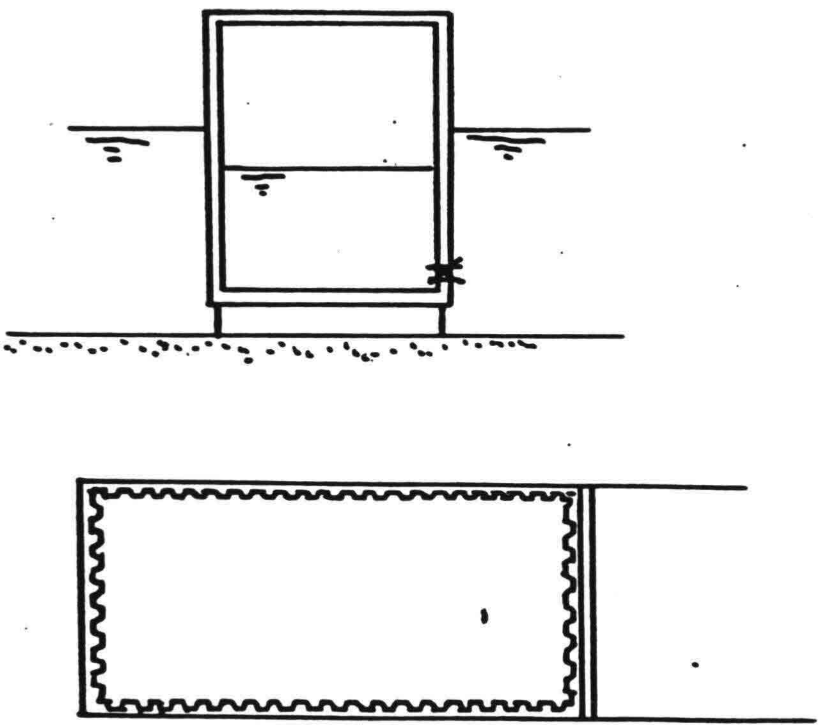

Fig. 11.5 Skirt

PART FOUR

Conclusion

11.3.2 Added flotation:

It is well possible to construct a caisson type of structure, omitting the bottom, so that the structure is not self floating. Floating transport can be realized in this case by artificial floats, connected to the structure.
Proper attention is to be paid to the stability and the dynamic behaviour in waves.
The structure is ballasted with quarry stone or with a low grade concrete. Sand cannot be used since this may be washed away through the bottom voids.

11.3.3 Dry transport:

Transport and placing of large prefab units can also be done over the breakwater crest. (Fig. 11.6). In this case, cylindrical caissons without a bottom are trensported and placed with the aid of a huge crane bridge.
The bottom is casted in situ (underwater concrete) and the units can thus be ballasted with sand.

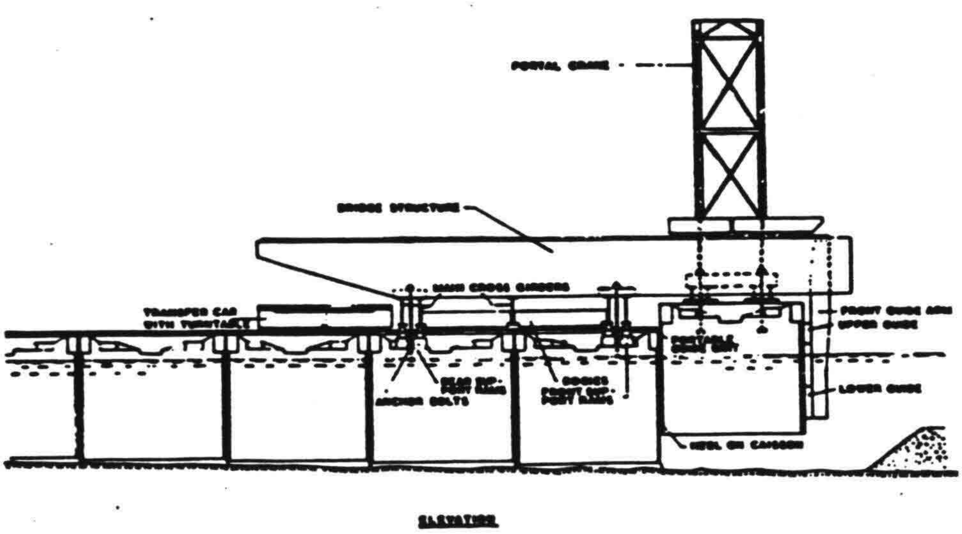

Fig. 11.6

12. CONCLUDING REMARKS

12.1 Rubble mound or monolithic

Rubble mound breakwaters are relatively simple structures. The design procedure is not complicated. The structure carries a large safety margin between incipient damage and final failure.
The main disadvantage is the rapidly increasing volume of material required when the breakwater is constructed in deep water.

Monolithic breakwaters are generally economic as far as the quantity of material in the cross section is concerned. Disadvantages of the monolithic breakwater are the very time consuming, labour intensive and complicated design procedure. The procedure involves an uncertain interpretation of model results as far as impact forces are concerned. Overloading leads to a sudden and complete failure. Foundation problems are serious and difficult to solve, mainly in case of a subsoil consisting of impermeable loosely packed sand and for clay.
Pending further soil mechanical research the application of monolithic breakwaters is to be discouraged in these conditions. When the subsoil consists of rock, gravel or densely packed coarse sand, the monolithic breakwater presents an acceptable solution which is certainly competitive with rubble mound breakwaters when the water depth is considerable.

12.2 Preparations

Before a breakwater can be designed, and a choice of design can be made, extensive site investigations have to be performed, covering : -

- hydraulic and hydrographic conditions;
- soil mechanical conditions;
- availability of materials.

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12.3 Model Investigations

A sound breakwater design cannot be made without model investigations. Research is indispensable during the design process. But also the final design is to be tested extensively under conditions compatable with the prototype. Final tests shall include overloading of the structure until failure.

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LECTURE NOTES

BREAKWATER DESIGN

by

K. d'Angremond

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