COASTAL ENGINEERING

Volume III

BREAKWATER DESIGN

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COASTAL ENGINEERING
Volume III - Breakwater Design

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"He knows enough who
knows to learn".

Abraham Lincoln

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BREAKWATER DESIGN

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1. INTRODUCTION

W.W. Massie

1.1. Scope

This third volume of the series on coastal engineering concentrates on a single specialized topic: breakwater design. The subdivisions into four categories found in the previous two volumes is not found here; all of this volume relates to harbors in some way. Of course, some information presented here can be used elsewhere. For example, knowledge of wave impact forces, important for the design of monolithic breakwaters, can also be handy when designing offshore structures.

A more direct tie can be made between the design methods used for breakwaters and those needed for coastal defense works - volume I, chapter 30.

1.2. Contributors

The primary authors are listed at the beginning of each chapter; final editing and coordination was done by W.W. Massie, layout by W. Tilmans, J. van Overeem and J.D. Schepers. Table 1.1 lists the staff members of the Coastal Engineering Group who contributed to this volume.

1.3. References

One general reference is so handy for breakwater design that it is not repeatedly mentioned. This book is the *Shore Protection Manual* published in 1973 by the U.S. Army Coastal Engineering Research Center. Information presented well there will not be duplicated here; these notes complement rather than replace the *Shore Protection Manual*.

1.4. Miscellaneous Remarks

As in previous volumes, the spelling used is American rather than English. A list of Dutch translations of the more important technical words is available.

The notation used is kept as consistent as possible with previous volumes and with internationally accepted practice. A symbol table is included in this volume, even though most symbols are defined in each chapter as they appear.

Literature is listed in the text by author and year; a more complete listing is included separately in the book.

More general introductory material may be found in chapter 1 of volume I of these notes.

Table 1.1 Contributors to this volume

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2. GENERAL CONSIDERATIONS

W.W. Massie

2.1. Purpose

Most generally speaking, breakwaters are built to change the coast in some way.* The development of the need for breakwaters has paralleled that of harbor and approach channel development outlined in chapters 14 and 15 of volume I.

More specific purposes for breakwaters were described in chapter 18 of volume I, but shall be treated in more detail here.

The most obvious purpose of a breakwater is to provide protection against waves. The protection may be provided for an approach channel or even for a harbor itself. This type of protection is necessary in order to provide quieter water for ships to navigate and moor. Motion of moored ships in harbors can be detrimental to cargo handling efficiency, especially for container ships. Wave action in approach channels can increase the danger for tugboat crews and make navigation more difficult. Furthermore, dredging in exposed locations is relatively expensive - see chapter 16 of volume I. Figure 2.1 shows a small harbor protected by a breakwater.

A breakwater can also serve to reduce the amount of dredging required in a harbor entrance. This can result from the cutting off of the littoral transport supply to the approach channel, or it can result from natural scouring action in an artificially narrowed channel. This purpose was highlighted briefly in chaper 18 of volume I. Figure 2.2 shows such an application constructed in an attempt to increase natural channel scouring.

At locations where little or no natural protection exists, breakwaters often serve as quay facilities as well. Such dual usage of the breakwater is economical in terms of harbor area but requires a different type of breakwater structure. This aspect will be discussed further in section 4 of this chapter.

A fourth possible important purpose of a breakwater can be to guide the currents in the channel or along the coast. It has already been shown (volume I ch. 18) how the channel currents can be artificially concentrated to maintain depth. On the other hand, a breakwater can also be built to reduce the gradient of the cross current in an approach channel.

Ships moving at slow speed in a channel are relatively difficult to hold on course. A constant cross current makes the pilot's job more difficult but can often be tolerated. On the other hand, an abrupt change in cross current strength as the ship progresses along the channel can cause dangerous navigation situations. This is shown schematically in figure 2.3. One of the primary considerations in the design of the Europoort breakwaters in The Netherlands was the limitation of the cross current gradient. The resulting current pattern, observed in a physical model is shown in figure 2.4.

Obviously, a single breakwater can serve more than one of these four main purposes. The design requirements implied by these functional demands are discussed in section 4; in the following section we examine the general design data required.

* This definition includes coastal defense works; the rest of the discussion is limited to harbor breakwaters, however.

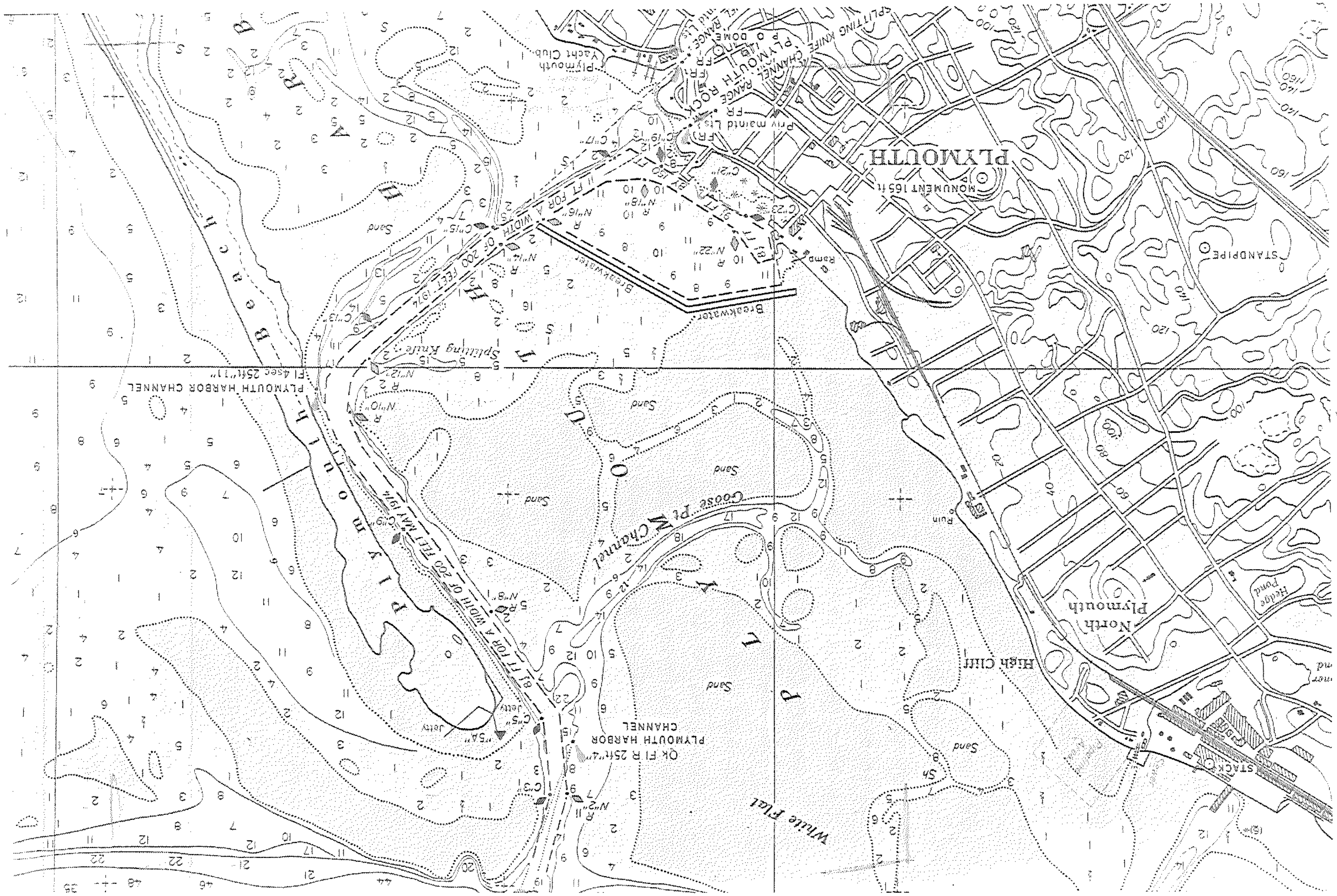


Figure 2.1
PLYMOUTH HARBOR, U.S.A.

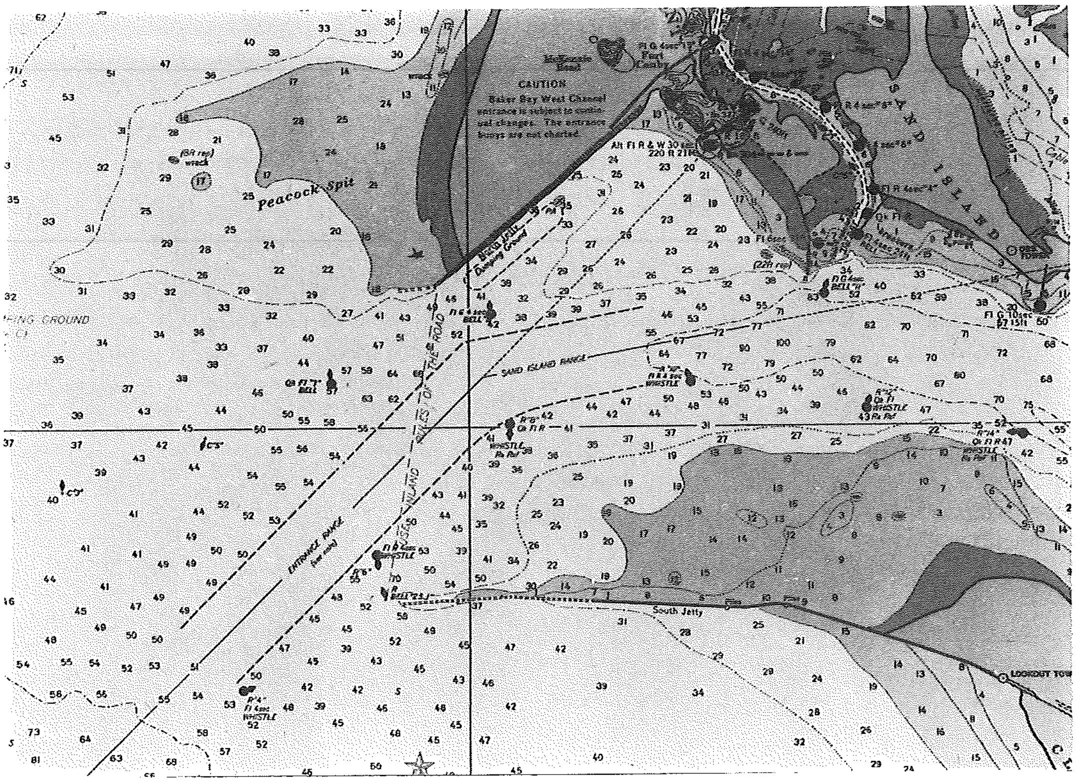
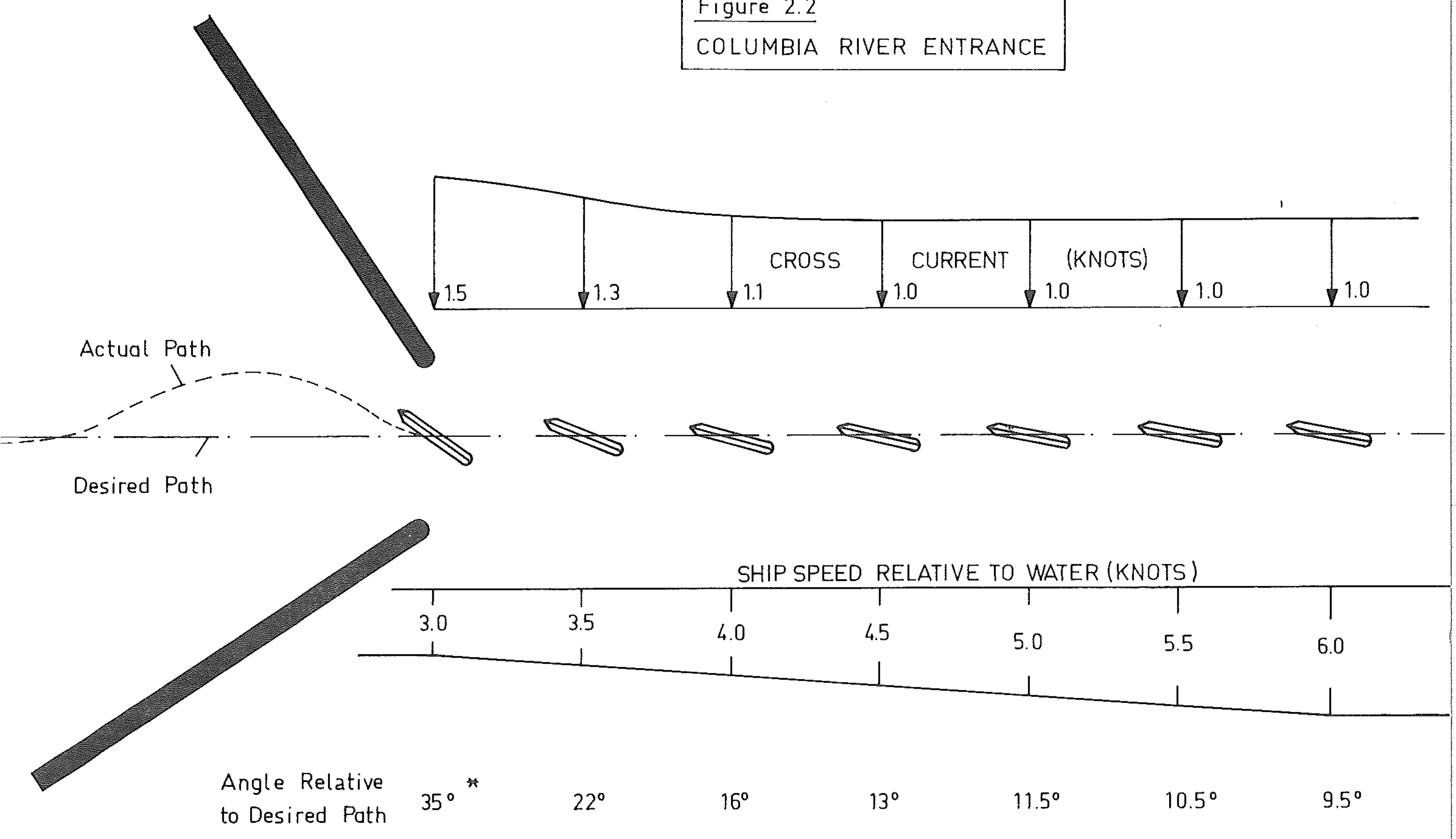


Figure 2.2
COLUMBIA RIVER ENTRANCE



* value increased from 30° by moment generated by abrupt current change.

Figure 2.3 INFLUENCE OF CROSS CURRENT ON SHIP

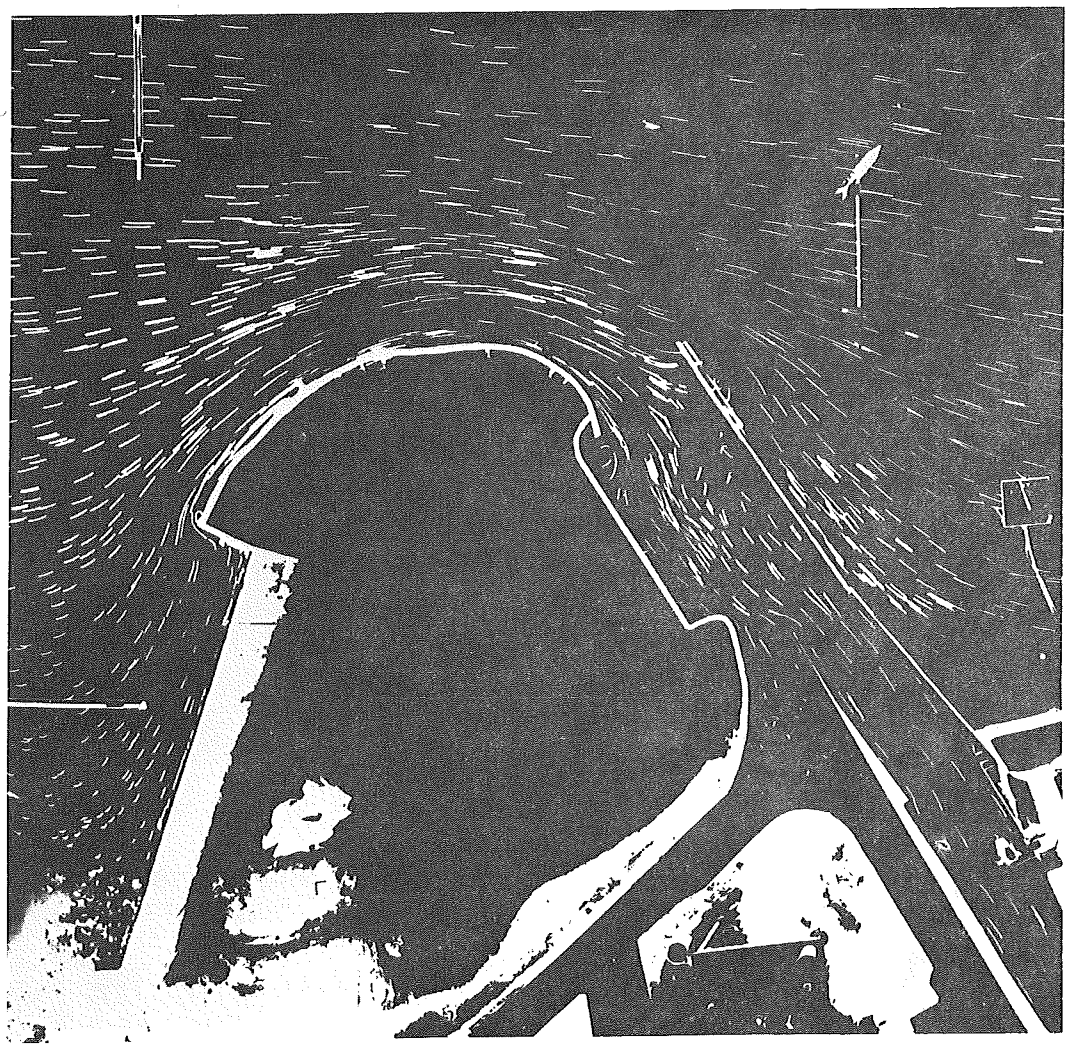


Figure 2.4
CURRENT PATTERN AT EUROPOORT ENTRANCE
HALF AN HOUR BEFORE H.W. HOOK OF HOLLAND

2.2. General Design Information

Hydrographic data are obviously important for the design of a breakwater. Bathymetry is extremely important; the volume of a rubble mound breakwater increases quadraticly with water depth. Water level changes caused either by tides or by storm surges can be important for determining the crest elevation of the breakwater. These water levels, by influencing the total water depth can also limit the wave attack to some maximum value.

Wave heights and their frequency of occurrence form the most important input to an optimum design procedure for a breakwater. The statistical relationships needed have already been presented in chapters 10 and 11 of volume I. When wave data itself is not available, waves can often be predicted from meteorological data- see volume I chapter 12 and the *Shore Protection Manual*.

Horizontal tides can also be important. In addition to hindering shipping, these currents can also result in erosion which endangers the breakwater foundation.

Meteorological data are also important. Winds are not only important for local wave generation, but can also be important for estimating the quantity of overtopping by spray from the broken waves. When the inner side of a breakwater serves as a quay, the ship mooring forces - dependent partially on wind influences - can be important in the design.

Temperature data can be important for the selection of construction materials. Special concrete must be used if repeated cycles of freezing and thawing are expected.

Special navigational aids may be needed on a breakwater in a location where fog forms frequently. These aids can range from radar reflectors to radio beacon installations.

Since every breakwater must have some sort of foundation - however simple - knowledge of the local soil conditions is necessary. The grain size distribution, cohesion, bearing capacity, and consolidation characteristics can all influence the design of a structure.

The history of the coastal morphological changes can be helpful for estimating the influence which our structure will have on the coastal environment. While not involved directly with the breakwater construction, resulting coastal morphological changes can influence the total project economics significantly. Methods for predicting these changes and reducing their detrimental effects are discussed in volume II.

Information about any special design wishes is also necessary. For example, it may be required that the entire structure be visible from within a given distance; this has implications for the crest elevation. It may be desirable to design a breakwater suitable for use by sport fishermen under certain weather conditions.

One last item involves the availability of construction materials. Since large volumes of material are needed to construct a breakwater, a local supply is nearly always required in order to keep transport cost within reason.

2.3. Sources of Design Data

Much of the preliminary hydrographic data can be obtained from navigation charts. They often provide sufficient data for site selection. The user should keep in mind, however, that indicated depths are usually minimum depths; this is in keeping with their primary use in navigation. The most up-to-date charts are usually issued by local (national) hydrographic agencies. The British Admiralty, however, issues charts covering nearly all the coasts of the world. These same hydrographic survey agencies usually accumulate and publish tidal information as well.

Meteorological data is usually accumulated most systematically by the local (national) weather forecasting service. Data on waves are also often recorded at coastal and offshore stations along with meteorological information. As an alternative, wave statistics can sometimes be derived from other information as explained in chapter 12 of volume I. Storm surge data is also often recorded at coastal stations by the weather service. Theoretical prediction is sometimes possible when measurements are lacking; an approach to the problem is outlined in volume I chapter 3.

Information about the soil conditions at a site is often more difficult to find. Possibly local public works agencies or dredging contractors who have worked in the area may be able to provide some information. Even so, a detailed geotechnical survey of the area will very often be required, especially if a large or special project is involved.

Any information concerning special design specifications, such as recreational requirements will be provided by the authority initiating the project.

Data from which an impression of coastal morphological changes can be obtained may be held by public works agencies or may be derived from comparison of present and past navigation charts. Libraries often have map collections which can be used for these comparison studies.

2.4. Performance Requirements

Several factors which can influence our choice of breakwater type have already been mentioned. These have been grouped under purpose and under design information in earlier sections of this chapter. In this section other factors affecting the choice of design type will be considered. A catalog of types of breakwaters with their advantages and disadvantages will be presented in chapter 3.

In contrast to dikes, the performance requirements for breakwaters are usually much less stringent. For example, a breakwater may be needed only temporarily such as those used to establish the beachheads in World War II. On the other hand, a permanent structure may be desirable, but this structure need only be effective intermittently. One can conceive of a ferry harbor entrance which only need be protected from wave action when the ferry is moving in or out.

Available construction and maintenance methods can also result in modified designs. If, for example, navigational aids and the breakwater itself must be repaired quickly, then a higher crest elevation may be dictated by the need to move equipment along the dam during severe weather. Indeed, for some purposes, a breakwater need not be much higher than the still water level, while for others it must be nearly as high as a dike. If quay facilities are to be provided on the inner side of the breakwater, special foundations will be required to withstand the additional loads from cargo handling and to limit settlement.

Another contrast with dike is that a breakwater need not always be impermeable. Some types of breakwaters such as air bubble curtains or floating breakwaters do little to restrict currents.

2.5. Review

The more important purposes and design and performance requirements of breakwaters have been outlined in a general way. In the following chapter, many types of breakwaters will be described briefly along with a summary of their advantages and disadvantages.

One of the most important tasks of the designer is to achieve a solution to a problem having the lowest *total* cost. This total cost can include much more than construction and maintenance costs of the breakwater; recreational, environmental, and indirect damages within a harbor resulting from breakwater failure should also be considered. This concept of optimum design has been introduced in chapter 13 of volume I.

3. TYPES OF BREAKWATERS

J.F. Agema
W.W. Massie

3.1. Introduction

The purpose of this chapter is to review and compare the various types of devices and structures available as breakwaters. This comparison treats rubble mound and monolithic breakwaters in a rather summary way; these specific types - with many variations - are discussed in more detail in later chapters. They are included here for completeness; sufficient variety is illustrated to show their versatility. These comparisons are presented in a sort of outline form in an effort to preserve the survey character of this chapter. Twenty different breakwater types are listed in alphabetical order and compared in the following section.

Specific references and examples of many of the various types are given. Two general references - *Shore Protection Manual* and Wiegel (1964) - are not listed for each type individually.

3.2. Comparison of Types

a. Air Bubble Curtains

Description: Permanent submerged pipeline discharging air to cause currents in water which tend to cause waves to break. Adapted to intermittent use to protect small areas.

Advantages: Uses no space
Reduces density currents - see Vol I, ch. 23.
Can be quickly constructed.
Does not bother shipping.
Aesthetic - invisible.
Undamaged by large waves.

Disadvantages: Expensive in operation.
Ineffective except for very short waves.
Air pipe may become covered by sediment, if used only intermittently.
Provides only a reduction in water and sediment movement.

Examples: figure 3.1

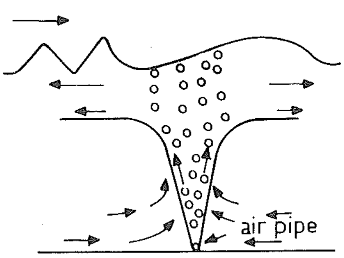


Figure 3.1
AIR BUBBLE CURTAIN

References: Schijf (1940), Laurie (1952), Taylor (1955), Griffin (1972)

b. Beaches

Description: Permanent, often natural sand or gravel slopes which destroy wave energy by breaking. Waves can be reduced in channels by refraction.

Advantages: Effective.
Use natural materials.
Usually very durable.
Usually very inexpensive to maintain.
Aesthetic - recreational value.

Disadvantages: Possible sand loss at exposed locations.
Need much space - slopes of 1:10 or flatter are usually needed.*

Examples: Europoort Entrance
References: Volume II of these notes.

c. Composite - Rubble Mound Front

Description: Permanent structure consisting of some form of monolithic vertical breakwater with a rubble mound form placed before and against it. This is often used to refurbish old monolithic vertical breakwaters.

Advantages: Low reflection of waves.
Moderate material use.
Impervious to water and sediment.
Can provide quay facilities on lee side.
Can be built working from structure itself.

Disadvantage: Expensive form of new construction since it uses a multitude of construction techniques.

Example: Improved old breakwaters at Scheveningen and IJmuiden.

d. Composite - Vertical Monolithic Top

Description: Permanent structure consisting of a rubble mound base surmounted by a monolithic vertical structure.

Advantages: Moderate use of material.
Adapts well to an uneven bottom.
Provides a convenient promenade.

Disadvantages: Suffers from impact forces of largest waves.
Reflects largest waves. This can damage the lower rubble mound portion.
Rubble mound must be carefully constructed in order to provide a good foundation for the monolithic top.
Destroyed when design conditions are exceeded.

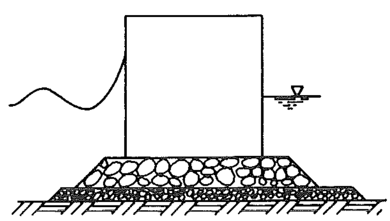


Figure 3.2
COMPOSITE BREAKWATER

Examples: figure 3.2

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* The slope needed is dependent upon the material grain size; finer materials need flatter slopes.

e. Floating Flexible

Description: Temporary flexible bouyant floating device which absorbs wave energy by friction with water and from internal deformation.

Advantages: Inexpensive, usually.
Easily moved from site to site.
Often very quickly fabricated.
Relatively independent of bottom conditions.

Disadvantages: Ineffective against long waves.
Must be anchored.
Some types such as brushwood mattresses require much skilled labor for fabrication.

Examples: Brushwood mattresses.
Floating auto tires.
floating plastic mats.

References: Wiegel, Friend (1958), Griffin (1972), Kowalski (1974).

f. Floating Rigid

Description: Usually a temporary solution consisting of a large floating body. This may be a ship or a large shallow pontoon.

Advantages: Easily moved to new site.
Usually consume little space.
Can provide temporary quay facilities.
Independent of bottom except for anchors.

Disadvantages: Ineffective for long waves.
Must be anchored.
Can resonate leading to poor performance at some wave frequencies.
Damaged when design conditions exceeded.

Examples: Large ships or pontoons.

References: Griffin (1972), Kowalski (1974).

g. Monolithic "Floating"

Description: Semipermanent concept for a monolithic breakwater suitable for use on mud coasts where the bottom material bearing capacity is limited. The structure consists of a large caisson or ship floating with its hull projecting some meters into the mud.

Advantages: Easily placed.
Well adapted to very soft bottom.
Not prone to settle.

Disadvantages: May move with large mass slides of the mud - see vol. I, ch. 27.
Subsequent dredging prohibited in the area.

h. Monolithic - Porous Front

Description: A permanent monolithic structure having a porous front wall which acts to absorb the oncoming wave energy.

Advantages: Uses relatively little material compared to rubble mound.
Less wave impact and reflection than conventional monolithic structure.
Needs little space.
Provides quay on lee side.

Disadvantages: Difficult to construct.
Need high quality concrete and workmanship.
Even bottom needed.
Intolerant of settlement.
Foundation problems on fine sand.
Severe damage when design condition exceeded.

Examples: Ekofisk storage tank, North Sea
Baie Comeau, Canada

References: Jarlan (1961)
Marks & Jarlan (1969)
Griffin (1972)
chapters 13 through 19.

i. Monolithic - Sloping Front

Description: A monolithic structure with the upper portion of the vertical face sloping back at an angle of in the order of 45^{o}.
This is often called a Hanstholm type of breakwater.

Advantages: Economical of material.
Rather quickly constructed.
Less wave impact and reflection when compared to conventional monolith.
Occupies little space.
Quay facilities can be provided on lee side.

Disadvantages: Needs even bottom.
Intolerant of settlement.
Can have foundation problems on fine sand.
Severe damage when design condition exceeded.

Examples: Bristol, England

References: chapters 13 through 19.

j. Monolithic Sunken Caisson

Description: A temporary structure floated into place and sunk and ballasted to form an initial breakwater. Often used to cut off currents so that it can then be burried in a natural beach, or other more permanent breakwater.

Advantages: Very quickly placed on the site.
Can provide quay facilities on lee side.
Occupies little space.
Uses little material.
Provides promenade.
Provides work road for later construction phases.

Disadvantages: Size limited by towing limitations.
Easily damaged - often by only a moderate storm.
Foundation difficulties on fine sand bed.
Requires smooth bed.

Examples: Normandy beachhead - world war II.

References: chapters 13 through 19.

k. Monolithic Vertical - Constructed in Place

Description: Permanent structure consisting of large elements stacked upon each other in a regular pattern forming a massive vertical wall.

Advantages: Economical of material.
Rather quickly constructed.
Occupies little space.
Quay can be provided on lee side.
Adapted to use of pile foundation.
Top is accessible to construction equipment.

Disadvantages: Needs even bottom.
Wave impact forces can be locally severe.
Waves are reflected.
Erosion can take place near the bottom.
Inflexible if settlement occurs.
Needs very heavy construction equipment.
Foundation problems on fine sand, except when on a pile foundation.
Severly damaged when design conditions are exceeded.

Examples: Original breakwaters in Scheveningen and IJmuiden.

Reference: Chapters 13 through 19 of this book.

l. Oil Slick

Description: very temporary emergency measure used at sea to reduce spray in heavy seas. Effectiveness derives from surface tension influences.

Advantages: Inexpensive.
Easily implemented under emergency conditions.

Disadvantages: Little, if any, actual wave reduction.
Aesthetic - pollution source.

m. Pile Row

Description: Permanent structure formed by driving a row of piles either close together or spaced apart. Suitable for groins as well as simple breakwaters.

Advantages: Inexpensive.
Uses very little space.
Well adapted to poor foundation conditions.
Can be incorporated in quay structure.
Can be rather watertight or open as desired.

Disadvantages: wave reflection.
Possible scour at bottom.
Wood piles attacked by worms and rot.

Examples: Evanston, U.S.A.

References: Wiegel (1961).

n. Resonant Breakwater

Description: A series of rectangular basins connected to a harbor entrance such that each is tuned to absorb energy of a given commonly occurring wave period. In contrast to ch. 19 of Vol. I, a seiche is encouraged in these basins.

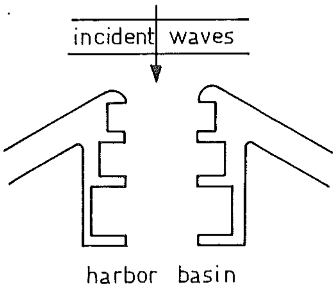


Figure 3.3
RESONANT BREAKWATER

Advantages: Can help reduce seiches in main harbor.
Can be built on soft ground.

Disadvantages: Sharply tuned to specific waves.
Takes much space.

Example: Dunkerque near lock.

References: Valembois (1953)
figure 3.3.

o. Rubble Mound - Pell - mell Artificial Armor Units

Description: A permanent structure consisting of layers of stone and gravel protected on the exposed surfaces by a layer of randomly placed artificial armor units. A massive structure may be incorporated in the crest to save material.

Advantages: Durable.
Flexible - accommodates settlement.
Easily adapted to irregular bathymetry.
Needs no large natural units.
Functions well even when severely damaged.

Disadvantages: Need factory for armor units.
Large quantities of material needed.
Needs underlayer if built on sand.
Unsuited to soft ground.

Example: Europoort, The Netherlands
Sante Cruz, U.S.A.

References: Agema (1972)
chapters 4 through 12.

p. Rubble Mound - Placed Units

Description: Permanent structure similar to that with pell - mell unit placement except that units are now individually placed in a precise pattern. A monolithic crest construction is usually used.

Advantages: Durable.
Flexible - adapts to settlement.
Uses least material of rubble mound types.
Adapts well to irregular bathymetry.
Well adapted to "dry" construction.

Disadvantages: Armor units must be fabricated
Needs much skill in construction.
Impossible to place armor under water.
Unsuited to very soft ground.
Needs underlayer if built on sand.

Examples: Nawiliwili Kauai, U.S.A.

References: Palmar (1960), Agema (1972)
chapters 4 through 12

q. Rubble Mound - Stone

Description: Permanent structure consisting of successive layers of stone. The exposed surface is covered with heavy armor stones.

Advantages: Very durable - resists severe attack well.
Functions even when severly damaged.
Adapts to ground settlement.
Uses natural commonly available materials.
Easily adapted to irregular bathymetry.
Construction possible with limited skilled labor.
Uses common construction equipment.
Materials are usually inexpensive.
Much experience available.

Disadvantages: Uses the most material of all types.
Must be adapted for construction on sand.
Unsuited to very soft ground.

Examples: Marina Del Rey, U.S.A.
Winthrop Beach, U.S.A. - See Vol. I, ch. 28 fig. 28.7a.

References: Chapters 4 through 12.

r. Rubble Mound - Stone with Asphalt Spotting

Description: A stone armored rubble mound breakwater with lighter armor partially keyed together by scattered patches of asphalt.

Advantages: Lighter armor units than would otherwise be possible with stone.
Flexible for settlement.
Easily adapted to uneven bathymetry.
Adapts to ground settlement.

Disadvantages: Asphalt plant needed.
Very skilled labor needed to place asphalt.
Asphalt can be ineffective in hot weather.
Failure can lead to severe damage.

s. Submerged - vertical or rubble mound

Description: Permanent structure sometimes used to create an artificial tombolo, for groins.

Advantages: Can be designed for desired wave reduction.
Aesthetic - invisible.
Reduces longshore sand transport.

Disadvantages: Prevent onshore sand transport.
Hazardous to shipping.
Foundation problems on sand sometimes important.

Examples: Groins on Dutch Coast.

References: Johnson, Fuchs, Morison (1951) chapter 5.

t. Vertical Sheet Pile Cells

Description: Permanent breakwater or groin construction consisting of sheet pile cells filled with sand, and usually capped with pavement.

Advantages: Inexpensive.
Can be constructed from land with small equipment.
Well suited to sand and mud bottom.
Usually quite durable.
Rather fast construction.
Provides road or promenade.
Insensitive to bottom settlement.

Disadvantages: High wave reflection.
Corrosion can limit life.
Possible local bottom scour.

Examples: Presque Isle, U.S.A.
Port Sanilac, U.S.A.

3.3 Conclusions

It is obvious from the previous section that no one type of breakwater is always best. Further, the choice of a breakwater for a given situation is dependent upon so many factors that it is nearly impossible to give specific rules of thumb for determing the "best" type. A few general rules can be given, however:

- Rubble mound structures are the most durable, and as such are best suited to extremely heavy wave attack.
- Monolithic structures use less space and material; this is especially true in deeper water.
- Special types of breakwaters are usually best suited to specific special applications.

Details of rubble mound breakwaters are worked out in the following nine chapters; problems of monolithic breakwaters are taken up in chapters 13 through 19.

4. RUBBLE MOUND BREAKWATERS

J.F. Agema

4.1. Definition

What is a rubble mound breakwater? The cynic's description "a pile of junk" is not too bad provided that a couple of qualifications are added. The first qualification is that the "junk" must be some relatively dense material such as stone or concrete elements (compressed scrap auto bodies have also been suggested). The second is that the "pile" must be built up in a more or less orderly fashion. In the remainder of this chapter we briefly describe the parts of a rubble mound breakwater and their interrelationships.

4.2. Two Distinct Types

The use to be made of the area directly leeward of a rubble mound breakwater plays an important role in the choice between an overtopping or non-overtopping rubble mound structure. In general, the less important or critical the activity on the lee side, the more overtopping that may be allowed. For example, if containers are to be loaded in the immediate lee area (an operation very sensitive to harbor wave action), very little, if any, wave overtopping would be acceptable. If, on the other hand, a breakwater served primarily to guide the current near a harbor entrance, the regular overtopping would be of no consequence.

If a breakwater is designed to be overtopped, then special measures must be taken to assure that the upper portion of the *inner* slope is not damaged. A non-overtopping breakwater, on the other hand, must be so designed that it is, indeed, nearly never overtopped. Typical cross sections of these two types are shown in figures 4.1 and 4.2.

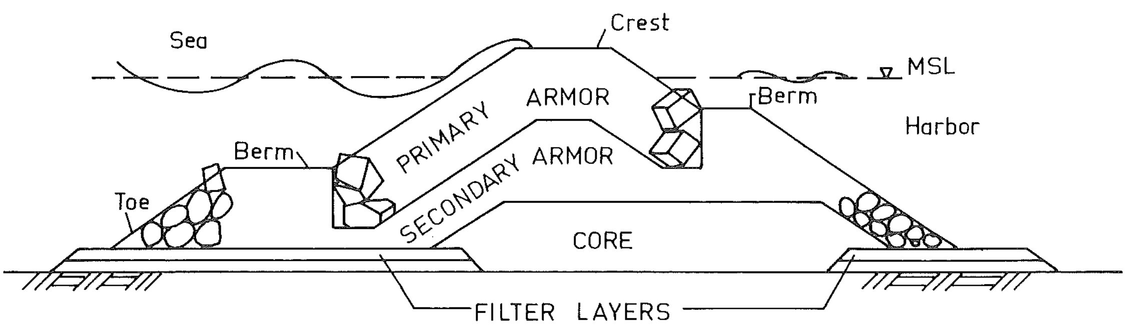


Figure 4.1
OVERTOPPING BREAKWATER

A non-overtopping breakwater is usually somewhat higher - relative to the design still water level - than an overtopping one. The amount of wave run-up and overtopping on a given slope of given height is discussed in chapter 5.

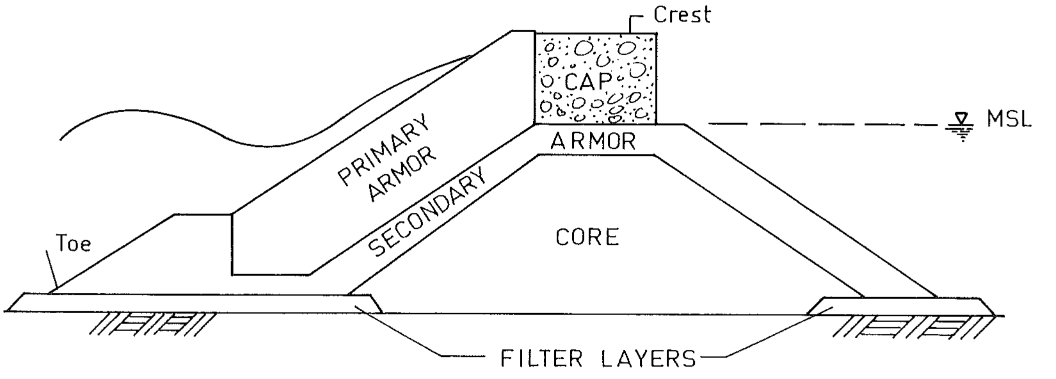


Figure 4.2
NON OVERTOPPING BREAKWATER

4.3. Basic Construction Principles

Nearly every rubble mound breakwater is constructed in layers. These have already been indicated in figures 4.1 and 4.2. As a general rule, each layer of the breakwater must be so designed that the adjacent layer of finer material cannot escape by being washed through its voids. Obviously, the outer layers - both in final form and during construction - must be designed to withstand the expected wave attack. This is discussed in detail in chapter 7. Of course, these layers must also be designed such that they can be constructed with the available equipment - see chapter 10.

The choice of construction materials is largely determined by availability in the quantities needed. Necessary properties of these construction materials - especially of armor units - are cataloged in chapter 6.

Many times the outer layers of the breakwater can be supported by a rather undescribable core material. Usually, the cheapest available material is thrown in - see chapter 8.

The rule that adjacent layers may not be allowed to wash through voids applies to the natural bottom material layer under the breakwater as well. There are no problems when a rubble mound is constructed on a rock bottom. If, on the other hand, the bottom material is fine sand, then a filter must usually be constructed. This filter is described in detail in chapter 9.

Once a breakwater has been conceived (its general dimensions and properties are sketched) this concept must be economically evaluated. This application of the optimum design technique, described in chapter 13 of volume I, is handled in detail in chapter 11.

5. WAVE RUN-UP AND OVERTOPPING

A. Paape

5.1. Introduction

Reflection of waves against a slope or the breaking of waves on some form of breakwater leads to water level fluctuations on the slope surface which can considerably exceed the amplitude of the incident waves. For example, when waves are fully reflected by an impermeable vertical barrier, the water level fluctuation at the wall is theoretically two times the height of the incident waves, H_i.

When waves break on a slope, a portion of their momentum is transferred to a tongue of water rushing up the slope. The run-up, R, is defined as the maximum vertical elevation reached by this tongue measured relative to the still water level - see figure 5.1. It is implied in this definition that the crest of the slope is higher than the run-up. Since the run-up is measured relative to the still water level, the run-up, R, also includes effects of wave set-up caused by the radiation stress - volume II.

5.2. Run-up Determination

When regular waves are considered, a unique relationship exists between the wave run-up, R, and the wave properties, height and period, and structure characteristics, toe depth, slope angle, roughness, porosity, and foreshore slope. These parameters are also shown in figure 5.1. Thus:

$$R = f(H_i, T, h_t, \alpha, \beta, r, n)^{*} \qquad (5.01)$$

where:

H_i is the incident wave heigt,
h_t is the depth at the toe of the slope,
n is the porosity of the slope,
r is the roughness of the slope,
R is the vertical wave run-up,
T is the wave period,
α is the slope of the structure
β is the slope of the foreshore

* It has been assumed that the wave crests approach parallel to the breakwater.

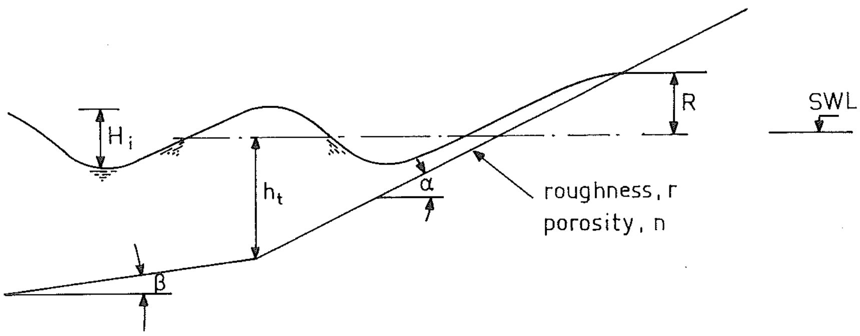


Figure 5.1
WAVE RUN-UP DEFINITION SKETCH

The energy of the waves approaching is, in general, partially destroyed by breaking, partially reflected, and partially expended in run-up. The wave height, water depth, and wave period determine the initial wave steepness. This steepness, combined with the slope, α, determines the breaking characteristics of the wave - see volume I chapter 8. This characteristic determines the ratio of reflected momentum to momentum consumed in run-up. Thus, for constant slope and foreshore properties (h_t, α, β, r, n) and wave period (T), the run-up will *not be a linear function* of the incident wave height. Experimental data is presented in figure 5.2. In this figure, H_0' is the equivalent deep water wave height, hadthere been no refraction; and λ_0 is the deep water wave length - see volume I chapter 5. The slopes listed give the ratio vertical: horizontal and correspond, therefore, to the cotangent of the slope angle, α. The smooth slopes are impervious. Sand beaches can also be treated as impervious. The curves for rubble mound slopes are for complete rubble slopes and not for just a rubble-covered surface.

The influence of the slope, α, is obvious from figure 5.2. For steep slopes, the reflection is greater and the run-up is, in general, less. On the other hand, for very flat slopes, the up-rush is retarded by friction over the long distance so that the height reached is also less than the maximum.

Nearly all of the run-up information available is of an experimental nature, and most applies to impervious structures such as dikes. An extensive critical bibliography can be found in an anonymous report (1972) entitled *Golfoploop en Golfoverslag*.*

It is obvious that a more complicated situation exists when irregular waves are involved. Because the wave properties now vary continuously the run-up also becomes a stochastic variable. d'Angremond and van Oorschot (1968) report that the statistical properties of the run-up are dependent upon more than just wave characteristics for a given slope.

* An English translation has also been prepared.

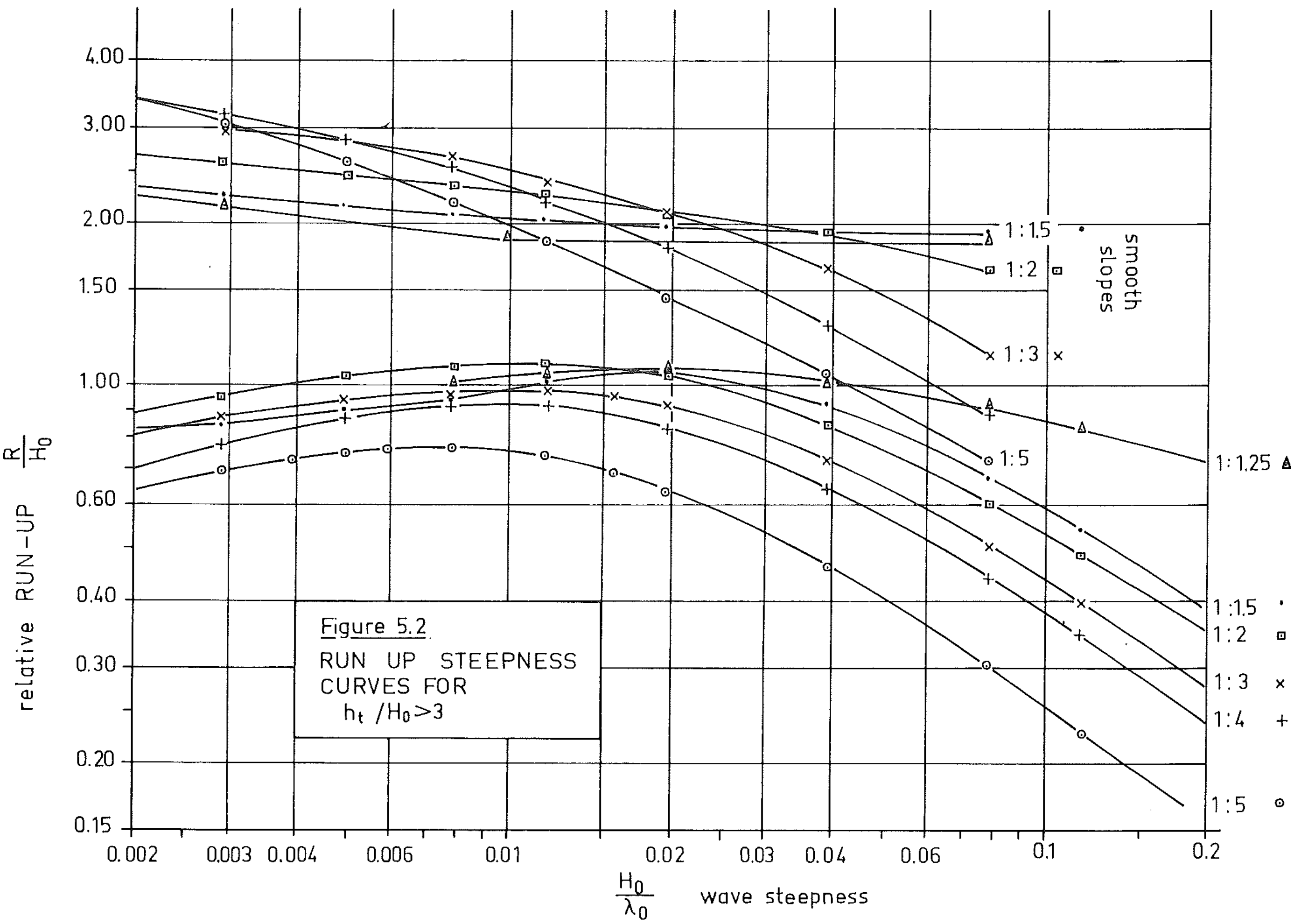


Figure 5.2
RUN UP STEEPNESS CURVES FOR $h_t/H_0>3$

The form of the wave spectrum in addition to its characteristic wave heigt and period is important for the statistical description of the run-up. Saville (1962) and Battjes (1974) have made reasonably successful attempts to relate run-up data obtained with regular waves to that obtained with irregular waves. All of this was done for smooth impermeable slopes.

Still less is known about run-up caused by irregular waves on rough permeable slopes such as found on rubble mound breakwaters. The principles involved are the same, but the roughness and permeability also have a definite influence and tend to make the effect of other parameters less pronounced. These facts are revealed by figure 5.2.

Obviously, run-up is very important for the design of a dike; its importance in breakwater design is highlighted in the next section.

5.3. Run-up in Relation to Breakwater Design

Three factors are of importance when considering run-up influences on a breakwater. These are: the stability of the structure, the use of the crest, and the effect of overtopping on the harbor. Each of these is examined in more detail below.

The stability and safety of a structure are only jeopardized by run-up when the crest and inner slope cannot withstand water running over their surfaces; This is often true of dikes. Under such conditions, it is reasonable to design the structure so as to prevent run-up reaching the crest (overtopping), even under exceptional wave and water level conditions such as those used to determine the face stability. Such an extreme limitation is usually uneconomical for a breakwater.

When the crest has a function in the harbor operation, such as acting as a roadway or pipeline street, then very occasional overtopping can usually be allowed. "Occasional" here usually means that it occurs under relatively moderate wave conditions such as might occur once or a few times per year. Obviously, this results in a lower crest elevation than that determined by the first criterium. With such a design the effects of mass overtopping under extreme conditions must be adequately considered in the design evaluation. Resulting damage to a highway or pipelines must be included, for example.

The effect of overtopping, either by wave run-up or spray is difficult to estimate. Overtopping by run-up will be considered in section 5.5. Overtopping by spray is more dependent upon the wind and breakwater slope properties than on the crest elevation. Spray should preferably be reduced by avoiding the formation of "spouting" breaking waves.* These can be reduced by limiting the vertical portions and abrupt discontinuities on the front slope.

5.4. Conclusions about Run-up

Wave run-up on rubble mound structures is, fortunately, usually less critical than on dikes or sea-walls. In spite of its restrictions, data presented in figure 5.2 can often be used. When using this figure with irregular waves, the significant wave height is usually used in place of the monochromatic wave height. Such an approach yields a fair, and usually safe, preliminary design. However, only if the project is of very modest size or the crest elevation of the breakwater must be relatively high for other independent reasons, is it justifiable not to conduct model experiments to investigate run-up and overtopping effects. One should be especially careful when long wave lengths are encountered. Several model studies have indicated that unexpectedly great overtopping can occur then.

5.5. Wave Overtopping

If the crest elevation is lower than that corresponding to maximum run-up, then up-rushing water will spill on to and over the crest of the structure. The usual unit of measurement of overtopping is volume per unit time and crest length. This quantity of overtopping is sometimes used as a damage criterium for sea walls. It can also be used to dimension a drainage system to remove this overtopping water. The "direct"

* This should be compared to chapter 15.

relevance of overtopping is usually less for a breakwater than for a seawall unless important harbor operations are carried out from or close behind the structure.

In principle the factors which lead to a decision on allowable run-up also lead to a decision with regard to overtopping. However, some pertinent observations are in order.

Overtopping which may endanger a breakwater's stability has never been related to the quantity of water as such. Model test results relate the wave conditions and crest elevation directly to structural damage or required armor unit weights. This is, of course, more straightforward.

The amount of overtopping can be a criterium to evaluate a design employing the breakwater crest in the harbor operation. This evaluation is parallel to that already mentioned in section 5.3.

When the overtopping flow is considerable and the water must return to the sea via the harbor, currents will be generated behind the breakwater. Obviously the quantity of overtopping must be appreciable; the crest elevation is relatively low. A special model study of overtopping was carried out for the Europoort Project. A few other examples can be found in the literature but not enough is known to establish a general prediction relationship; usually special model studies are needed.

When the crest elevation is still lower, the overtopping water will generate waves in harbor basins as well. This wave generation is dealt with in the following section.

5.6. Wave Transmission

When the crest of a breakwater is relatively low compared to the wave height the resulting large volume of overtopping can generate appreciable waves on the lee side. The following rules of thumb are suggested:

$$\text{for } \frac{z_c}{H_i} > \frac{3}{4} : \text{ minor waves} \tag{5.02}$$

$$\text{for } \frac{z_c}{H_i} = 0 : \frac{H_t}{H_i} \approx \frac{1}{2} \tag{5.03}$$

$$\text{for } \frac{z_c}{H_i} < -\frac{1}{2} : \frac{H_t}{H_i} > \frac{3}{4} \tag{5.04}$$

where:

H_i is the incident wave height,

H_t is the transmitted wave height, and

z_c is the elevation of the crest above the still water level.

The above equations can be used with regular as well as with irregular waves if the significant wave height is taken to characterize the spectrum.

The above rule of thumb is only very approximate. In principle, all of the factors governing wave run-up as well as the breakwater crest width affect wave transmission. In practice, the most important parameters are the incident wave characteristics - determined by H_o', T, and h - and the crest elevation, z_c. The slope roughness and angle are only important for gentle slopes and wide crests (10 m or more).

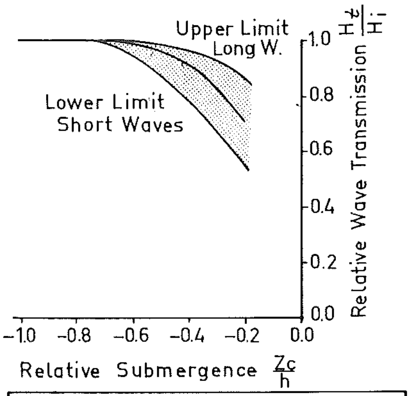


Figure 5.3
WAVE TRANSMISSION FOR SUBMERGED BREAKWATERS

For a submerged structure (z_c negative), the most important parameter is $\frac{z_c}{h}$. Figure 5.3 shows some experimental results. The effect of wave steepness is also indicated. Longer waves result in greater wave transmission. Figure 5.3 does not disagree with relations 5.03 and 5.04. This figure *may not be* extrapolated!

When the crest is near the still water level, or the waves are short and steep, a more dependable parameter for wave transmission is the ratio $\frac{z_c}{H_i}$. Thus, figure 5.3 becomes less dependable near $\frac{z_c}{h}$ equal to zero. See Hall and Hall (1940).

Some further data is presented in the *Shore Protection Manual* but not presented in a very handy usuable form. One must be very careful when attempting to use their graphs such as ~~figure~~ page 7.59 in that book; *all* of the parameters must match those used to make their figures.

A correct conclusion is that too little information on wave transmission is available in the literature to allow accurate estimates to be made during design. A factor which makes the establishment of allowable limits for wave transmission even more difficult is the simultaneous presence of waves which penetrate through the harbor entrance. The resulting total wave height *is not* simply the sum of the wave height components! Even a sum based upon wave energy proves to be unreliable. Large scale model tests can provide insight into the problem for specific harbors.

For completeness, we should realize that waves may also penetrate *through* rubble mound breakwater. After all, it is, in principle, often a permeable structure. In practice, this permeability to wind waves is usually low, due to the fact that the waves are relatively short and the possible presence of a breakwater core consisting of fine material - see chapter 8. However, if the breakwater is built almost exclusively from coarse material (concrete blocks, for example) and the wave period is long (more than 12 seconds in order of magnitude), this wave penetration may no longer be negligible. Because of the nonlinear character of the flow through such a coarse porous medium, scale effects can cause severe problems for the interpretation of model data. Veltman-Geense (1974) has attacked the problem of wave penetration both theoretically and experimentally.

Properties required of armor units used to protect the exposed faces of breakwaters are discussed in the following chapter.

6. CONSTRUCTION MATERIALS

J.F. Agema

6.1. Necessary Properties

Obviously materials used in rubble mound breakwaters must have certain properties. One of the more important properties is durability; the material must be able to resist its environment for the economic life of the breakwater.

Environmental attack can come from various sources. Waves, especially breaking waves, can exert high dynamic pressures on material surfaces. The outer armor layer, especially, must be able to resist these forces - see chapter 15. As will be indicated there, impact forces are most severe on flat vertical or nearly vertical surfaces. Therefore, irregularly shaped armor units are most often used. Sea water and polluted harbor water can attack breakwater materials chemically. Thus, the materials may not disolve or even corrode rapidly in the environment. Sunlight can influence the long term properties of materials such as Nylon used for filter constructions - see chapter 9. Normally, such filters are well protected from sunlight and no problems result. Asphalt can soften under the influence of heat from the sun. This may have contributed to the damage caused to the breakwater in IJmuiden by a late summer storm.

In addition to resistance to environmental attack, the materials must have a reasonably high density. As will be shown in the following chapter, the weight of individual armor units required is strongly dependent upon their density. Obviously, they must be more dense than water, but additionally their resistance to displacement resulting from friction forces is also related to their net underwater weight.

Additionally, it is necessary that the breakwater materials be inexpensive. This is especially true for a rubble mound breakwater which uses a relatively large volume of material. Inexpensive does not necessarily mean that the cheapest raw material must always be used, however. For example, use of a more expensive material such as special concrete armor units may result in sufficient savings on other materials and construction equipment to prove to be economical. This item will come up again in chapter 11 on optimum design.

As in indicated in section 3 of chapter 4, each succeeding layer of a rubble mound breakwater must be capable of "containing" its adjacent layer of finer material. This implies that the voids between elements of a layer may not be too large relative to the size of material in adjacent layers.

6.2. Desirable Properties

While the following properties are not absolutely necessary, materials having these properties in addition to those listed above can prove to be more economical.

Materials which pack into rather porous layers (have high void ratio) tend to damp the waves more effectively. Also a savings in total weight of material results and wave forces acting on the outer layers are reduced. On the other hand, this desirable large porosity can be in conflict with the containment property for armor layers listed above.

Armor units which more or less interlock can prove to be more resistant to wave forces since a locally high wave force is distributed throughout several units. If, this interlocking is disturbed, however, severe damage can result. Conservatism in the design of breakwater crests and ends is often advisable, since interlocking effects are least pronounced where an armor layer curves sharply - see chapter 7.

6.3. Characterizing Coefficients for Armor Units

Now that the properties of rubble mound breakwater materials in general and of armor units in particular are well defined, we need to translate these properties into quantitative parameter values suitable for use in computations. Luckily, these properties can be reduced to four parameters, two of which are important for stability. These are each discussed a bit below; values for them for specific armor units are given in the following section. Their use in computations is explaned in chapter 7.

The most straightforward property of an armor unit to express quantitatively is its mass density, ρ_a. Since the density is only dependent upon the material used in the armor unit, densities of the common armor unit materials will be discussed here.

Granite, the most common natural armor stone ranges in density from 2650 kg/m^3 to 3000 kg/m^3 with most sorts having a density near 2700 kg/m^3. Basalt, another commonly used stone, has a density of 2900 kg/m^3. Very occasionally, limestone blocks are used in a breakwater. Its lower resistance to environmental attack and lower density - 2300 to 2750 kg/m^3 - are a handicap.

Concrete for armor units usually ranges in density between 2300 and 3000 kg/m^3. Special aggregates needed to achieve even higher concrete densities usually prove to be too expensive to be economical. The concrete used should have a 28 day strength of at least 30 N/mm^2.
The remaining properties of an armor unit - shape, degree of interlocking, roughness, location on breakwater, etc. - are combined into one so called damage coefficient, K_D. This emperically determined coefficient and the density, ρ_a, determine the necessary block weight for a given slope geometry and wave condition - see chapter 7.

Two other parameters are of primary importance for dimensioning and pricing a breakwater. The first of these indicates the degree to which the armor units pack together and is called a layer coefficient, K_Δ. It represents the ratio of the length of a typical dimension of the armor unit to the length of the edge of an equivalent cube and is used to determine layer thicknesses.

Lastly, the volume of voids in an armor layer is given by its porosity, n, the ratio of void volume to total volume. This is used, primarily, in determining the number of armor units needed for a given project.

Details about a variety of armor units, listed in alphabetical order, are given in the following section. Agema (1972) and Hudson (1974) also give summaries of available block forms.

Unless otherwise specified, damage coefficient values are given for a double layer of randomly placed armor units subjected to non-breaking waves in the main body of the breakwater.* "Percent damage" refers to the percentage of armor units in the area exposed to attack which are displaced so far that they no longer fulfill their function as armor. This rather arbitrary damage measurement is chosen for its ease of measurement (via counting) and utility in optimum design procedures.

6.4. Armor Unit Types

a. Akmon

An anvil shaped plain concrete block - the name comes from the Greek for anvil - developed in 1962 by the Delft Hydraulics Laboratory. A photo of such a block is shown in figure 6.1. Because of their high K_D value, a massive monolithic crest is suggested. The density of the blocks is the same as that for concrete. The damage coefficient has been found to vary according to the allowable damage as follows:

Damage (%)	K_D
0	4.8
1	11.
2	12.
5	$\approx$ 17

Further, slopes of up to 1:1.33 are possible. The porosity, n, is 55 to 60%, and the layer coefficient, K_Δ is about 1.00. The data presented above are based upon only a limited number of model tests.

Reference: Paape and Walther (1962)

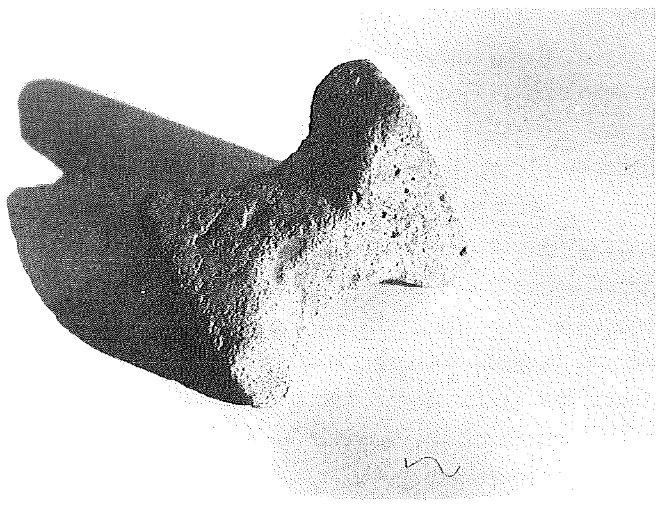


Figure 6.1
AKMON ARMOR UNIT

* See chapter 7 and *Shore Protection Manual*.

b. Cob

The cob is a hollow concrete block made by casting only the edges of a cube - see figure 6.2. They are normally placed in a regular pattern in a single layer; they must be placed with their sides touching.

Preliminary model test data indicates that cobs have very high damage coefficient values, but give no quantitative information. Instead, it is suggested that model tests be conducted when specific applications are being considered. A monolithic crest construction will be required in order to guarantee their stability.

Cobs have a porosity of about 58% and a layer coefficient, K_Δ of 1.33. This high porosity implies that a major part of the core containment function must be accomplished by lower armor layers.

Reference: Anon (1970): Artificial Armouring of Marine Structures.

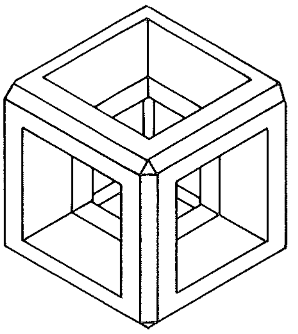


Figure 6.2
COB

c. Cube

Cubes of stone or concrete have been used as breakwater armor for centuries. As such, they are, with natural stone, the oldest units. Figure 6.3 shows a photo of a concrete cube. Obviously, their density is dependent upon the concrete used. Cut stone cubes are no longer economical now that concrete can be worked so efficiently.

Damage coefficient values are listed below:

Damage (%)	K_D
0	3.5
1	7.
2	8.
5	$\approx$ 14.

Randomly placed cubes have a porosity of about 47% and a packing coefficient, K_Δ, of about 1.10.

Reference: Paape and Walther (1962).

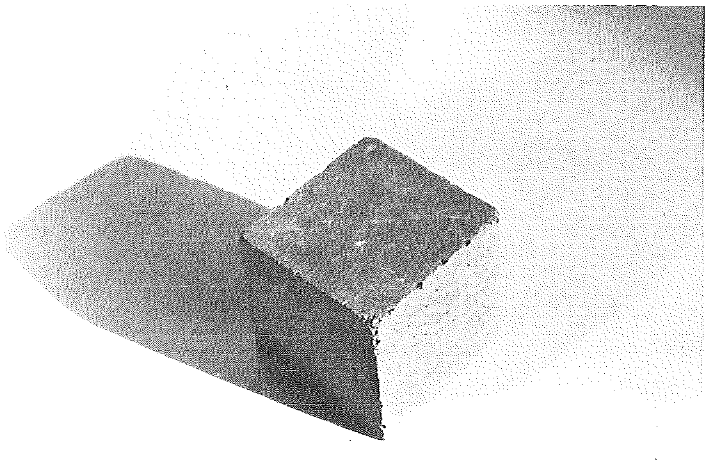
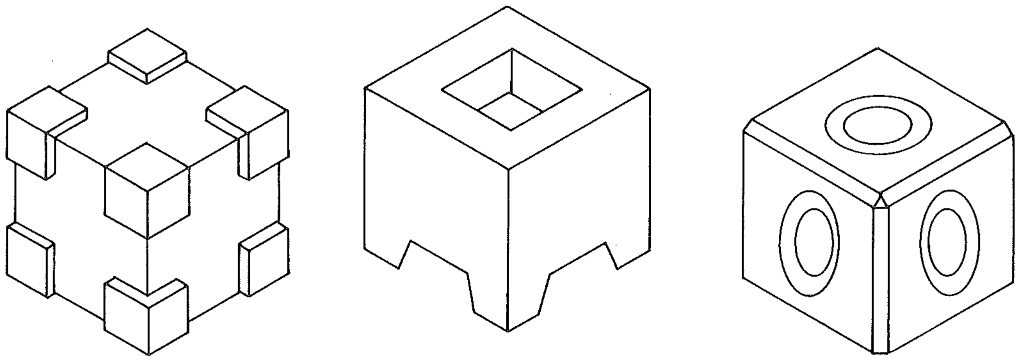


Figure 6.3
CONCRETE CUBE

d. Cube, modified

Various attempts have been made to modify the cube form in order to increase its damage coefficient value and save material. Three of the forms proposed are shown in figure 6.4; all are made from plain concrete. Since so little data is available and a certain degree of confusion exists about the naming of these blocks, no specific design data is presented.

References: Agema (1972)
Shore Protection Manual
Hudson (1974)



a. BH 3 block   b. modified cube   c. stolk block

Figure 6.4
MODIFIED CUBE FORMS

e. Dolos

Dolosse are anchor shaped plain concrete armor units designed to interlock with each other even when placed randomly. Figure 6.5 shows such a unit, developed in South Africa.

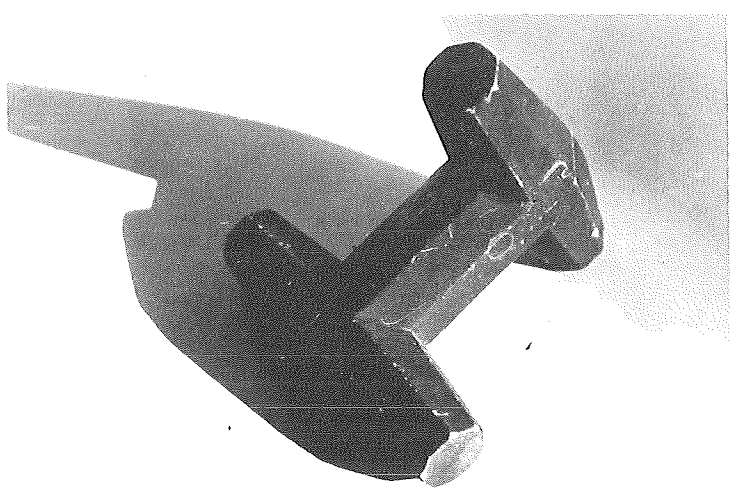


Figure 6.5
DOLOS

Because of its good interlocking capability, the dolos has the highest damage coefficient value - K_D = 22 to 25. Because of this, a breakwater face may fail by means other than armor unit displacement down the slope. A slip failure of the entire slope is the most probable unless slopes flatter than 1:2 (vertical:horizontal) are used.

Dolosse have a porosity, n, of 63% and a layer coefficient, K_Δ, of 1.00.

f. Quadripod - see Tetrapod

g. Quarry Stone - Rough

This is natural stone obtained by blasting within a rock quarry. It is characterized by a very rough, angular, irregular shape.

Such stone has a damage coefficient dependent upon the acceptable damage.

Damage (%)	K_D
0-5	4.0
5-10	4.9
10-15	6.6
15-20	8.0
20-30	10.0
30-40	12.2
40-50	15.0

Its porosity in a layer, n, is about 37% and it has a layer coefficient, K_Δ, of between 1.00 and 1.15.

Reference: *Shore Protection Manual*

h. Quarry Stone - Smooth

This is also stone obtained by blasting within a quarry, but more regularly shaped and smoother than the previous sort. Since its smoothness reduces its effective friction between armor elements, it tends to have lower damage coefficients than other stone:

Damage (%)	K_D
0-5	2.4
5-10	3.0
10-15	3.6
15-20	4.1
20-30	5.1
30-40	6.7
40-50	8.7

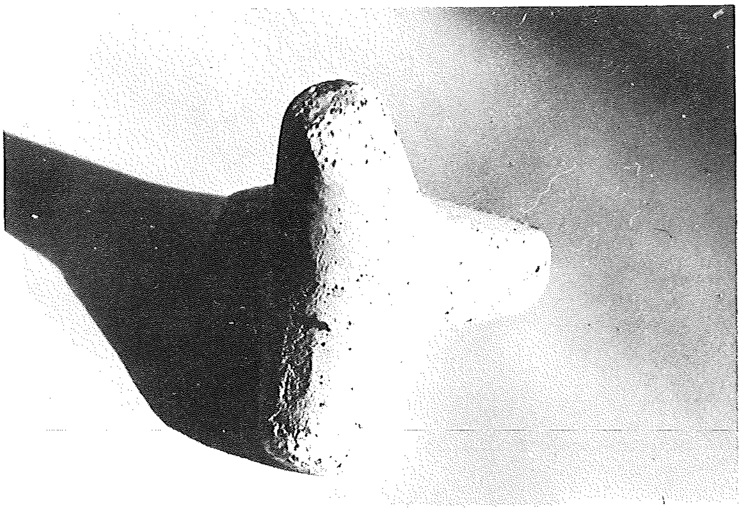
Smooth stone has a porosity of about 38% and a layer coefficient of 1.02.

Reference: *Shore Protection Manual*

i. Tetrapod and Quadripod

Both tetrapods and quadripods are plain concrete armor units consisting of four arms projecting from a central hub. The angular spacing between all arms of a tetrapod is the same; Three of the four arms of a quadripod extend horizontally while the fourth arm extends vertically. The tetrapod was developed by SOGREAH in France in 1950; the quadripod by the U.S. Corps of Engineers in 1959. These units are listed here together because they have identical design properties. Figure 6.6 shows a photo of a tetrapod.

Figure 6.6
TETRAPOD



The damage coefficient values vary with the allowable damage:

Damage (%)	K_D*
0-5	8.3
5-10	10.8
10-15	13.4
15-20	15.9
20-30	19.2
30-40	23.4
40-50	27.8

* The values listed are given by Hudson (1974); Paape and Walther (1962) report much lower values.

Here, also, because of a high K_D value, a monolithic crest construction is usually required to guarantee that the units do not slide up the breakwater slope.

Tetrapod armor layers have a porosity, n, of 50% and a layer coefficient, K_Δ, of 1.04.

Reference: Danel, Chapus, and Dhaille (1960)

j. Tribar

A tribar is a plain concrete unit consisting of three vertical cylindrical bars connected to a central hub. It was developed in the United States in 1958. Unlike the previous armor units, tribars are sometimes arranged in a single layer with the axes of the three cylinders perpendicular to the slope. Figure 6.7 shows such an armor unit.

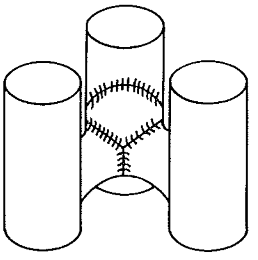


Figure 6.7
TRIBAR

In a single uniformly placed layer the tribar has a damage coefficient of about 14. When it is randomly placed in a double layer then the following values have been found:

Damage (%)	K_D
0-5	10.4
5-10	14.2
10-15	19.4
15-20	26.2
20-30	35.2
30-40	41.8
40-50	45.9

A monolithic crest construction is required to prevent the units from sliding up the breakwater face, especially when a single uniform layer is used.

A single uniform layer of tribars has a porosity of 47% and a layer coefficient of 1.13. The high porosity has implications for the secondary armor layer which must be very effective at containing the lower layers. See chapter 7 section 4.

Reference: Hudson (1974)

6.5. Armor Selection

As one may conclude from the variety of armor unit shapes available, no single type of armor unit is universally acceptable. Quarry stone armor is usually cheapest per ton but a larger volume is needed than when concrete units are used. Why? - because the lower K_D value requires flatter slopes to achieve the same stability. See chapter 7. On the other hand, a concrete plant is not needed when quarry stone is used.

If, on the other hand, artificial armor units are selected, then often one having a relatively high K_D value such as tetrapods or dolosse can prove most economical since the breakwater cross section can be made much smaller and/or lighter units can be used. The monolithic crest construction can even save total material cost by allowing - sometimes - a lower crest and lighter lee side armor than would otherwise be possible.

In the following chapter, where computations of necessary armor unit weights are presented, some of these items come up again.

6.6. Methods to Increase Stability

It is conceivable that armor layers having even higher effective damage coefficient values can be economical. What are the methods available to increase the K_D value of armor units?

One technique used on the breakwater extension at IJmuiden was to add asphalt to the stone armor layer. This served as a binder causing the armor layer to function as a unit and was, therefore, more resistant to wave attack than the individual stones. Unfortunately, the asphalt was also sufficient to form a water-tight covering such as is common on dikes. This required that the armor layer resist the resulting hydrostatic uplift forces. Further, the reduced porosity increased the wave run-up the slopes. These last two problems are, of course, detrimental to a design.

A proposed alternative is to use smaller quantities of asphalt placed here and there on the armor layer surface to tie individual armor units together into larger units *but not to form a closed layer.* The hope is expressed by proponents of this that sufficient prorosity will be maintained to prevent hydrostatic uplift pressures and to still absorb the wave energy.

Development of these concepts is proceeding slowly, partially because of the difficulty of scaling the elasto-plastic properties of asphalt in a model.

7. ARMOR COMPUTATIONS

L.E. van Loo
W.W. Massie

7.1. History

Until less than fifty years ago, rubble mound breakwaters were designed purely based upon experience, usually in prototype. Castro (1933) seems to have published the first modern work on this subject. Initial attempts to compute necessary armor unit sizes were based upon theoretical considerations of the equilibrium of a single armor unit on a slope. One need only to visualize the complex flow patterns in a breaking wave rushing up a breakwater slope to conclude that a purely theoretical approach is impossible. The theoretical background of the currently used formula is indicated in the following section.

7.2. Theoretical Background

Consider a single armor unit resting on a slope making an angle θ with respect to the horizontal as shown in figure 7.1.

The wave force, F, acting on the block can be approximated very crudely by considering the drag force of the water excerted on the block. This approach yields a force proportional to the unit weight of water, the projected area of the armor unit and the water surface slope. When we further let the surface slope be proportional to the wave height (This is reasonable since the wave length is determined by the wave period and water depth only.) then in a mathematical form:

$$F \propto (\rho, g, H, d^2) \qquad (7.01)$$

where:

F is the drag force,
d is a characteristic dimension of the block,
g is the acceleration of gravity,
H is the wave height,
ρ is the mass density of water, and
α denotes "is proportional to".

Other assumptions about the force description can be made; all run into difficulties somewhere. Therefore, (7.01) will be transformed into an equation by introducing a proportionality constant, a:

$$F = a \rho g H d^2 \qquad (7.02)$$

This force can act either up (uprush) or down (backwash) the slope as shown in figure 7.1.

Using figure 7.1, equilibrium of forces perpendicular to the slope yields:

$$N = W_{sub} \cos \theta \qquad (7.03)$$

where N is the normal force.

This normal force is related to the friction force, f, by the coefficient of static (Coulomb) friction, μ:

$$f = \mu N \tag{7.04}$$

Equilibrium parallel to the slope in figure 7.1a (uprush case) yields:

$$f = F - W_{sub} \sin \theta \tag{7.05a}$$

and for backwash (fig. 7.1b):

$$f = F + W_{sub} \sin \theta \tag{7.05b}$$

or, more generally:

$$f \geq F - W_{sub} \sin \theta \tag{7.06a}$$

and

$$f \geq F + W_{sub} \sin \theta \tag{7.06b}$$

respectively. These become:

$$W_{sub}(\mu \cos \theta + \sin \theta) \geq a \rho g H d^2 \tag{7.07a}$$

$$W_{sub}(\mu \cos \theta - \sin \theta) \geq a \rho g H d^2 \tag{7.07b}$$

The submerged weight of the armor unit can be expressed as its unit weight, ρ_a g, times its volume minus the weight of displaced water. It is assumed, further, that the volume of the armor unit may be expressed as some constant, b, times the cube of its characteristic dimension, d. In equation form:

$$W_{sub} = (\rho_a - \rho) g b d^3 \tag{7.08}$$

Substitution of (7.08) into (7.07) yields:

$$(\rho_a - \rho) g b d^3 (\mu \cos \theta + \sin \theta) \geq a \rho g H d^2 \tag{7.09a}$$

$$(\rho_a - \rho) g b d^3 (\mu \cos \theta - \sin \theta) \geq a \rho g H d^2 \tag{7.09b}$$

which reduce to:

$$\left(\frac{\rho_a - \rho}{\rho}\right) b d (\mu \cos \theta + \sin \theta) \geq a H \tag{7.10a}$$

$$\left(\frac{\rho_a - \rho}{\rho}\right) b d (\mu \cos \theta - \sin \theta) \geq a H \tag{7.10b}$$

for uprush and backwash respectively.

Analogous to the notation used in density currents (volume I chapter 22), let:

$$\Delta = \frac{\rho_a - \rho}{\rho} \tag{7.11}$$

Substituting (7.11) in (7.10), rearranging, and cubing both sides yields:

$$b^3 \ d^3 \geq \frac{a^3 \ H^3}{\Delta^3(\mu \cos \theta + \sin \theta)^3} \tag{7.12a}$$

$$b^3 \ d^3 \geq \frac{a^3 \ H^3}{\Delta^3(\mu \cos \theta - \sin \theta)^3} \tag{7.12b}$$

The weight, in air, of our armor unit is:

$$W = \rho_a \ g \ b \ d^3 \tag{7.13}$$

(7.13) in (7.12) results in:

$$W \geq \frac{\rho_a \ g \ \frac{a^3}{b^2} \ H^3}{\Delta^3(\mu \cos \theta + \sin \theta)^3} \tag{7.14a}$$

for uprush, and

$$W \geq \frac{\rho_a \ g \ \frac{a^3}{b^2} \ H^2}{\Delta^3(\mu \cos \theta - \sin \theta)^3} \tag{7.14b}$$

for backwash.

This is effectively the formula derived by Iribarren (1938).

A primary disadvantage of equation 7.14 is its abundance of emperical coefficients; a, b, μ, and ρ_a all must be determined for a given armor unit type. This has led to many emperical alternative proposals to replace Iribarren's formula with a simpler one.

While these alternative formulations have even less of a theoretical background, they often prove to be more handy in practice. A summary of these formulas is presented in a *Report of the International Commission for the Study of Wave Effects* of the PIANC (1976). It would serve no purpose to discuss all of these formulas here individually. Instead, the shaded area in figure 7.2 shows the range of results obtained using the various available formulas. Angular stone armor units having a given density and exposed to a constant wave height were assumed.

One of the more convenient alternatives to equation 7.14 is developed in the next section.

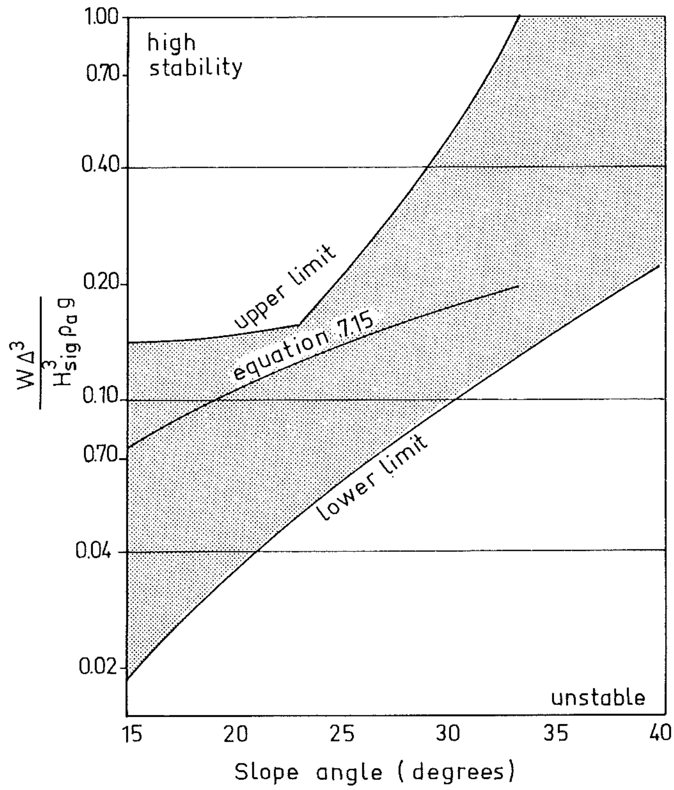


Figure 7.2
LIMITS OF ARMOR EQUATIONS

7.3. The Hudson Formula

Hudson (1953) developed an emperical formula for the weight of armor units based upon his analysis of model data obtained at the U.S. Army Corps of Engineers Waterways Experiment Station.
As an equation it is:

$$W = \frac{\rho_a\ g\ H^3}{K_D\ \Delta^3\ \cot(\theta)} \tag{7.15}$$

where:

g is the acceleration of gravity,
H is the wave height,
K_D is the damage coefficient,
W is the weight of the armor unit,
Δ is the relative density of the armor unit,
$\Delta = \frac{\rho_a - \rho}{\rho}$
θ is the slope of the breakwater,
ρ_a is the mass density of the armor unit, and
ρ is the mass density of (sea) water.

Equation 7.15, often called the Hudson Equation, has been used and verified in large and small scale models as well as prototypes during the years since 1953. Additionally, it avoids the separate equations for uprush and backwash proposed by Iribarren. Even so, it has some significant and important limitations:

a. It is valid only for slopes such that $\cot(\theta)$ is not less than 1.5 ($\theta < 33.7^{0}$).
b. It was developed for the front of a breakwater subject to non-breaking waves. This implies that the depth at the toe of the breakwater, h_t, is sufficiently great that the oncoming waves are not broken or unstable. When this is not the case (i.e. $\frac{H}{h_t} > 0.6$)* then this can be accounted for by lowering the value of K_D** - see the *Shore Protection Manual.*
c. It is valid only for the front slope of a breakwater. Since attack of the crest or inner slope of the breakwater by overtopping waves is not considered, then it is implicitly assumed that the crest elevation is greater than the run-up.
d. The wave (storm)conditions are characterized by a single parameter, H. While the effect of breaking of the waves has been considered above (item b), the effects of a storm's duration is not considered at all. Font (1968) and Nijboer (1972) have investigated this aspect, however. The latter author found that the damage was reasonably independent of the storm duration except when the design wave height was exceeded by more than 30 percent. These model tests were conducted using regular waves and stone armor units.

In contrast to Iribarren's formula, the properties of the armor unit are described by only two parameters, ρ_a and K_D. Values for K_D for many types of armor units were given in the previous chapter.

Generally, the characteristic wave height chosen for a rubble mound breakwater design is the significant wave height, H_{sig}. Hudson's original tests were conducted with regular waves. Nijboer (1972) points out the danger of replacing a monochromatic wave height with a significant wave height from a spectrum. He found in a model study of stone armor that the damage caused by a spectrum of waves characterized by H_{sig} was greater than that caused by monochromatic waves of the same height. This effect became more pronounced as the spectrum width increased.***

The fact that the characteristic wave for the Hudson Formula is the significant wave, H_{sig}, has a simplifying consequence for the optimum design of a rubble mound breakwater. The procedure for combining the long-term and Rayleigh wave height distributions (volume I, chapter 11) can be skipped. Details of what must be done for the current problem are given in chapter 11 of this volume.

* This value is more conservative than that given in the *Shore Protection Manual.*

** K_D values for breaking waves are about 87% of the corresponding values for non-breaking waves. Ahrens (1970) has studied this further.

*** This could logically lead to the choice of a different (higher) characteristic wave height for use in equation 7.15.

The Hudson Formula was developed for use on the outer layer of the main (trunk) portion of a breakwater. Further, as already mentioned, it applies only to the front slope. While the formula is very helpful even with these restrictions it can sometimes be applied to other cases as well; this is discussed in the following section.

7.4. Special Applications

Breakwater ends

The convex shape of the end of a breakwater can be expected to increase the exposure of the armor units to wave attack. In addition, the convexity can reduce the degree of interlocking between adjacent armor units. Both effects can be incorporated in the Hudson Formula, equation 7.15, by reducing the value of the damage coefficient, K_D, appropriately. This reduction amounts to between ten and forty percent depending upon the type of armor unit. The reduction is usually greatest for armor units having the higher K_D values (most interlocking). The *Shore Protection Manual* tabulates K_D values for ends of breakwaters (structure head). Often the lower K_D value is compensated by selecting somewhat flatter slopes at the end so that the same armor size may be used.

Toe

The Hudson Formula can be applied directly to the design of the toe of a breakwater exposed to breaking waves. This is discussed in more detail in chapter 9.

Secondary armor

A breakwater must be stable during construction as well as after its completion. Thus, it is necessary that the inner layers directly under the primary armor (secondary armor) be dimensioned to withstand the waves that can be reasonably expected during the construction period. The Hudson Formula may be applied directly to this problem in the same way that it is used for the primary armor layer. Because of the limited exposure time, however, a somewhat less severe storm can be used. Usually, this secondary layer will be made from stone having a weight of about 1/10 of that of the primary armor.

When especially porous armor unit placement is used in a single layer we must be especially aware of the containment function of the secondary armor. This extra function is most apparent when cobs or tribars are used for the outer armor. See chapter 6.

Angular wave attack

As we have seen in volumes I and II, the angle of wave approach is very important to the stability of a beach. For a breakwater, however, the angle of wave attack *is not* important for the stability of the armor. Even waves propagating along orthogonals parallel to the breakwater axis have been observed to damage the armor layer. The reason for this has not yet been sufficiently investigated, but may be that the weight of the armor unit no longer contributes directly to its stability when equilibrium along a slope contour line is being considered - see figure 7.3 and compare to figure 7.1.

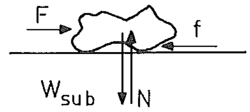


Figure 7.3
EQUILIBRIUM ALONG CONTOUR

Inner slope

The Hudson formula may be used to investigate the stability of the inner slope of a breakwater subject to *direct attack* from waves on the *lee side* of the structure. These waves may be generated within the harbor by winds or passing ships or may enter the harbor through the entrance or by overtopping *another portion* of the breakwater.

The Hudson Formula is *inadequate*, however, to predict armor weights necessary to withstand the attack from waves spilling over the breakwater crest from the opposite side of the structure.*
Detailed model studies are required to investigate the behavior of breakwaters too low to prevent overtopping.

Crest

The Hudson Formula is also inadequate to dimension armor units for the crest of a breakwater overtopped by waves; Once again, detailed model tests are required.

Armor units having higher damage coefficient values need additional support at the top of their slope. Monolithic crest structures are then required. Even though these are usually more expensive to construct, in themselves they can save enough total material to be economical.

7.5. Sensitivity of Hudson Formula

Not all of the parameters in the Hudson Formula, equation 7.15, can be exactly determined for a given design problem. Therefore, it can be instructive to examine the influence of small changes of the various parameter values upon the resulting weight of the armor unit. In the following discussion the influence of a given change in a parameter is reflected in a change in the armor weight, W. All other parameters are assumed to be constant. For convenience, equation 7.15 is repeated here:

$$W = \frac{\rho_a \; g \; H^3}{K_D \; \Delta^3 \; \cot(\theta)} \qquad (7.15)$$

When the wave height increases by 10%, the required armor weight increases by 33%. A 10% decrease in wave height decreases the blockweight by 27%. Thus, the formula magnifies small errors in wave height.

Increasing the density of the armor unit by 10% decreases the armor weight needed by about 30% for normal values of armor and water densities.** Decreasing the density by 10% increases the necessary weight by 55%! What is the effect of substituting Swedish Granite (ρ_a = 2650 kg/m^3) for Basalt (ρ_a = 2900 kg/m^3) for armor units? The ratio of the armor weights is:

---

* This is the reason that the crest elevation was earlier assumed to exceed the run-up.

** ρ = 1025 kg/m^3 and ρ_a = 2600 kg/m^3.

$$\frac{W_{granite}}{W_{basalt}} = \frac{(2900 - 1025)^3}{2900} \times \frac{2650}{(2650 - 1025)^3} \qquad (7.16)$$

$$= 1.40 \qquad (7.17)$$

The granite blocks must be 40% heavier than the basalt stone to achieve the same stability.

Increasing the K_D value by 10% decreases the necessary armor weight by 9%. This change in the damage coefficient could be accomplished by selecting a different type of armor or possibly by accepting a greater damage to the structure during exposure to a given storm; see chapter 6.

7.6. Choice of Armor Units

The sensitivity of the Hudson Formula to wave height changes has been demonstrated in the previous section. The wave height chosen for design purposes is seldom accurately related to a frequency of occurrence. Equivalently, the significant wave height associated with a given frequency of occurrence, such as once per ten years, is seldom accurately determined. Thus, it seems appropriate to select an armor unit which forms a layer most resistant to waves (storms) which may exceed the design condition.

Normally, the K_D values used in equation 7.15 are associated with only slight damage to the armor layer - perhaps 1% of the units effectively removed. On the other hand, if we wish to accept a higher damage to our design we can account for this by increasing the damage coefficient values in the Hudson Formula. This is the background of the tables of K_D versus percent damage given for some armor units in chapter 6. How can this information be used to predict damage when the design wave heights are exceeded?

Once we have made a design and selected an armor unit, then the only variables left in the Hudson Equation are K_D and H. Equation 7.15 can be transformed to show the relationship:

$$K_D = \frac{\rho_a \; g}{W \; \Delta^3 \; \cot(\theta)} H^3 \qquad (7.18)$$

yielding:

$$H^* = \sqrt[3]{\frac{K_D^{*}}{K_D'}} . H \qquad (7.19)$$

where:

H^* is the unknown wave height causing a chosen experimentally determined damage,

H is the wave height for no damage,

K_D^* is the damage coefficient for the damage percentage caused by H^*, and

K_D' is the damage coefficient for no damage.

Thus,

$$\frac{H^*}{H} = \left[\frac{K_D^*}{K_D}\right]^{1/3} \qquad (7.20)$$

We can use equation 7.20 to compare tribars to tetrapods, for example. Using data from chapter 6, we can make the computation shown in table 7.1 in which the wave height ratios are computed using equation 7.20. The results are also shown in a graph, figure 7.4; it appears that tribars are superior.

TABLE 7.1 COMPARISON OF ARMOR UNITS

Damage	Tetrapods K_D^*	Tetrapods $\frac{H^*}{H}$	Tribars K_D^*	Tribars $\frac{H^*}{H}$
(%)	(-)	(-)	(-)	(-)
0-5	8.3	1.00	10.4	1.00
5-10	10.8	1.09	14.2	1.11
10-15	13.4	1.17	19.4	1.23
15-20	15.9	1.24	26.2	1.36
20-30	19.2	1.32	35.2	1.50
30-40	23.4	1.41	41.8	1.59
40-50	27.8	1.50	45.9	1.64

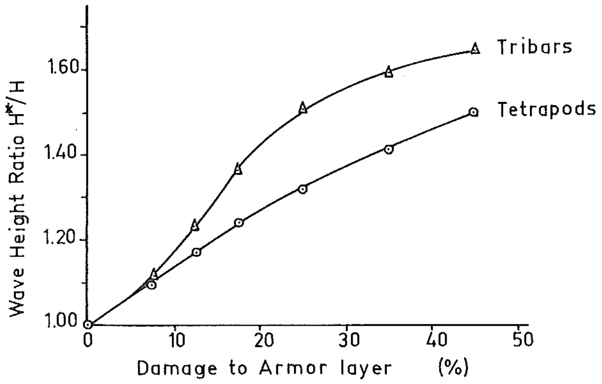


Figure 7.4
COMPARISON OF ARMOR UNITS

One must be careful about drawing conclusions based solely upon computations of the sort just carried out. Nothing is indicated about the absolute block weights required or about differences in capital costs of various armor units.

Data necessary for determining figure 7.4 are available only for a few types of armor units. For other armor, detailed model tests are needed too determine the relationship shown in the figure. Except for very small projects it is strongly recommended that model tests always be used to verify the given coefficients for the specific project under consideration.

Armor layer design considerations unrelated to the Hudson Formula are considered in the following two sections of this chapter.

7.7. Layer Extent and Thickness

Since the primary armor layer can be more expensive to construct than other portions of the breakwater, it is advantageous to limit the area covered by primary armor units as much as possible consistent with stability needs. Only a few rules of thumb exist to indicate the necessary extent of this armor layer. These should be confirmed by experiments if the project is at all extensive.

Normally the primary armor units are extended downward on the breakwater slope to an elevation of 1.5 H below the still water level. Whether an extreme storm flood water level and a severe storm must be chosen or a moderate storm with low water level depends upon which condition results in the lowest absolute elevation for the bottom of the primary armor.

The primary armor extends upward along the front slope at least to the crest elevation. If the crest elevation has been chosen such that no or very little overtopping can be expected, then there is no great reason to extend the armor over the crest and down the inner slope. It is uneconomical to construct a rubble mound breakwater higher than needed to prevent overtopping*. The only need for a still higher construction - visibility for shipping - can be realized more cheaply using daybeacons or even lights.

The armor on the inner slope of a non-overtopping breakwater can be dimensioned in the conventional way using the wave climate on the lee side as design input.

If, on the other hand, moderate to severe overtopping is expected, then the primary armor must extend across the crest and down the inner slope to an elevation slightly below the lowest still water level. Severest damage will probably occur at the top of the inner slope where the armor units are least protected from the water spilling over the crest.

When a monolithic crest construction is used to provide additional support to special armor units such as tetrapods or tribars, then overtopping is not usually allowed. The possible extra height needed to prevent this overtopping is compensated by a steeper slope and simpler construction on the lee side.

The layer thickness, t, can be computed from the following semi-emperical formula:

$$t = m\ K_\Delta \left(\frac{W}{\rho_a\ g}\right)^{1/3} \qquad (7.21)$$

where:

m is the number of layers of armor units - usually 2, sometimes 1 or 3,

K_Δ is an emperical layer coefficient listed for each type of unit in chapter 6, and

t is the layer thickness.

The number of armor units needed per unit of primary armor layer surface area can be estimated from:

$$C = m\ K_\Delta (1 - n) \left(\frac{\rho_a\ g}{W}\right)^{2/3} \qquad (7.22)$$

where:

C is the number of armor units per unit area of armor layer, and

n is the armor unit layer porosity expressed as a decimal and listed in chapter 6 for each type of armor unit.

* It is often uneconomical to build one this high!

7.8. Crest Width

The crest width of a rubble mound breakwater is determined by the degree of wave overtopping and construction requirements. When there is no overtopping, the waves no longer influence the choice of crest width. When overtopping is expected and primary armor units cover the crest, then the crest should be at least wide enough to allow three armor units to be placed across it. Thus:

$$B = m' \; K_{\Delta}\left(\frac{W}{\rho_a \; g}\right)^{1/3} \qquad (7.23)$$

where:

B is the crest width, and

m' is the number of armor units across the crest - usually at least 3.

When a breakwater is to be constructed or maintained by construction equipment working from the crest, then the crest width will possibly be dictated by the space needed for efficient use of the chosen equipment. This will be discussed again in chapter 10.

7.9. Review

The background and use of the currently popular semi-emperical relations for rubble mound breakwater armor layer computations have just been presented. Because of their emperical nature, the equations must be used with caution. Extrapolation, for example, is incorrect and irresponsible.

In practice the formulas presented here and the coefficients listed in chapter 6 should, at best, be considered to be guidelines. Extensive model testing is required for all except the most modest projects.

The requirements for and design of the deeper layers of a breakwater are discussed in the following two chapters.

8. THE CORE

J.F. Agema
E.W. Bijker

8.1. Function

The primary function of the core material of a rubble mound breakwater is to support the covering armor layers in their proper position. A secondary function, stipulated when the breakwater must be sand-tight, is that the core be reasonably impermeable. It need, in fact, only be impermeable to sand; water may continue to flow through it. In practice, however, a designer should not plan on constructing a sandtight dam which will allow much water to pass through it - at least not for long. Marine growth within a breakwater core can reduce its permeability significantly within a few years.

Occasionally, it is required that a breakwater be watertight. This is often true, for example, when a breakwater must serve to guide the cooling water for a thermal power station. In such applications direct transfer of discharged water to the intake water can be detrimental to the thermodynamic efficiency of the plant. Special impermeable core constructions must then be provided. These types of cores are described in the literature and courses on dikes.

The choice of a core material will have an influence on the armor units. As the permeability of the core decreases, the portion of the wave energy expended upon the armor layers increases, resulting in a higher effective attack on these units. Quantitative information can be obtained only from model experiments.

8.2. Materials

Since most any non-floating material will be sufficient to support the cover layer, the choice of a core material is usually dictated by constructional or economic requirements.

When quarry stone is used for armor, then the finer tailings - scrap material from the quarry, often called quarry run - can be advantageously used in the core. This material, because of its well distributed range of grain sizes, (usually) forms a rather impervious core.

If this sort of well graded material is not available, other core constructions can be conceived. Small (a few hundred kilogram) concrete blocks have been used in some cases. Rubble from razed masonry buildings has even been used occasionally.

If an impermeable core is required, but the available core materials will remain too permeable for sand and water, asphalt or grout can be injected into the core to decrease its permeability. Of these materials, asphalt is probably to be preferred since it maintains a degree of plasticity during settlement of the structure.

The core of a breakwater or even that of a seawall is fundamentally different from that of a dike. First, the core is usually the only impermeable part of a breakwater while a dike usually has several impermeable layers. Second, at best a breakwater need only be absolutely impervious to sand; there is usually no need to prevent water seepage - something which can be disasterous to a dike.

8.3. Construction Methods

When reasonably fine material can be used for a core, much of this core can often be placed simply by dumping the material from bottom dump hopper barges. This sort of construction technique is less advantageous when coarser material must be used.

One must be cautious in design to provide adequate protection for the core material during construction. This will be highlighted as part of chapter 10.

9. FILTER AND TOE CONSTRUCTIONS

E.W. Bijker

9.1. Description and Functions

Filter layers are the undermost layers of a rubble mound breakwater which serve to prevent excessive settlement of the structure. This prevention is accomplished by hindering the erosion of bottom material by water moving through the pores of the breakwater. Thus, filter constructions are most necessary when the natural bottom consists of easily eroded material such as fine sand.

Toe constructions form an extension of the filter beyond the limits of the normal breakwater cross section and serve to support the lower edge of the armor layer. In addition, these toe constructions can act as a bottom revetment along the breakwater to prevent scour immediately adjacent to the toe from jeopardizing the foundation integrity. These, too, are most necessary when the bottom material can be easily eroded.

9.2. The Physical Phenomona Involved

The erosion of bottom material under a breakwater is caused by local currents resulting from wave pressure fluctuations. This is shown in schematic form in figure 9.1.

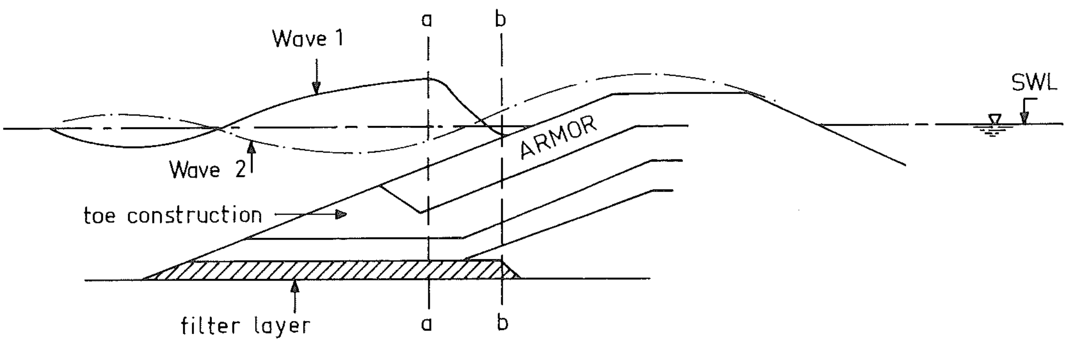


Figure 9.1 REPRESENTATION OF PRESSURES WITHIN BREAKWATER

When there are no waves, the pressures at the bottom at sections a and b are equal, there is no flow and we have no problems. However, at an instant when wave profile 1 is present, a pressure gradient results in a flow from a to b through the breakwater pores. A short time later - wave profile 2 - the pressure gradient and flow direction are reversed. This alternating flow can cause local scour of bed material resulting in settlement of the breakwater.

The short wave theory presented in chapter 5 of volume I is inadequate to predict the pressure distribution within the breakwater. An extra pressure damping is introduced by the material of the breakwater.

This damping is a function of the breakwater material grain size and was investigated by de Lara (1955) and by Le Méhauté (1957-58).

The velocities resulting from the pressure fluctuations are even harder to determine. Physical models run into problems since the porosity of the breakwater material does not follow simple scaling laws. Veltman-Geense (1974) has investigated this. Even though *average* flow velocities near the bottom of a breakwater may be small, the irregularity of the flow channel form can lead to locally high velocities which result in scour and thus settlement. Obviously, this settlement does not continue indefinitely. As the breakwater material penetrates deeper the damping influence become greater; eventually an equilibrium is reached. Unfortunately if no filter were built, settlements of several meters could be possible, resulting in much waste of material. Therefore, it is usually more economical to built a filter under a breakwater located on an erodible bed. The purpose of this filter will be to prevent the occurrence of flow velocities high enough to cause erosion of fine bed material.

An additional purpose of the toe construction is to prevent the armor units from sliding down the face of the breakwater. This is also shown in figure 9.1.

9.3. Design Criteria for Filters

An adequate filter construction on a sand bed must satisfy two criteria:

a. it must prevent the erosion of material from under the breakwater caused by horizontal currents, and

b. it must prevent the formation of a quicksand condition caused by an abrupt vertical flow (pressure gradient) in the sand.

Most filter constructions which satisfy one of the above conditions will satisfy the other as well. Model tests of filters run into scale difficulties; often full scale tests are conducted for large or important breakwater projects.

9.4. Design Criteria for Toes

In addition to the criteria already listed for filters in the previous section, toe constructions must also remain stable under the action of waves, currents and the lateral load from armor units on the slope. In addition, extended revetment type toe constructions must be flexible enough to follow changes in the bottom profile which can result from local scour near the revetment edge.

The currents which cause erosion in this area may result from wave pressure fluctuations, but may also be caused by tides or a longshore current.

9.5. Filter Layer Constructions

A conventional filter layer is usually built up of a few layers of progressively coarser gravel. The construction work must be carried out with reasonable care, since a gap in a layer of the filter can result in eventual failure. A certain degree of overdimensioning is usually justified.

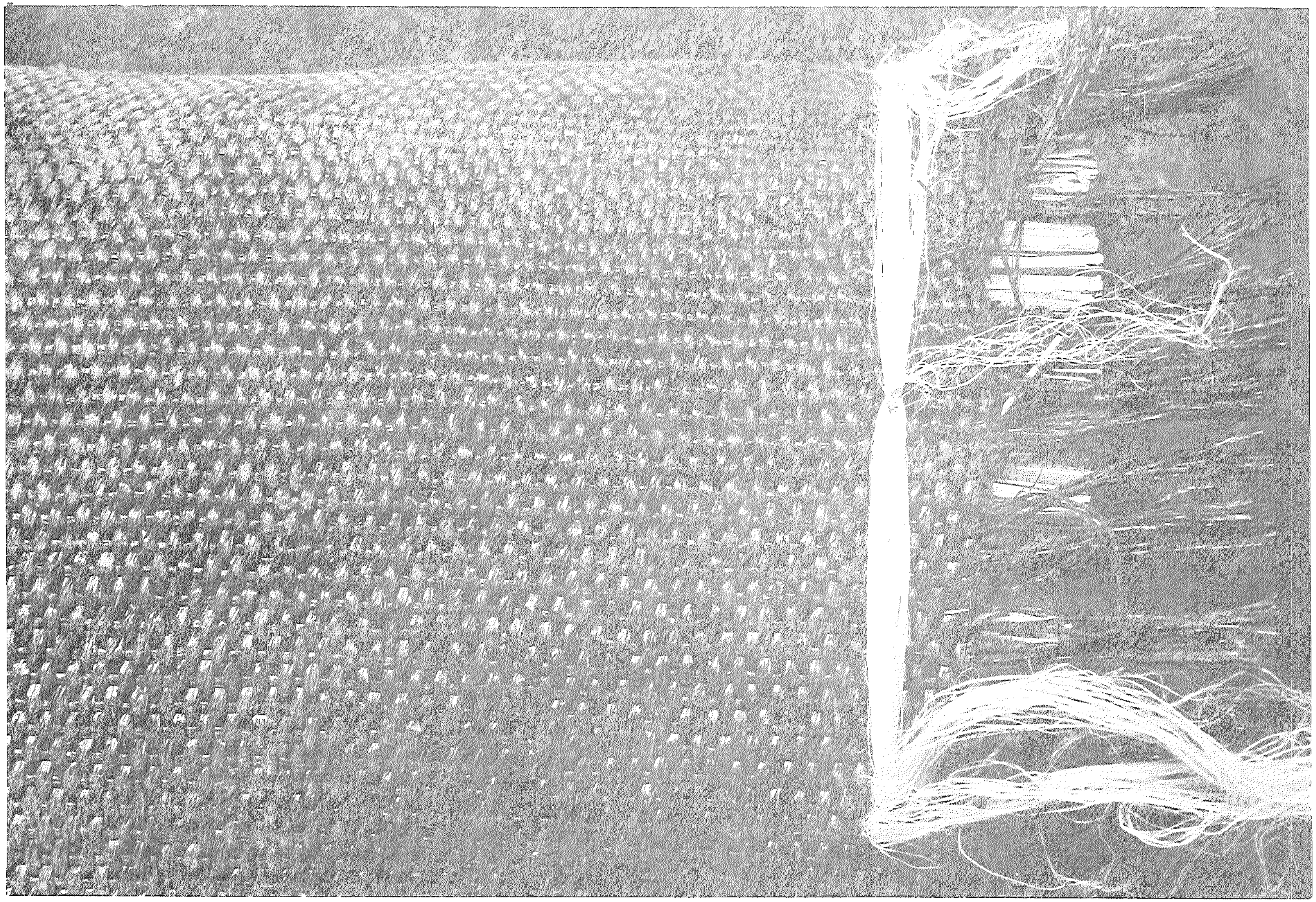


Figure 9.2

WOVEN FABRIC MATTRESS

Thus, a total filter construction is usually at least 1 to 1.5 m thick. Details of construction techniques will be given in chapter 10.

If it is necessary to construct a breakwater in an area which is exposed to severe waves or currents, it is possible that a gravel filter layer will be swept away nearly as fast as it is laid. In such a case fascine mattresses, specially fabricated so that they are more sand tight than usual, can be used. This sand-tightness can be achieved by incorporating a layer of heavy woven fabric within the mattress. De Jong and Peerlkamp (1973) summarize the development of filter constructions well. See fig. 9.2, also. Such a special fascine mattress can be sunk into place and held there with stone ballast. Such a filter is usually thinner than a more conventional gravel filter.

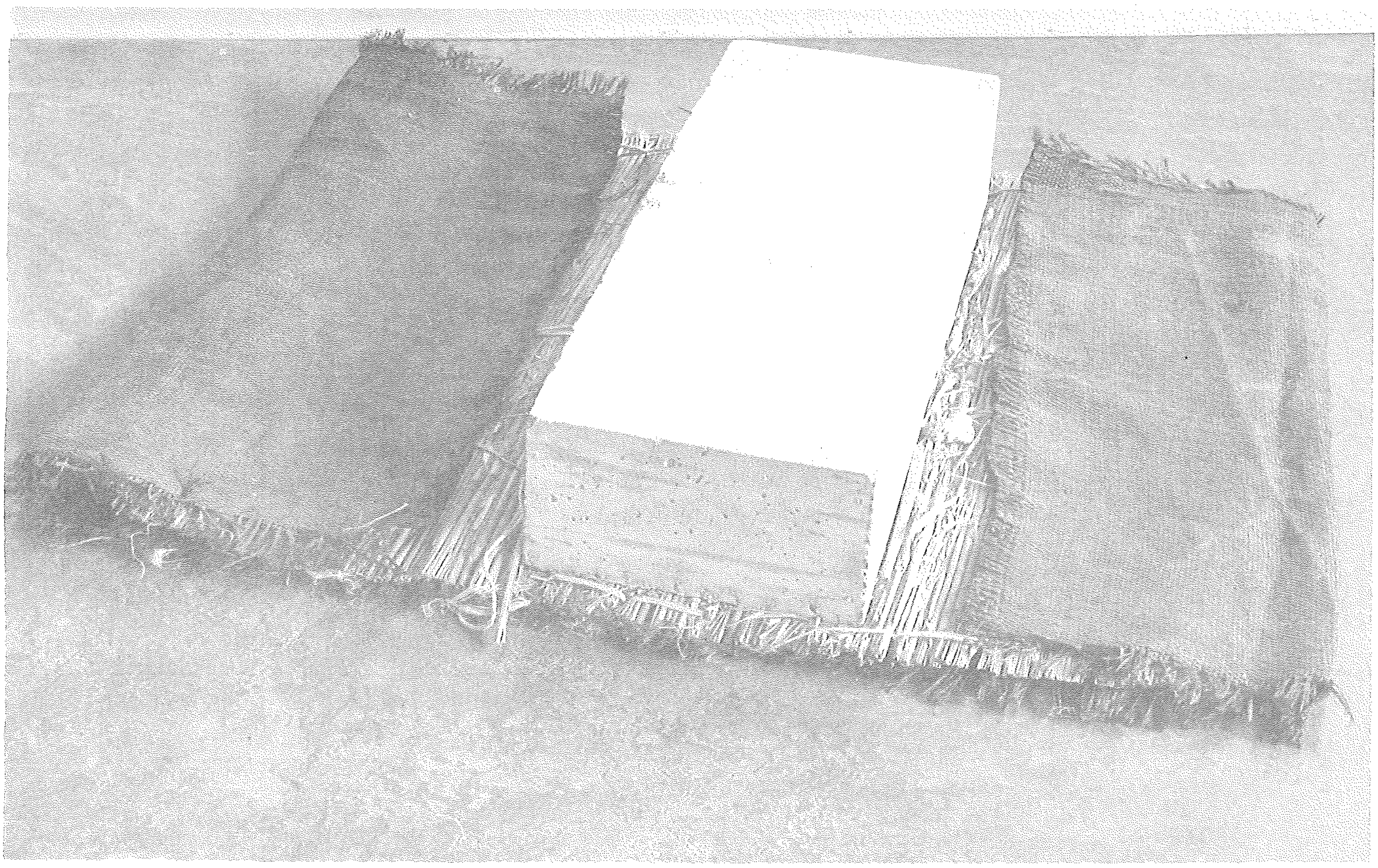


Figure 9.3
WOVEN FABRIC MATTRESS WITH CONCRETE BLOCK

Another possibility is to attach concrete ballast blocks to a woven fabric. A single layer of reeds sometimes separates the blocks from the fabric in order to prevent damage from friction. Such a filter is still thinner than the above types, and can be placed by unwinding it from a floating spool upon which the ballasted mat has been rolled. Figure 9.3 shows a photo of such a mat.

Still another commonly used filter or bottom protection consists of a layer of asphalt placed under water. Various contractors have developed what appear to be very successful techniques for accomplishing a uniform underwater placement of asphalt.

9.6. Toe Constructions

Most toe constructions consist of light armor units used to support the lower portion of the primary armor layer and protect the filter (revetment) from direct wave attack. Toe constructions are most critical when a breakwater in shallow water is subjected to breaking wave attack. The problem and its possible solutions are illustrated via the following example:

Given data

A rubble mound breakwater (toe) is to be designed for a water depth of 7.5 m. Maximum wave heights are limited by the water depth. The face slope is 1 : 1.5. Rough Quarry stone is to be used.

Solution

The design wave for this structure will be determined by the breaking index $\gamma = 0.6$.* Thus:

$$H_{sig\ d} = (0.6)(7.5) = 4.5 \text{ m} \tag{9.01}$$

where

$H_{sig\ d}$ denotes the design significant wave height.

Using the rule of thumb presented in the previous chapter, the primary armor should extend to an elevation of about

$$(1.5)\ H = (1.5)(4.5) = 6.75 \text{ m} \tag{9.02}$$

below the still water level.

From the *Shore Protection Manual* and chapter 6, we find that for granite stone, $\rho_a = 2650$ kg/m^3 and $K_D = 3.5$ for breaking waves.

Substituting this into equation 7.15:

$$W = \frac{\rho_a\ g\ H^3}{K_D\ \Delta^3\ \cot(\theta)} \qquad (7.15) \qquad (9.03)$$

yields:

$$W = \frac{(2650)(9.81)(4.5)^3}{(3.5)(\frac{2650 - 1025}{1025})^3(1.5)} \tag{9.04}$$

$$= 113 \times 10^3 \text{ N} \tag{9.05}$$

* See section 7.5 of volume I and chapter 7 of this volume.

The thickness of this layer follows from equation 7.21.

$$t = m K_{\Delta} \left(\frac{W}{\rho_a g}\right)^{1/3} \qquad (7.21) \quad (9.06)$$

which yields:

$$t = (2)(1.15)\left(\frac{113 \times 10^3}{(2650)(9.81)}\right)^{1/3} \qquad (9.07)$$

$$= 3.8 \text{ m} \qquad (9.08)$$

when a double layer is used.

The lower inner corner of this layer enters the sea bed. This presents obvious problems of support for these stones. However, one solution is to excavate the bed and construct the toe in a pit.

Under this armor layer we need a layer of lighter stone having a mass ranging from 0.5 to 1.5 tons; such a layer will be

$$(2)(1.15)\left(\frac{1.00}{2.65}\right)^{1/3} \qquad (9.09)$$

$$= 1.7 \text{ m} \qquad (9.10)$$

thick.

A filter layer 1.5 m thick should be constructed under this.

When all of this is put together, a pit 6.5 m deep, shown in figure 9.4, will be required.

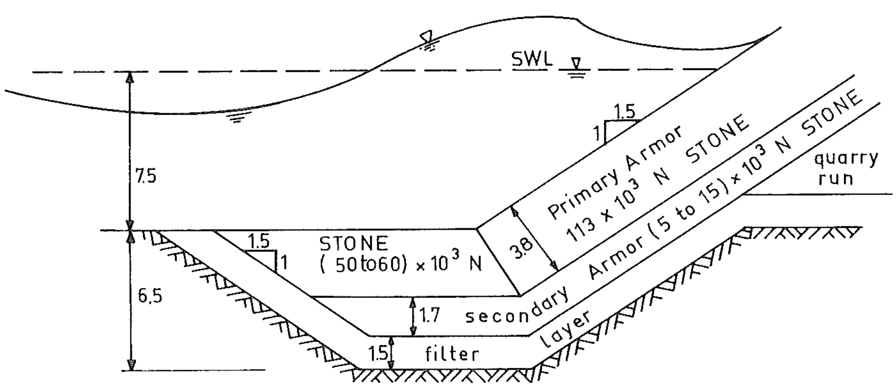


Figure 9.4
CONVENTIONAL EXCAVATED TOE CONSTRUCTION
Scale: 1:200

Since this excavation work will be very expensive, it can be advantageous to reduce it. One method, shown in figure 9.5, is to reduce the thickness of the primary armor layer near the toe. The toe supporting stones of 5-6 ton mass are extended under this single armor unit layer as shown. The filter layer under the toe supporting stone has been increased in thickness to 2.0 m to compensate for the removal of the secon-

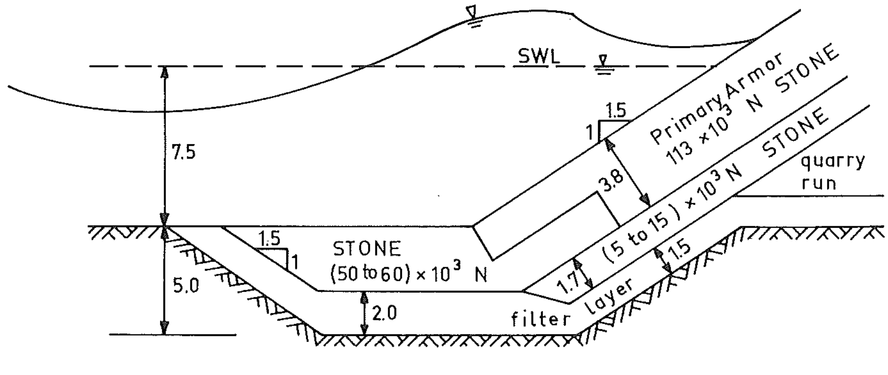


Figure 9.5
ALTERNATIVE TOE CONSTRUCTION
scale: 1:200

dary armor layer in that area. Even so, the depth of the excavation has only been reduced from 6.5 to 5.0 m, and this solution involving the thinner primary armor layer is difficult to construct under water.

Still another alternative uses a heavily supported toe constructed without excavation. This is shown in figure 9.6. A relatively large quantity of toe support stone is needed to give adequate support to the primary armor. Some loss of this stone from the toe support can be expected and tolerated.

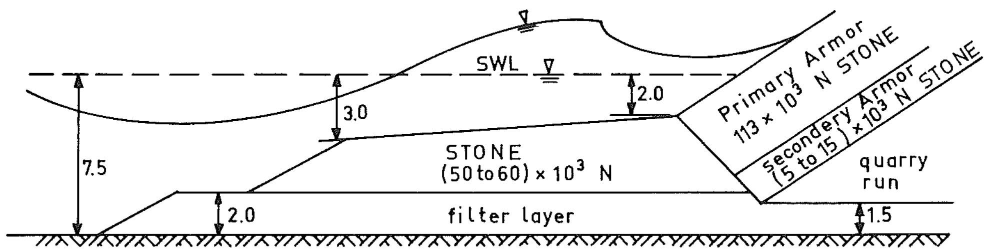


Figure 9.6
TOE CONSTRUCTION WITHOUT EXCAVATION
scale: 1:200

The maximum toe slope of this toe protection can be determined using the Hudson Formula. The sketched slope of 1:7.5 is somewhat flatter than that required.

As in the previous alternative, the filter layer under the toe has been thickened to 2.0 m to support the coarser stone.

9.7. Other Foundation Problems

Obviously a rubble mound breakwater subjects the subsoil layers to loadings. Settlement just as under any other structure can be expected, therefore, and predicted using classical soil mechanics techniques.

In addition, rubble mound structures usually settle within themselves. Wave action will cause some displacement of breakwater materials decreasing the porosity of the structure. This decrease is evidenced by a settlement of the crest relative to the lower part of the breakwater.

Since a rubble mound breakwater is a flexible construction, neither of these settlements is really detrimental to the structural integrity of the breakwater. However, the resulting crest lowering can have consequences for wave overtopping, and thus, damage to the inner slope or increased wave transmission.

Often the breaking waves near the toe of a breakwater can cause sufficiently high pore pressure fluctuations in sandy soils to generate a quicksand condition immediately in front of the toe. If there are currents this sand will be removed resulting in a scour hole. Even when currents do not exist, this sand will no longer contribute support to the toe construction or anything else, for that matter. It is therefore necessary to discount the presence of this sand when investigating the integrity of the foundation as a whole with regard to possible slip failures. Often times a critical slip circle passing through both the breakwater mass and the supporting soil will determine the horizontal extent of the toe construction and bottom revetment in front of the toe. Under extreme conditions such a slip circle analysis can even limit the maximum allowable slope of the breakwater face.

The analysis just mentioned should be carried out in addition to that required to investigate possible slip failures purely within the breakwater. As has already been pointed out in chapter 6, section 4, this is usually most important for artificial armor units which have relatively high damage coefficients such as tribars, tetrapods or akmons.

10. RUBBLE MOUND BREAKWATER CONSTRUCTION

J.F. Agema

10.1. Introduction

Sometimes a logical sequence of project execution is to first complete a design and then to worry about construction techniques. Such an approach to rubble mound breakwater design is irresponsible, however, since the construction method chosen can have a significant influence on the cost of construction. Therefore, we now consider how rubble mound breakwaters can be constructed and then, in the next chapter, combine all of the information presented into an optimum design.

The remainder of this chapter will be concerned with available construction methods and their relative merits.

10.2. Construction Methods

It is usually impractical to construct a breakwater working in a temporary dry building pit.* Even though construction "in the dry" is more precise and less expensive, the resulting savings do not, in general, balance the additional cost of a temporary cofferdam. Thus, at least a portion of a breakwater must be constructed under water.

What are the methods available to transport and place the large volumes of material required? Floating equipment can usually move large volumes of material most economically. Other methods include dumping of material from a temporary bridge or cableway or even from a road extending over the already completed portion of the breakwater. Another method, used occasionally, drops material from helicopters. Details of each of these methods are listed below.

Use of floating equipment

Direct placement of breakwater materials by dumping from barges can be especially economical when material is supplied by ship, and can be placed using these same ships. Types of ships for this work can include various types of bottom dump barges as well as side unloading barges. The bottom dump barges tend to deposit their cargoes quickly in a concentrated mass, while the side unloading barges discharge more gradually and are capable, therefore, of spreading a thin layer of material. Obviously, barges of these sorts can only construct a breakwater to an elevation over which they can still maneuver. In practice, this means that the maximum elevation is about 3 meters below the water level.**

Special barges can be built which place material at higher elevations on the breakwater by using an attached or separate floating crane. Design and fabrication of such specialized construction equipment is usually too expensive to be economical for small projects.

---

* There are cases in which an extreme tide range can be used to advantage to construct a major portion of a seawall or breakwater "in the dry".

** This water level may well take advantage of a large portion of the tidal range - it may be higher than mean sea level.

The use of floating equipment is handicapped by its dependence upon reasonable weather conditions for navigation - storms and poor visibility can halt operations. A second problem involves the positioning of the ships and their dumped cargoes. Sophisticated navigation systems are often needed.

Construction from fixed structures

Rubble mound breakwaters can be constructed by working from some form of fixed structure. This structure may be a temporary bridge supported on pillars which become burried in the breakwater. Materials and construction equipment are transported over this bridge. Such a bridge must, of course, be high enough to protect the construction equipment from the waves to be expected during construction.

An alternative to a bridge, needing fewer but larger foundations, is a cableway. Materials dropped from a cableway cannot be as accurately placed as those moved by cranes from lower structures. On the other hand, construction is the least hampered by the weather. Because of their long straight spans, cableways are only suitable for use on breakwaters which have long straight segments.

A special form of "bridge" from which to construct the breakwater can be the breakwater itself. Construction begins at the shore; material is supplied over the crest of the completed portion to construction equipment at the exposed end. This construction technique places special requirements on the breakwater itself; its crest must be high and wide enough to permit the efficient supply of equipment and materials in all weather conditions. This may require a higher and wider crest than would be needed otherwise. Even with a high and wide crest, construction speed is often limited by the capacity of the crane at the end of the breakwater.

This possible bottleneck to construction can be alleviated somewhat by placing cranes on jack-up platforms - see volume I, chapter 32 - beside the breakwater location. Materials are still supplied over the crest. A photo showing jack-up or self elevating platforms in use at IJmuiden is included in the *Shore Protection Manual* - volume II, page 6-92.

When armor units are used to protect the crest of the breakwater, they can provide too rough a surface for efficient transport of materials and equipment. Two solutions to the problem are possible: chinking of the crest armor with finer material, and delaying of the placing of the crest armor until the rest of the structure is completed. This second technique allows equipment to travel over the smoother but lower underlayer. Since chinking materials will be washed away in time, both methods suggested will result eventually in a rough surface which may make maintenance work more difficult.

An alternative design using a monolithic crest will eliminate these problems but is often expensive. On the other hand, such a massive crest can support special armor units which may possible be placed on a steeper slope or be of lighter weight. Either of these modifications (of slope* or weight) can mean that lighter construction equipment can be used.

Special methods

Use of helicopters to place breakwater materials has been attempted successfully on an experimental basis. A disadvantage of helicopters, their extreme dependence upon favorable weather conditions, is offset by their excellent maneuverability. Helicopters may prove to be very servicable in the future for maintenance work since they can easily place small loads of material at a variety of places on the breakwater.

Combinations of methods

Often the major portion of the deeper breakwater parts are constructed by dumping from barges. After this lower portion has been built up as high as conveniently possible in this way, the structure is completed by working over the crest of the structure as outlined above.

10.3. Specific Constructional Aspects

Constructional problems specific to particular portions of a rubble mound breakwater are discussed below.

Filters

The filter layers, when necessary, can form the most important part of the breakwater construction; the rest of the breakwater will not remain stable if its foundation is poor. Therefore, in contrast to what might be called popular belief, the construction of a filter should be done most carefully.

Except in very shallow water, gravel filters are normally constructed by dumping materials slowly from moving side dumping barges. Dumping rates and barge speeds should be chosen in such a way that each grain size of the filter is laid down in a series of sublayers. This gives a more uniform distribution of material over the resulting layer and hence, less chance of local imperfections which would eventually lead to failure.

Asphalt and nylon filters are single ply, normally. These must be constructed so accurately that work from anchored ships is required. The ships move by using cables to anchors placed outside the working area. Such filters are often covered with a layer of gravel, but this is intended primarily to protect them from direct impact forces from coarser material being dumped on top.

---

*How is slope related to crane size? The crane boom length necessary to reach the breakwater toe is shorter with steep slopes.

Core

The really rough work in a breakwater is the placement of the core material. If waves and currents did not disturb the operation, then the only problem would be that of achieving the desired slopes when dumping material under water - either from a barge or from a crane bucket. Of course, submerged portions of side slopes can be re-profiled working from a fixed point using a crane, but it can be more economical to avoid this if possible. Protection of the core from waves and currents during construction is one of the topics discussed in section 10.4.

Armor units

Primary armor units are almost exlusively placed by crane - either floating or fixed. Obviously, the crane used must be capable of placing an armor unit anywhere on the slope to be protected. The availability of cranes can influence the choice of armor units. Even when random or pell-mell placement of armor units is specified, accurate placement of individual armor units is required in order to guarantee a uniform covering. Sometimes, placement plans specifying exact locations for each armor unit are used even with so-called pell-mell armor placement. When specific placement patterns are required for stability - as with tribars, for example - extra care is called for; so much care, in fact, that this cannot be successfully accomplished under water.

When artificial armor units of several different sizes are required, time and confusion at the armor unit fabrication site can sometimes be reduced by modifying the density of the concrete used rather than by casting a new size of unit - see chapter 7. This techniques can result in considerable savings at the fabrication site, and can result in a lighter weight block than would otherwise be required which has, again, consequences for the crane selection.
Details of armor unit placement schemes are usually worked out in models. These may be the hydraulic models used to investigate durability or separate construction models may be built to determine exact cover layer properties such as porosity.

Crest

The crest of the breakwater must be broad and smooth enough to accomodate construction and material transport equipment if over-the-crest construction or maintenance is planned. The width needed during construction is sometimes more than that needed for maintenance. Since much more equipment is moving along the crest during construction a two lane roadway may prove economical, especially if the breakwater is long.

Massive monolithic crest constructions are often used with special armor units such as tribars and tetrapods. Such monolithic structures provide an excellent roadway, but are not without problems. Since a rubble mound breakwater is more or less designed to settle a bit, these monolithic crest elements must also be tolerant of this. This means in practice that relative displacements of the crest elements must be allowed at specific locations.

10.4. Special Construction Problems

Waves and currents during construction can attack a partially completed breakwater and cause a certain degree of damage. Unprotected core material is the most susceptible to damage. If the expected attack is only minor it can be most economical to simply accept a loss of core material due to erosion and thus, to place somewhat more core material than would be needed otherwise.

If wave and current influences are too severe, special measures must be taken to protect the core material during construction. This can be accomplished by first building up the secondary armor units and then filling in between the armor unit ridges with core material. This construction sequence is shown in figure 10.1.

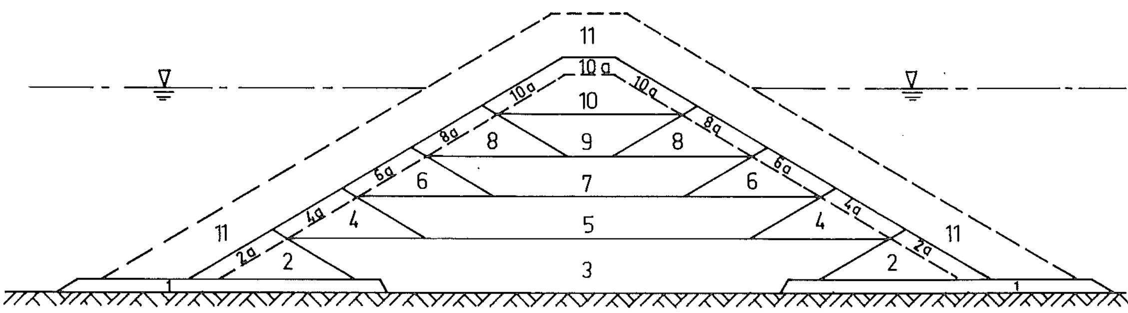


Figure 10.1
BREAKWATER CONSTRUCTED WITH CORE PROTECTION
CONSTRUCTION PROGRESSES IN NUMERICAL SEQUENCE

The construction steps shown in this figure proceed in numerical sequence, or, being more specific:

1. Filter layers are placed at each toe.
2. Ridges of secondary armor are placed. Only portion 2a is needed for stability of the final structure. A similar statement is true of the remaining even numbered layers.
3. Core material is placed between the ridges.

4-9 Alternate ridges and core layers are placed.

10. The upper layer consists entirely of secondary armor.
11. Primary armor is added after completion of the rest of the cross section.

    If there is severe attack, this armor may be added sooner, gradually as the other construction progresses.

Somewhat more secondary armor is used than would otherwise be the case. When this secondary armor is stone - as is usually the case - there are normally only minor economic consequences since secondary armor is no more expensive than core material. The success of this construction technique depends upon the secondary armor ridges to protect the core material sufficiently to prevent its mass erosion. In some cases where currents are very strong - closure of estuaries, for example - the entire core of the breakwater is built up of small armor units; this is an exception, however.

The economical construction of a breakwater requires that materials flow smoothly and that various production and transport units are well adapted to each other. When, for example, lower portions of a breakwater are constructed from ships with the upper position constructed from the crest, then even these two operations must be well coordinated.

Local availability of labor and materials also influences breakwater design and construction method choice. Concrete armor units are very often used in breakwaters in The Netherlands, primarily because stone of armor unit quality would have to be imported from foreign countries while concrete can be made locally.

10.5. Review

In this and the previous four chapters we have examined those factors which influence the design of a rubble mound breakwater. The designer's task is to combine all of these factors in such a way that all portions of the resulting breakwater are equally durable in relation to their individual environmental attack This balanced design will then ideally suffer either no damage or will be uniformly damaged by a severe storm.

The method for choosing the design storm is outlined in the following chapter on optimum design.

11. OPTIMUM DESIGN

J.F. Agema
W.W. Massie
A. Paape

11.1. Introduction

Optimum design refers to the dimensioning of a structure such that some chosen criterium has an extreme value. This definition is very general. The criterium used might be minimization of maintenance costs, for example, or the maximization of the ratio of benefits to costs. The choice of the criterium will have an effect on the resulting design. Other requirements for a project to be suitable for design optimization are explained in chapter 13 of volume I. As was pointed out there, some damage must always be accepted. The problem is one of finding the most economical balance between construction costs and damage (repair) costs such that the total of the two is minimized.

The discussion which follows will be restricted to the design of rubble mound breakwaters. (The application of optimum design techniques to monolithic breakwaters is the subject of chapter 19). In addition, a specific criterium function has been chosen: we shall want to minimize the sum of the construction and capitalized damage costs. Specific details of the optimization application will be discussed in the following sections of this chapter.

11.2. Parameters and Their Interrelationships

What are the parameters in the design of a rubble mound breakwater that can be varied in order to arrive at an optimum design? This can best be answered by examining the sources of damage expense. These sources fall into two categories, direct and indirect damage.

Direct damage is that associated with the breakwater itself. This includes all maintenance and repair costs of that structure.

Indirect damage costs occur within the area protected by the breakwater and result from its failure in some way. This failure can be different from that resulting in direct damage costs; for example, wave overtopping may make a harbor entrance so rough that ships cannot navigate through it during a storm even though no structural damage to the breakwater has occurred.

Expressed a bit more concretely, the total harbor optimization problem can be schematized as finding the minimum total project cost as a function of the following variables:

- breakwater location,
- crest elevation,
- breakwater type,
- details of construction such as armor unit type,
- wave climate.

For now, we neglect the first two of these factors; they determine the indirect damage costs.

Reviewing, direct damage involves repair of the breakwater, while indirect damage involves the operations which are normally carried out in its lee. Why do we separate these?

The two types of economic damage are separated because they influence two different aspects of our design. Direct economic damage results from the loss of stability of some part of our breakwater. This stability is dependent only upon the details of the design of a typical cross-section of the breakwater. Indirect damage, on the other hand, results from wave action in the harbor. This wave action is only influenced in a minor way by the details of a cross section (core porosity and crest elevation) while it is strongly dependent upon the geometry of the total harbor layout (location and width of entrance and harbor and breakwater layout). The design problem lends itself well to being split into two more or less independent parts. The first problem uses an analysis based upon approximate breakwater costs and indirect damage costs to design the harbor layout and determine the amount of wave energy which may be transmitted either through or over a breakwater. The breakwater designer then uses this limitation along with detailed breakwater cost figures and direct damage estimates to complete his portion of the optimum design. After completing this design, the resulting breakwater construction cost figure should be checked against that used in the layout optimization. In a complex, extensive harbor layout project, this iteration may go on for several cycles.*

Since we as breakwater designers are most interested in the breakwater details, we shall devote most of our attention here to the second part of this iteration cycle; the optimization of a cross section based upon construction and direct damage costs. On the other hand, we must fully realize that we are treating only a single facet of a much larger problem of which our optimum solution forms only a part.

In the remainder of this chapter we shall attempt to carry out the optimization of a single cross section of a rubble mound breakwater. In order to do this we will be given cost and wave data so that we may attempt to find an economic optimum design. We can achieve this optimum by varying the slopes and type of armor and, to a limited extent, the crest elevation.

11.3. Given Data

The following example is hypothetical in that data have been taken from various sources and were never intended to be used together in this combination. While the tie to a specific reality has been lost, the procedure illustrated is still perfectly valid.

Storm conditions

Wave conditions measured at a deep water site near our design location are given in table 11.1 and figure 11.1. Storm water levels mea-

* This discussion will be picked up again in section 11.8.

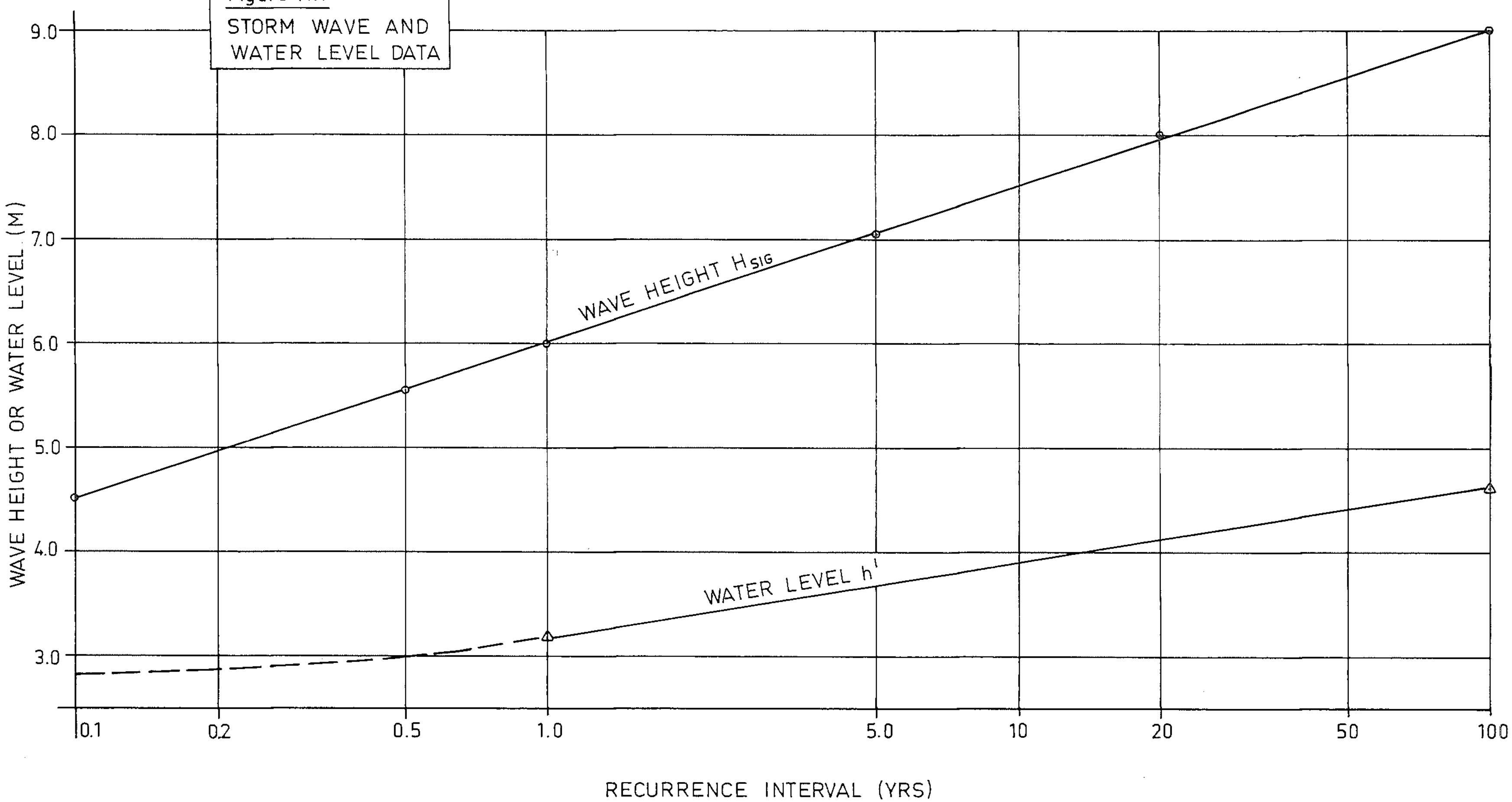


Figure 11.1
STORM WAVE AND WATER LEVEL DATA

sured essentially at our proposed breakwater site are also included in table 11.1 and are shown also in figure 11.1. It will be assumed that both the water levels and the storm waves occur simultaneously. In that table, H_{sig_0} is the significant wave height in deep water and h' is the water level relative to mean sea level.

Table 11.1 Storm Data

Recurrence Interval (yrs)	H_{sig_0} (m)	Period T (s)	h' (m)
0.1	4.5	7.4	
0.5	5.5	9	
1	6.0	10	3.2
5	7.0	11	
20	8.0	12	
100	9.0	13	4.6

Tides

The normal astronomical tide is such that high tide is 2.3 m above mean sea level and normal low water is 2.0 m below mean sea level. Tidal influences have been included in the water level data just given.

Site conditions

The depth at the design site is 10.0 m relative to mean sea level. The bottom material is sand having a mean diameter of 160 μm and the bottom slope is 1:100 at that depth.

Cost of materials

The following costs are assumed to be valid and are listed in table 11.2. Since the prices are intended only as a relative indication of costs, no monetary units are given. Costs would have to be determined individually by project, anyway, in a real case.

Table 11.2 Costs of Materials in place

Material	Use	Unit	Placement method: barge dumped	over crest
Natural Stone* (ρ = 2700 kg/m^3)	-	ton	35.	45.
Gravel		m^3	40.	50.
Normal concrete	Massive	m^3	-	150.
(ρ = 2400 kg/m^3)	Armor Cubes	m^3	200.	250.
	Special Armor	m^3	230.	280.
Basalt Concrete	Armor Cubes	m^3	230.	280.
(ρ = 2650 kg/m^3)	Special Armor	m^3	260.	310.

Requirements from harbor optimization

The crest of the breakwater is to be used only to reach a navigation light at the end for occasional maintenance. Since maintenance operations on this light need not be carried out during storms, waves can be allowed to break over the crest of the breakwater up to 5 times per year. Waves generated in the harbor by this overtopping will not hinder operations there. The economic life, ℓ, of the breakwater is to be 50 years; the interest rate, i, is 8% per year.

11.4. Preliminary Calculations

The following calculations must be carried out irrespective of the cross section chosen. They involve the transformation of the deep water wave data to that at the site. Data is taken from table 11.1 and interpolated using figure 11.1 for water levels. The computation shown in table 11.3 progresses as follows:

The deep water wave length, λ_0, is computed from the period, T, using equation 5.05a from volume I:

$$\lambda_0 = 1.56\ T^2 \qquad \text{(I-5.05a)} \quad (11.01)$$

The total depth, h, is the water level, h', plus the depth to mean sea level, 10 m.

The ratio H/H_0 is obtained from the value of h/λ_0 using table C-1 in volume III of the *Shore Protection Manual*. Refraction influences have been neglected. Value of $\frac{H_0}{\lambda_0 m^2}$ are computed using a given bottom slope, m, of 0.01. This is used as a breaker type parameter in table 8.1 of volume I. Since these parameter values are so large, the breaker parameter, p, is taken to be rather small as well: 0.1 is assumed. The breaker index, γ, is then computed from equation 8.03 of volume I:

---

* Only stone ranging in size up to 20 tons is available.

$$\gamma = 0.33p + 0.46 \qquad \text{(I-8.03)} \qquad (11.02)$$

yielding $\gamma = 0.49$.

The value of H_{sig} at the breakwater site is computed using either

$$H_{sig} = \gamma\ h \qquad (11.03)$$

or

$$H_{sig} = (\frac{H}{H_0})(H_{sig_0}) \qquad (11.04)$$

whichever yields a smaller value. Equation 11.03 determined the wave height for recurrence intervals > 1, while the waves of minor storms are affected only by shoaling, indicated by equation 11.04. These resulting wave heights have been plotted as a function of recurrence interval in figure 11.2. For convenience in later work, the frequency of occurrence- reciprocal of the recurrence interval - has also been included in table 11.3 and figure 11.2.

Primary armor will be extended down the front face of the breakwater to an elevation equal to 1.5 times H_{sig} below the water level. We choose the lowest of the following elevations:

a. common storm (H_{sig} = 4 m) at low tide:

$$h_a = (1.5)(4.) + 2.0 = 8.0 \text{ m below M.S.L.} \qquad (11.05)$$

b. severe storm, for example p = 0.01, at H.W.:

$$h_a = (1.5)(7.2) - 4.6 = 6.2 \text{ m below M.S.L.} \qquad (11.06)$$

c. as b above, but assuming low tide, the wave height is then:

$$(4.6 - 4.3 + 10)(0.49) = 5.05 \text{ m}$$

yielding (11.07)

$$h_a = (1.5)(5.05) - (4.6 - 4.3) = 7.2 \text{ m below M.S.L.}$$

where the tide range is 4.3 m.

Taking the greatest depth indicates that the primary armor should extend to MSL - 8.0 meters.

On the inner slope, the primary stone armor is continued to a depth 1 m below low water level, thus to MSL - 3.0 m.

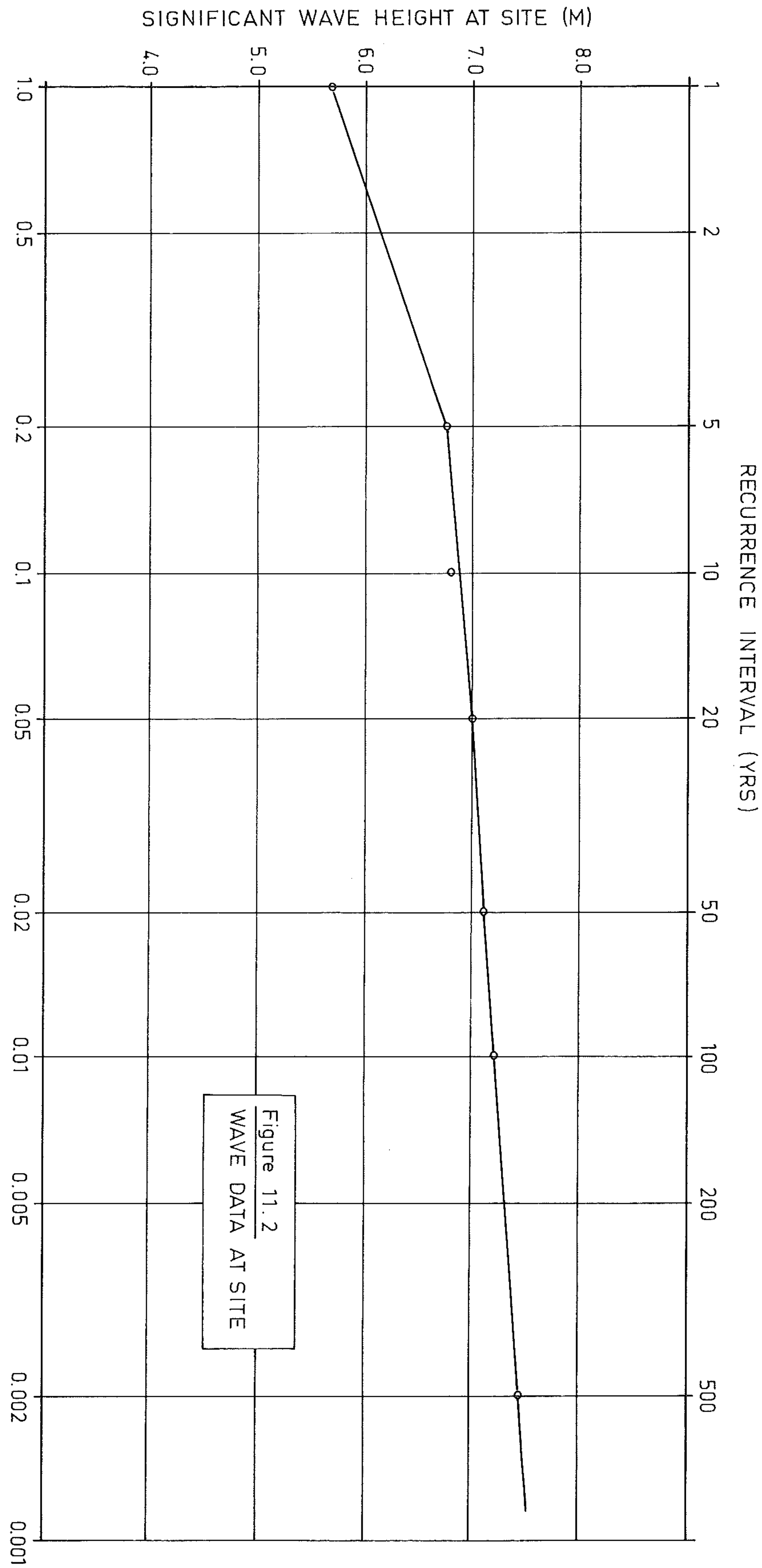


Figure 11.2
WAVE DATA AT SITE

TABLE 11.3 Wave Shoaling

Data from figure 11.1

Recurrence Interval (yrs)	H_{sig_o} (m)	T (s)	h' (m)	Wave length λ_o (m)	total depth h (m)	h/λ_o (-)	$\frac{H}{H_o}$ (-)	$\frac{H_o}{\lambda_o m^2}$ (-)	p (-)	γ (-)	H_{sig} (m)	frequency P ($\frac{\text{storms}}{\text{year}}$)	
0.1	4.5	7.4	2.8	85.	12.8	0.1506	0.9133	529.	0.1	0.49	4.1	10	
0.2	4.9	8	2.9	100.	12.9	0.1290	0.9172	490.	0.1	0.49	4.5	5	
0.5	5.5	9	3.0	126.	13.0	0.1028	0.9308	437.	0.1	0.49	5.1	2	
1	6.0	10	3.2	156.	13.2	0.0845	0.9487	385.	0.1	0.49	5.7	1	
5	7.0	11	3.7	189.	13.7	0.0725	0.9667	370.	0.1	0.49	6.7	0.2	broken wave
10	7.5	11.5	3.9	207.	13.9	0.0673	0.9766	362.	0.1	0.49	6.8	0.1	broken wave
20	8.0	12	4.2	225.	14.2	0.0631	0.9858	356.	0.1	0.49	7.0	0.05	broken wave
50	8.5	12.5	4.4	244.	14.4	0.0590	0.9958	348.	0.1	0.49	7.1	0.02	broken wave
100	9.0	13	4.6	264.	14.6	0.0553	1.006	341.	0.1	0.49	7.2	0.01	broken wave
500	10.0	14	5.1	306.	15.1	0.0493	1.025	327.	0.1	0.49	7.4	0.002	broken wave

11.5. Cost of Quarry Stone Breakwater

Since the maximum available armor unit mass is 20 tons, the Hudson Formula, equation 7.15, can be modified and solved for the slope:

$$\cot(\theta) = \frac{\rho_a\ g\ H^3}{K_D\ \Delta^3\ W} \tag{11.08}$$

where:

g is the acceleration of gravity,
H is the design wave height,
K_D is the damage coefficient,
W is the weight of the armor unit,
Δ is the relative density of armor,
ρ_a is the armor unit density, and
θ is the slope angle.

As an initial guess, let us design for a storm having a frequence of 0.05. From table 11.3 we see that the breakwater is attacked by breaking waves with H_{sig} = 7.0 m. For a double layer of rough quarry stone in breaking waves a damage coefficient value of 3.5 is found in tabel 7.6 of the *Shore Protection Manual.*

This value of K_D is incorrect, because it is based upon an assumption that no overtopping occurs. Since there will certainly be overtopping with the proposed design, this assumption has been violated. Unfortunately, damage coefficient values for overtopped breakwaters are not available. Therefore, the suggested value of K_D will be used further* *with specific acknowledgement of this error* since all of the computations must be verified via model tests, anyway. Thus, concluding, we must see the present computation as *only a preliminary estimate.*

Substitution into 11.08 yields:

$$\cot(\theta) = \frac{(9.81)(2700)(7.0)^3}{(3.5)\left(\frac{2700 - 1030}{1030}\right)^3 (20 \times 10^3 \times 9.81)} \tag{11.09}$$

$$= 3.10 \tag{11.10}$$

or: $$\theta = 18^o \tag{11.11}$$

This seems reasonable.

The crest elevation must be high enough to prevent overtopping more than 5 times per year. The wave height for this design criteria is then 4.5 m, and the water depth is 12.9 m. The wave length λ for h = 12.9 m and λ_o = 100 m is 78 m. Entering figure 11.3 with $\frac{H}{\lambda} = \frac{4.5}{78} = 0.057$ and a 1:3 rubble slope yields $\frac{R}{H}$ of 0.84. For the crest elevation we get:

---

* Damage determinations will, however, be modified to account for overtopping in section 11.6.

$$z_c = (0.84)(4.5) + 2.9 = 6.7 \text{ m} \qquad (11.12)$$

above mean sea level. (The mean water level is 2.9 m above M S L).

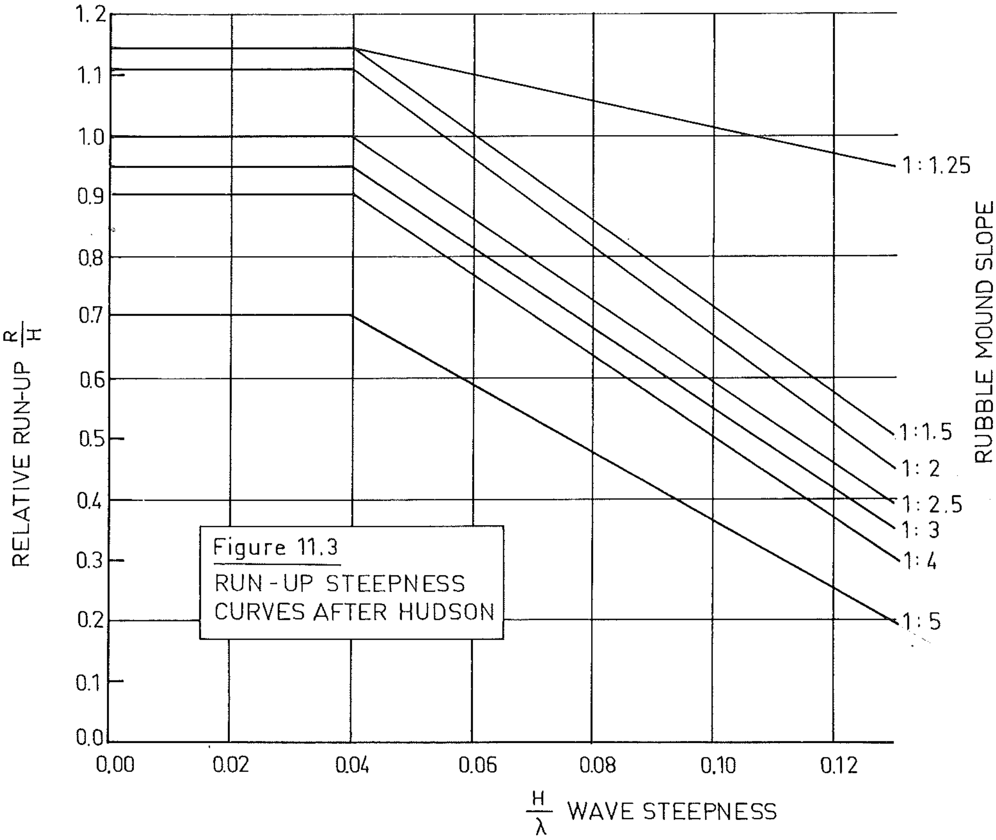


Figure 11.3
RUN-UP STEEPNESS CURVES AFTER HUDSON

The minimum crest width follows from equation 7.23:

$$B = m' \, K_\Delta \left(\frac{W}{\rho_a \, g}\right)^{1/3} \qquad (7.23) \quad (11.13)$$

where:

B is the crest width,

K_Δ is the packing coefficient, and

m' is the number of armor units across the crest.

Choosing m' = 3 and selecting K_Δ= 1.02 from chapter 6 yields:

$$B = (3)(1.02)\left(\frac{20 \times 10^3}{2700}\right)^{1/3} \qquad (11.14)$$

$$= 6.0 \text{ m}$$

This is wide enough for construction equipment, if necessary.

The thickness of the armor layer, t, comes from equation 7.21:

$$t = m\ K_\Delta\ \left(\frac{W}{\rho_a\ g}\right)^{1/3} \qquad (7.21) \qquad (11.15)$$

where

m is the number of units in the layer.

Since m = 2 has already been chosen, 11.15 yields:

$$t = (2)(1.02)\left(\frac{20 \times 10^3}{2700}\right)^{1/3} \qquad (11.16)$$

$$= 4.0 \text{ m}$$

We can now start a sketch design shown in figure 11.4.

This design needs a special toe construction on the front face.

The secondary armor units must be dimensioned. These would have a mass of at least 1/10 of that of the primary armor but should also withstand the less severe storms.* Taking, for this, a design storm frequency of 10 per year yields H_{sig} = 4.1 m and

* This is important during the construction phase.

$$W = \frac{(2700)(9.81)(4.1)^3}{(3.5)\left(\frac{2700 - 1030}{1030}\right)^3 (3.10)} \qquad (11.18)$$

$$= 3.9 \times 10^4 \text{ N} \qquad (11.19)$$

Since this is heavier then 1/10 of the primary armor weight, this will be used. The layer thickness is now:

$$t = (2)(1.02)\left(\frac{3.9 \times 10^4}{(2700)(9.81)}\right)^{1/3} \qquad (11.20)$$

$$= 2.3 \text{ m}$$

If this same stone is used for toe protection then the slope of this toe will be:

$$\cot(\theta) = \frac{(2700)(9.81)(7.0)^3}{(3.5)\left(\frac{2700 - 1030}{1030}\right)^3 (3.9 \times 10^4)} \qquad (11.22)$$

$$= 16.6 \qquad (11.23)$$

This is outside the range of validity of the Hudson Formula, and is an extremely flat slope. This can be improved only by choosing a heavier stone for the toe construction. Choosing stone having a mass of 6 tons yields:

$$\cot(\theta) = \frac{(2700)(7.00)^3}{(3.5)\left(\frac{2700 - 1030}{1030}\right)^3 (6 \times 10^3)} \qquad (11.24)$$

$$= 10.35;\ \theta = 6^{\circ} \qquad (11.25)$$

This is still pretty flat!

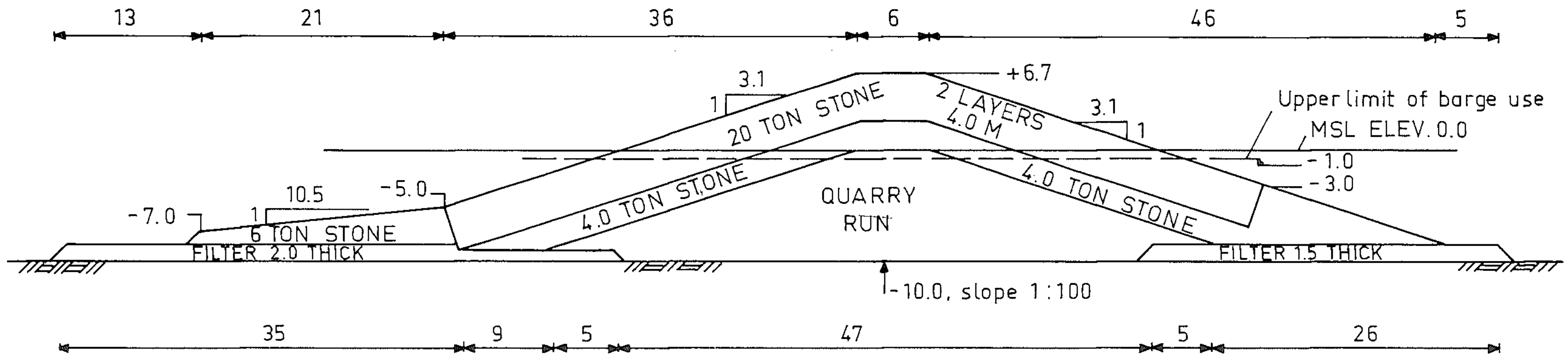


Figure 11.4

SKETCH DESIGN OF STONE BREAKWATER
ORIGINAL SCALE 1:500

TABLE 11.4 Initial Cost Estimate - Stone Breakwater

Item and dimensions (scaled from fig. 11.4)	Volume (m^3/m)	Unit price	Total price
Filter Gravel			
31 x 1.5	46.5		
14 x 1.5	21.0		
35 x 2.0	70.0	20/ton	
	137.5	$40/m^3$	5 500.
Toe Stone			
23 x 1.5	34.5		
22 x 2 x ½	22.0		
	56.5	$60/m^3$	3 390.
Quarry Run (barge placed)			
47 x 1.5	70.5		
46 x 7.5 x ½	172.5		
11 x 7.5	82.5		
	325.5	$70/m^3$	22 785.
Secondary Armor (barge placed)			
25 x 2.3	57.5		
22 x 2.3	50.6		
16 x 5 x ½	40.0		
	148.1	$60/m^3$	8 886.
Quarry Run (over crest)			
9 x 1.4	12.6	$90/m^3$	1 134.
Secondary Armor (over crest)			
6 x 2.3	13.8		
6 x 2.3	13.8		
7 x 2.3	16.1		
	43.7	$75/m^3$	3 278
Primary Armor (over crest)			
37 x 4	148.0		
30 x 4	120.0		
6 x 4	24.0		
	292.0	$75/m^3$	21 900.
		Total cost per meter:	66 873

All of these results are incorporated in figure 11.4; this is now sufficiently detailed to make an estimate of the construction materials required. This will be done for a 1 meter length of dam. Results are listed in table 11.4. Gravel listed for the filter layers is assumed to have a bulk density of 2000 kg/m^3. All materials below elevation -1.0 m are dumped from barges except the primary armor. All of this is placed by crane working from the crest.

The bulk density, ρ_b, of armor units follows from the density, ρ_a,

$$\rho_b = \rho_a (1 - n) \tag{11.26}$$

$$= (2700)(1 - 0.37) = 1700 \text{ kg/m}^3 \tag{11.27}$$

where n = 0.37 comes from chapter 6.
This yields a unit price for barge-dumped stone of 60./m^3 and 75./m^3 for crane-placed stone. Quarry run stone is assumed to have a bulk density of 2000 kg/m^3.

The cost figure just obtained at the end of table 11.4 is the construction cost of a breakwater designed to withstand a significant wave height of 7.0 m. In order to conduct an optimization, we need to investigate the construction costs for a whole series of wave heights. This involves, in principle, a whole series of cost determinations as just completed. We may, however, be able to short cut this lengthy computation for the problem at hand.

Since the crest elevation has been determined based upon an overtopping criteria, that elevation will remain relatively fixed. Run-up is rather independent of wave and slope parameters in this range - see fig. 11.3. The crest elevation is, therefore, considered to be constant. Also, since armor stone of maximum size is used, the crest width and primary armor layer thickness will remain constant. What will change, then? The side slopes, the size of the secondary armor (and hence the layer thickness), and the core volume will change. The volume of the toe and filter constructions will remain essentially the same.

The procedure used to compute table 11.5 from the data with H_{sig} = 7.0 m is outlined as follows:

a. The new slope follows from (11.09) with the new wave height.
b. Changes in primary armor volume arise exclusively from changes in slope length.
c. Secondary armor masses follow from (11.18) with the new slope; the wave height, 4.1 m, is maintained.
d. The layer thickness follows from (11.20).
e. The barge volume is derived from slope length and thickness changes.
f. The crane-placed volume changes only because of the layer thickness change.
g. The core volume changes result from width changes at the base.
h. Other volumes and all unit prices are assumed to remain the same.

The resulting costs can be plotted in a graph of initial construction cost as a function of design significant wave height. This will be done, but only after the damage costs have been determined in the following section.

Table 11.5 Cost as Function of Wave height for Stone Breakwater.
Note: Costs are listed with *italic* numbers.

Item					
Design Wave height (m)	5.7	6.75	7.0	7.25	7.50
Slope cot (θ)	1.68	2.78	3.10	3.45	3.82
Primary Armor					
volume (m^3/m)	184.9	267.1	292.	319.5	348.9
cost/m	*13 864.*	*20 031.*	*21 900.*	*23 965.*	*26 167.*
Secondary Armor					
mass (kg)	7400.	4500.	4000.	3600.	3300.
layer thick.(m)	2.8	2.4	2.3	2.2	2.2
barge volume (m^3/m)	119.0	142.3	148.1	154.0	165.3
cost/m	*7 140.*	*8 540.*	*8 886.*	*9 242.*	*9 921.*
crane volume (m^3/m)	39.8	42.9	43.7	44.5	47.4
cost/m	*2 985.*	*3 219.*	*3 278.*	*3 338.*	*3 555.*
Core					
barge volume (m^3/m)	220.1	309.2	325.5	363.5	393.4
cost/m	*15 406.*	*21 643.*	*22 785.*	*25 442.*	*27 539.*
Other items cost(m)	*10 024.*	*10 024.*	*10 024.*	*10 024.*	*10 024.*
Total cost/m	*49 419.*	*63 457.*	*66 873.*	*72 011.*	*77 206.*

11.6. Damage to the Breakwater

The second part of the optimization problem is to determine the equivalent capital investment necessary to finance the damage which can be statistically expected during the life of the breakwater.

The discussion which follows is somewhat different from that presented in chapter 11 of volume I. The most important differences are first, that we work directly with the significant wave height characterizing a storm and second, we are interested in a frequency of occurrence rather than a frequency of exceedance. The first of these differences implies that we no longer are concerned with the Rayleigh Distribution of wave heights within a storm; all the necessary information is contained in the long term distribution of wave heights shown in figure 11.2. The frequencies of occurrence can be derived from the exceedance frequencies given in that figure by dividing the wave heights into intervals characterized by a given value of H_{sig}, and determining the frequency of occurrence of that significant wave height by subtracting the frequencies of exceedance at the edges of the interval. The boundaries of the intervals chosen are shown in column 1 of table 11.6; the associated probabilities of exceedance, $P(H_{sig})$ taken from figure 11.2 are listed in the following column. The characterizing significant wave height and frequency of occurrence are listed in columns three and four.

In that table, each of the following groups of four columns is used for a different one of the five design cross sections worked out in the previous section. For illustrative purposes, the computation for the profile with a design wave of 7.0 m will be described in detail.

To proceed further, we must relate wave heights exceeding the design conditions to expected damage to the breakwater. In chapter 6 values of K_D, the damage coefficient, are given for a non-overtopped slope attacked by non-breaking waves. It is reasonable to assume the ratio of the damage coefficient for some percentage of damage to that for no damage is the same for both breaking and non-breaking waves. The effect of overtopping, however, is an increase in the damage to the structure since a single wave spilling over the crest will damage both the inner and outer slopes.* Therefore, the damage figures have been doubled. This results in the graph shown in figure 11.5 used with all five breakwater profiles.

The equivalent damage coefficient value follows from equation 7.20 modified to yield the ratio of damage coefficients.

$$\frac{K_D^*}{K_D} = \left(\frac{H^*}{H}\right)^3 \tag{11.28}$$

In this equation the K_D ratio follows from the ratio of H^* (listed in column 3 of table 11.6) to the design wave height for each cross section.

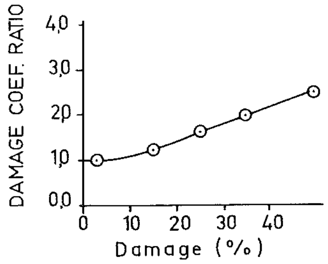


Figure 11.5
DAMAGE RELATIONSHIP FOR ROUGH QUARRY STONE

The damage percentages for each cross section and wave height come from figure 11.5, entering with the damage coefficient ratio and reading a damage to the armor layer in percent. Obviously, for waves smaller than the design wave there is no damage; why is this? See chapter 7!

The damage costs are found by multiplying the damage percentages by the initial cost of constructing that portion of the breakwater which must be repaired. Usually, for moderate damage, the cost of the primary armor layer from table 11.5 is chosen.** This resulting figure is then increased to compensate for the extra cost of mobilizing the construction equipment for such a relatively minor repair job. The increase factor and cost basis used are listed in the notes below table 11.6. These figures are *quite arbitrairly chosen* and should be checked with contractors in a real situation.

The annual cost of the damage is computed by multiplying the damage cost per storm, just computed, by the chance of occurrence of that storm listed in column 4 of table 11.6. These annual costs of damage are then added for each design profile at the bottom of the respective columns.

---

* See also Van de Kreeke and Paape (1964).

** Exceptions to this will be noted in table 11.6.

TABLE 11.6 Breakwater Damage Computations

Wave Conditions				Design Wave: 5.7 m				Design Wave: 6.75 m				Design Wave: 7.00 m			
H_{sig} (m)	$P(H_{sig})$ (-)	Char. H_{sig} (m)	ΔP (H_{sig}) (-)	K_D ratio (-)	Damage (%)	Damage cost (-/m)	Annual cost (-/m)	K_D ratio (-)	Damage (%)	Damage cost (-/m)	Annual cost (-/m)	K_D ratio (-)	Damage (%)	Damage cost (-/m)	Annual cost (-/m)
5.5	1.25														
		5.8	0.62	1.05	8.0	2218.	1375.								
6.0	0.63														
		6.3	0.35	1.35	18.	4991.	1747.								
6.5	0.28														
		6.7	0.195	1.62	25.	8996.	1754.	0.98	0.0	-	-				
6.9	0.085														
		7.0	0.065	1.85	31.	11155.	725.	1.12	12.	4807.	312.	1.00	2.5	1095.	71.
7.1	0.020														
		7.2	0.015	2.02	36.	12954.	194.	1.21	15.	6009.	90.	1.09	10.	4380.	66.
7.3	0.005														
		7.5	0.005	2.28	43.	21250.	106.	1.37	19.	7612.	38.	1.23	15.	6570.	33.
7.7	0.000						5901.				440.				170.

Costs used in damage comp.	Design Wave: 5.7 m	Design Wave: 6.75 m	Design Wave: 7.00 m
Primary Armor :	13 864.	20 031.	21 900.
Total Armor :	23 989.	31 790.	34 064.
Total Construction:	49 419.	63 457.	66 873.

Note: For damage up to 20%, the damage cost is based upon 2 times the primary armor cost.
For damage of 20% to 40%, the damage cost is based upon 1.5 times the total armor cost.
For damage above 40%, the damage cost is based upon the total construction cost.

Wave Conditions				Design Wave: 7.25 m				Design Wave: 7.50 m			
H_{sig}	$P(H_{sig})$	Char. H_{sig}	ΔP (H_{sig})	K_D ratio	Damage	Damage cost	Annual cost	K_D ratio	Damage	Damage cost	Annual cost
(m)	(-)	(m)	(-)	(-)	(%)	(-/m)	(-/m)	(-)	(%)	(-/m)	(-/m)
7.1	0.020										
		7.2	0.015	0.98	0.0	-	-				
7.3	0.005										
		7.5	0.005	1.11	11.	5272.	26.	1.00	2.5	1308.	7.
7.7	0.000						26.				7.

Costs used in damage comp.	Design Wave: 7.25 m	Design Wave: 7.50 m
Primary Armor :	23 965.	26 167.
Total Armor :	36 545.	39 643.
Total Construction:	72 011.	77 206.

11.7. Optimization of quarry stone breakwater.

In order to compare these annual costs to the initial construction costs, it is necessary to determine what sum of money, set aside now at compound interest, will just pay for this damage over the lifetime of the structure. This transformation involves determining the present value of a series of uniform withdrawals (payments) equal to the annual damage cost over the life of the structure. The present value of the maintenance payments is determined by multiplying the annual payment by the present worth factor, pwf. From finance,

$$pwf = \frac{(1+i)^n - 1}{i(1+i)^n} \tag{11.29}$$

where:

i is the interest rate per period expressed as a decimal, and
n is the number of periods.

Substituting an interest rate of 8% (i = 0.08) and a number of periods, $n = \ell = 50$, yields:

$$pwf = \frac{(1.08)^{50} - 1}{0.08(1.08)^{50}} \tag{11.30}$$

$$pwf = 12.2335 \tag{11.31}$$

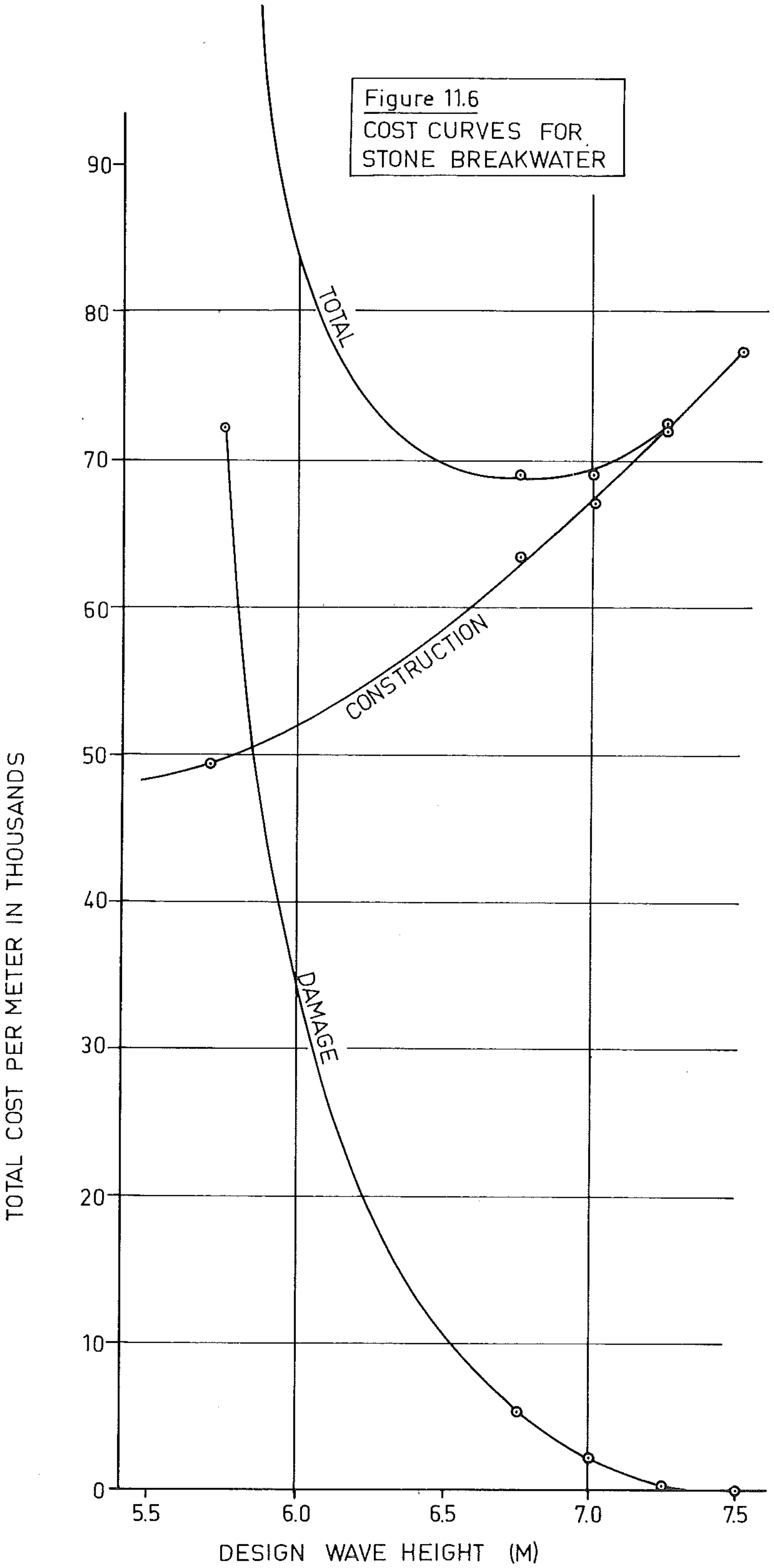
This present worth factor is then multiplied by each total annual cost figure for each cross section. These resulting present values can then be added to the initial construction costs to yield a total cost.* This data gathered and computed from tables 11.5 and 11.6 is summarized in table 11.7 and is shown graphically in figure 11.6.

Table 11.7 Cost Summary

Item					
Design Wave Height (m	5.7	6.75	7.00	7.25	7.50
Annual Damage Cost	5 901.	440.	170.	26.	7.
Capitalized Damage	72 190.	5383.	2080.	318.	86.
Construction Cost	49 419.	63 457.	66 873.	72 011.	77 206.
Total Cost	121 609.	68 840.	68 953.	72 329.	77 292.

The minimum point of the total cost curve in figure 11.6 occurs near a design wave height 6.75 m, while there is little difference in total cost between a cross section designed for a 6.75 m wave and one designed for 7.0 m (0.2% in total price). On the other hand, the maintenance costs of the design for a 7.0 m wave are only 39% of those for the 6.75 m wave. This would tend to make the design for the 7.00 wave seem preferable. It certainly would be if the difference in construction costs was no problem. The heavier design costs 5.4% more to build than the lighter of the two cross sections. This might present a problem if construction capital is in short supply (The extra maintenance cost of the lighter construction do not have to be paid now). One may argue that the reasoning just presented undermines the philosoply of the optimum design. This is not really the case, since a comparison is being made between two designs which cost essentially the same - the price different is less than the errors inherent in the cost determinations.

* An alternate, and equivalent total result could be achieved by depreciating the construction cost over the life, ℓ. This annual depreciation figure would be added, then, to the total annual maintenance cost from table 11.6.



Summarizing, the conclusion is use a 6.75 m design wave (recurrence interval of 5 years) if construction capital is scarce and design using a 7.00 m wave (recurrence interval 26 years) if capital is plentiful.

What is the effect of changing the economic life, ℓ, of the structure? As n decreases with a given interest rate, i, the *pwf* decreases making maintenance costs less important. Thus, the optimum design point shifts to the left in figure 11.6; this seems logical. Reducing the life to, say, 10 years yields an optimum nearer a design wave height lower than 6.75 m. This is revealed by constructing a new table similar to table 11.7.

How does the interest rate affect the optimum? As the interest rate decreases, the present worth factor increases making maintenance a more important contributor to the total costs; the optimum shifts to the right in figure 11.5. For example, with an annual interest rate of only 3%, and a life of 50 years, calculation of a new table similar to table 11.7 yields an optimum near 7.00 m; the total cost curve climbs steeply to the left of this point. This was not so pronounced in figure 11.6 and results from the relatively high current (1976) interest rate used.*

For "normal" designs the optimum design storm wave will have a recurrence interval of about 10 to 20 years. It is for this reason that the initial guess for a design wave height was 7.0 m corresponding to a recurrence interval of 20 years - see figure 11.2.

11.8. Additional Remarks

By now, everyone concerned with this chapter (authors, typist, proofreader, students) thinks or hopes that the problem is solved. Unfortunately, this is far from true. In sections 5 through 7 of this chapter we have found the optimum quarry stone breakwater consistent with the rest of the preliminary harbor design. This *is not necessarily* the optimum breakwater or even the optimum rubble mound breakwater. Theoretically, we should repeat the procedures just used in the previous three sections to determine optimum designs using various artificial armor units such as cubes or tribars. An optimum design for a monolithic vertical breakwater should also be made.** The true optimum solution would then be the cheapest of all these individual optimum solutions.

We should, in addition, tie our optimum breakwater design to an optimization of the total harbor complex. (These were split in section 11.2).
This can be very important when the breakwaters represent an important portion of the harbor investment. This involves adding another "dimension" to our optimization, namely the crest elevation. Thus, optimizations of stone breakwaters for a series of crest elevations can be made. Choosing from each crest elevation the best design yields a new curve of cost versus crest height. This can be combined with harbor data to optimize the total project.

* It can be argued that a very low real interest rate, equal to the borrowing interest rate minus the inflation rate, should be used in these computations.

** This will be done in chapter 19.

The work presented in sections 4 through 7 of this chapter can give the impression that a true optimum design can be made based purely upon computations; *this is certainly not the case.*
The relationship between wave height and percent damage (effectively figure 11.5) must be determined by experiment, especially when overtopping can be expected.
Effects of scour on the toe construction for the breakwater must be investigated via a model; all soil mechanics aspects have been ignored in the present analysis. Since the breakwater is exposed to breaking waves, the value of γ chosen or determined - equation 11.02 - can have an appreciable influence on the design. This factor, also, can be checked in a model.

All breakwater costs have been determined for a 1 meter long typical section of the structure. In a real harbor design problem various portions of the breakwater would be exposed to different wave climates because of variations in water depth and wind fetch, for example. Several cross sections must, therefore, be optimized. However, the designer must remain aware of the fact that the cross-sections are inter-related. It would normally not be economical, for example, to use a whole variety of different types of armor units on different sections of the same breakwater.

The extent of the total breakwater project also affects the optimization via the maintenance costs. How? In the presentation above it was assumed that the cost of repairing the primary armor layer was twice as much per unit volume as its construction cost. Since the cost of mobilizing the necessary construction equipment to a given site is pretty much independent of the amount of damage to be repaired, it is, in fact, relatively much more expensive to replace 100 armor units on a breakwater 500 m long than to replace 1000 units on a structure ten times as long. Therefore, the ratio of armor layer unit maintenence cost to unit construction cost - assumed to be 2, above - decreases as a project becomes larger. This is the reason that this factor decreased with increasing damage. Thus, maintenance costs become relatively more important for smaller - shorter - structures; the optimum point will shift to the right on figure 11.6, toward a higher design wave. For very small projects, such as a yacht harbor in a more or less protected location, it is often most economical to design the breakwater to withstand the maximum expected wave - a design for no damage.

The damage cost calculation presented in section 11.6 was based upon an assumption that damage to the breakwater was repaired immediately regardless of its extent. Such an approach is conservative. If unrepaired minor damage can lead to more severe damage in a later storm than would otherwise be expected, then such conservatism would be necessary. If, on the other hand, partial damage now-less than a certain percentage - does not affect future damage, then it is no longer necessary or economical to conduct minor repairs. Nijboer

(1972) has investigated this problem somewhat experimentally. Much further research is needed to determine which of the above hypotheses about partial damage is correct and what limit of partial damage can be tolerated before repairs are made.

J.F. Agema
W.W. Massie

12. EXAMPLE OF RUBBLE MOUND BREAKWATER

Complete descriptions of the design and background philosophy of specific rubble mound breakwaters are difficult to find in the literature. Information over stone rubble mound breakwaters is especially hard to find in published form. Obviously design reports are prepared but these are most often proprietary and are not for general publication such as in this book.

One example, the design of the new harbor entrance at Rotterdam, will be treated in detail. However, since both rubble mound and monolithic breakwaters were considered for that harbor entrance, this presentation is postponed until after the principles of monolithic breakwater design have been treated.

13. MONOLITHIC BREAKWATERS

E.W. Bijker

13.1. Definition

The most striking characteristic of a monolithic breakwater is adequately described by its name, monolith - a single large stone. Thus, a monolithic breakwater consists, eventually, of a single massive unit even though it may be constructed from smaller elements rigidly connected together.

This chapter will serve further to introduce the various problems in the design of monolithic breakwaters. These will be amplified in separate chapters which follow; examples will be given at the end in chapter 20.

13.2. General Features

The monolithic form of these breakwaters can be both an advantage and a disadvantage. Several of these advantages with respect to rubble mound breakwaters have already been mentioned in chapter 3. The most important advantages are savings in material and potentially quick construction. Its major disadvantage is that a loading exceeding the design condition can result in immediate total failure. The consequences of this for the optimum design procedure will be highlighted in chapter 19.

Most monolithic breakwaters are vertical - faced. This is not a necessary condition in terms of type characterization; it is simply a matter of construction convenience. The most traditional form of monolothic breakwater is constructed from large blocks as shown in figure 13.1. These blocks can be cut from stone, but dense concrete is probably more common. The blocks in this figures are 3 x 3 x 6 m with a volume of 54 m^3 and a mass of about 130 tons.

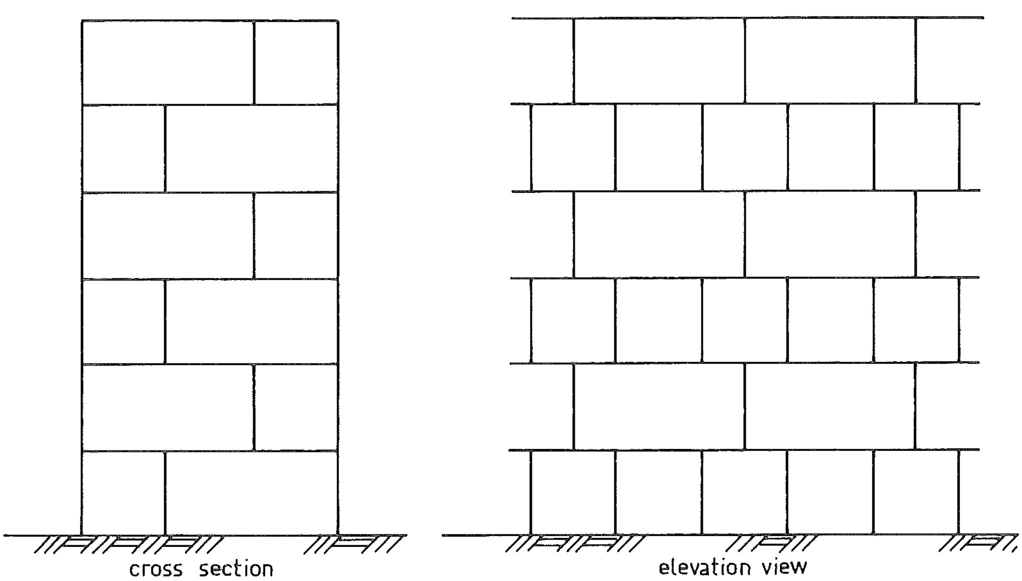


Figure 13.1
TYPICAL MONOLITHIC BREAKWATER
SCALE 1 : 150

Obviously, pretty heavy construction equipment is needed. This aspect is discussed in more detail in chapter 18.

Why use such heavy blocks if they make construction so difficult? This is done because this breakwater derives its stability under wave action almost exclusively from static friction forces between the blocks. This requires that at least the upper blocks be heavy. Sometimes the blocks are dowelled together vertically with heavy steel bars which transmit shear forces across the horizontal joints. Such a construction is impractical when natural cut stone blocks are used. Special properties of materials for use in monolithic breakwaters will be discussed in chapter 14.

It was assumed when making figure 13.1 that the ground upon which the breakwater was constructed was smooth and horizontal. Since the chance of this occurring naturally is small, another more flexible (in terms of foundation) form is chosen as shown in figure 13.2. As will become apparent in chapter 16, the fill material between the uneven bottom and the heavy blocks is subjected to especially difficult loadings. In fact, many vertical breakwaters fail due to foundation failure resulting from wave impact forces - see chapter 15.

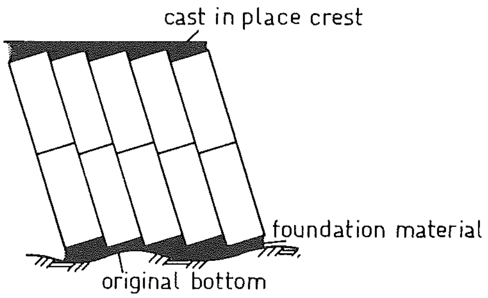


Figure 13.2
MONOLITHIC BREAKWATER ON ROUGH BOTTOM
SCALE 1:1500

At locations where there is a very limited available working time, hollow concrete caissons can be floated into position, sunk by flooding with water and then ballasted with rubble or sand. A cross-section of such a breakwater is shown in figure 13.3. The skirts may be added in order to increase the horizontal stability of the structure. Their effectiveness for the foundation is explained in chapter 16.

The cap shown in the figure is made from either asphalt or portland cement concrete after the fill has been placed; sand and grouted rubble are the most common fill materials.

Large caissons can be much bigger than that shown in figure 13.3. Rectangular units as large as 20 m high, 15 m wide and 60 m long have been built.

Methods for aligning and connecting adjacent caisson units are discussed in chapter 18.

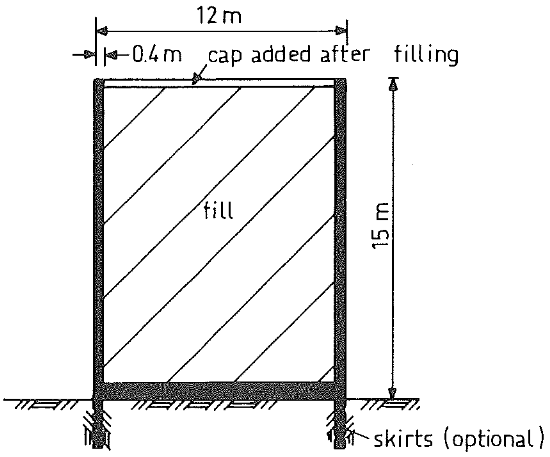


Figure 13.3
CAISSON CROSS SECTION
SCALE 1:1500

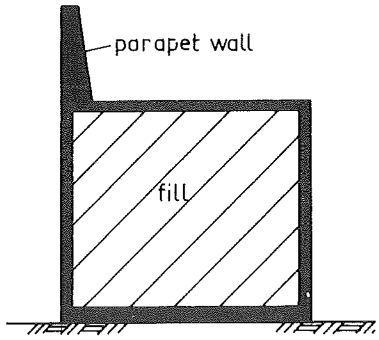


Figure 13.4
CAISSON WITH PARAPET
SCALE 1:1500

When stability considerations allow it, material can be saved by making the crest of the breakwater lower and extending a cantilevered wall upward on the sea side to prevent overtopping. This concept is shown in figure 13.4. This parapet is usually cast in place after the caissons have been placed*; as such, it can be used to create a neat appearance by camouflaging the misaligment of the units.

Other forms of caissons and other methods of placement can be used. Vertical cylindrical concrete caissons have been placed by a special crane operating from the crest of the completed breakwater. This was first done at Hanstholm, Denmark; that breakwater with a sloping front also employed there served as the prototype for what is now called the Hanstholm type of monolithic breakwater. As will be shown in chapter 15, the chamfered sloping front on such on monolithic breakwater can reduce the magnitude of wave impact forces considerably; other benefits for the foundation are described in chapter 16. Figure 13.5 shows such a breakwater.

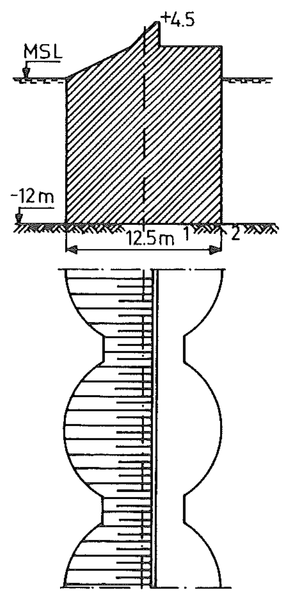


Figure 13.5
HANSTHOLM BREAKWATER

Another method of reducing the wave forces on monolithic breakwaters is construct a hollow perforated chamber on the weather side of the structure. Such a concept was used at Baie Comeau in Quebec, Canada and is shown in figure 6-71 of the *Shore Protection Manual*. This same principle was applied to the Ekofisk oil storage tank in the North Sea.

Wave action causes more than just direct loadings on a vertical breakwater. Serious erosion problems can be caused by a standing wave which can develop before a vertical reflecting breakwater - see chapter 17. Since these problems are most severe when the foundations are shallow, as with caissons, an alternate form of monolithic breakwater consists of vertical steel sheet pile cells. When bottom conditions are favorable, the interlocking sheet piles can be driven into the bottom to sufficient depth to avoid foundation problems. After the cells are completed they are

* A parapet wall can be used with any type of monolithic breakwater, however.

filled and capped just as is done with caissons. Unfortunately, a relatively long time is needed at the breakwater site to drive the sheet piles as compared to floating caissons into position.

When weather conditions dictate the use of caissons and foundation problems prohibit their permanent use, a composite form of structure such as shown in figure 13.6 is sometimes used. The initial construction consists of the caissons which are then protected on the weather side by the rubble mound slope placed against it. This armored slope can be designed using the techniques applicable for rubble mound breakwaters explained earlier in these notes. In figure 13.6 it is obvious that the caisson top provides an excellent work road for placing the rubble slope.
We start the treatment of specific details in the following chapter by discussing the necessary properties of monolithic breakwater materials.

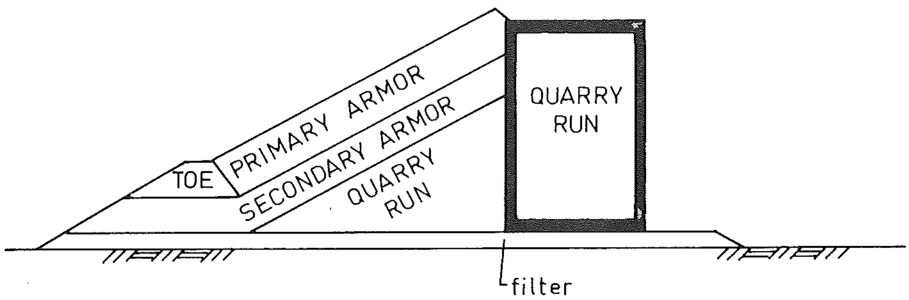


Figure 13.6
COMPOSITE BREAKWATER
SCALE 1:500

14. CONSTRUCTION MATERIALS

W.W. Massie

14.1. Introduction

Since the same materials as are used in rubble mound breakwaters are exposed to more or less equivalent environmental conditions one might conclude that the material properties required for monolithic breakwaters might also be the same as for rubble mound armor units. This is not the case; there are important differences in construction details and environmental attack. The properties of the materials listed below can be seen as an extension and modification of the list presented earlier in chapter 6.

14.2. Environmental Differences

In contrast to rubble mound breakwaters which usually absorb much of the oncoming wave energy, monolithic structures, because they are less permeable, tend to keep the wave energy "on their surface". In other words, the oncoming wave energy is either reflected back away from the breakwater or dissipated in run-up on the (impervious) surface.

The large flat surfaces which characterize so many monolithic breakwaters must be constructed of materials specially selected to resist the particular attack. Specifically, those waves which break against a monolithic structure can cause high (tens of atmospheres) but short duration (milliseconds) impact forces. These are described in more detail in the following chapter.

14.3. Consequences for Materials

Remembering Pascal's law and experiment from elementary fluid mechanics one realizes that if such a high hydrodynamic impact force should occur on a water filled joint or crack (even a hairline crack is sufficient) this pressure will act undiminished over all surfaces of this crack. This can lead to progressive fracture or spalling of the material. Obviously, prevention of crack formation is the simplest cure for this problem. Thus granite or basalt stone used to construct a breakwater of massive cut blocks should be fine-grained and not jointed.*

Concrete used should have an especially smooth surface.

Further, since many monolothic concrete structures contain steel, either as reinforcing or as pre-tensioning, the surrounding concrete must be sufficient to protect this steel from direct chemical attack.

Since monolithic breakwaters are designed to behave as a single massive unit, the density property of armor units for rubble mound breakwaters is much less important for monolithic structures.

*This word is used here in the geological sense.

15. WAVE FORCES ON VERTICAL WALLS

W.W. Massie

15.1. Introduction

When non breaking waves attack a vertical impermeable breakwater surface with their crests parallel to the breakwater axis, they are almost totally reflected. This reflected wave, or clapotis,* is discussed in the following section.

When, on the other hand, a breaking wave hits a vertical barrier, entirely different additional forces are generated. These are discussed in sections 15.3 and 15.5.

15.2. Standing Waves

As long as the water depth at the toe of the vertical wall is sufficient, the approaching waves will be reflected forming a non-breaking standing wave. We may remember from short wave theory that a standing wave results from the superposition of two travelling waves. An antinode of this standing wave will be found at the vertical wall location. The pressure distribition on this wall follows from the theory of short waves presented in volume I chapter 5. From equation 5.11 in that volume:

$$p = -\rho g z + \rho g H \frac{\cosh k(z+h)}{\cosh kh} \cos \omega t \qquad (15.01)$$

where:
- g is the acceleration of gravity,
- H is the wave height of the approaching wave,
- h is the water depth,
- k is the wave number $= \frac{2\pi}{\lambda}$,
- p is the instantaneous pressure,
- T is the wave period,
- t is time,
- z is the vertical coordinate measured from the water surface (positive up),
- λ is the wave length,
- ρ is the mass density of water, and
- ω is the wave frequency $= \frac{2\pi}{T}$.

Note that the crest to trough water level difference at the wall will be equal to 2H. Figure 15.1 shows the extreme pressure distributions acting on a vertical wall. An approaching wave of 2m height with 5 s period was used to plot the figure; the water depth is 12 m. The commonly used linear interpolation for the maximum pressure under a wave crest above the still water level is shown as a dashed line.

If we examine the pressure at some fixed point on the wall as a function of time, we see that it varies as a cosine function about

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* French, meaning standing wave.

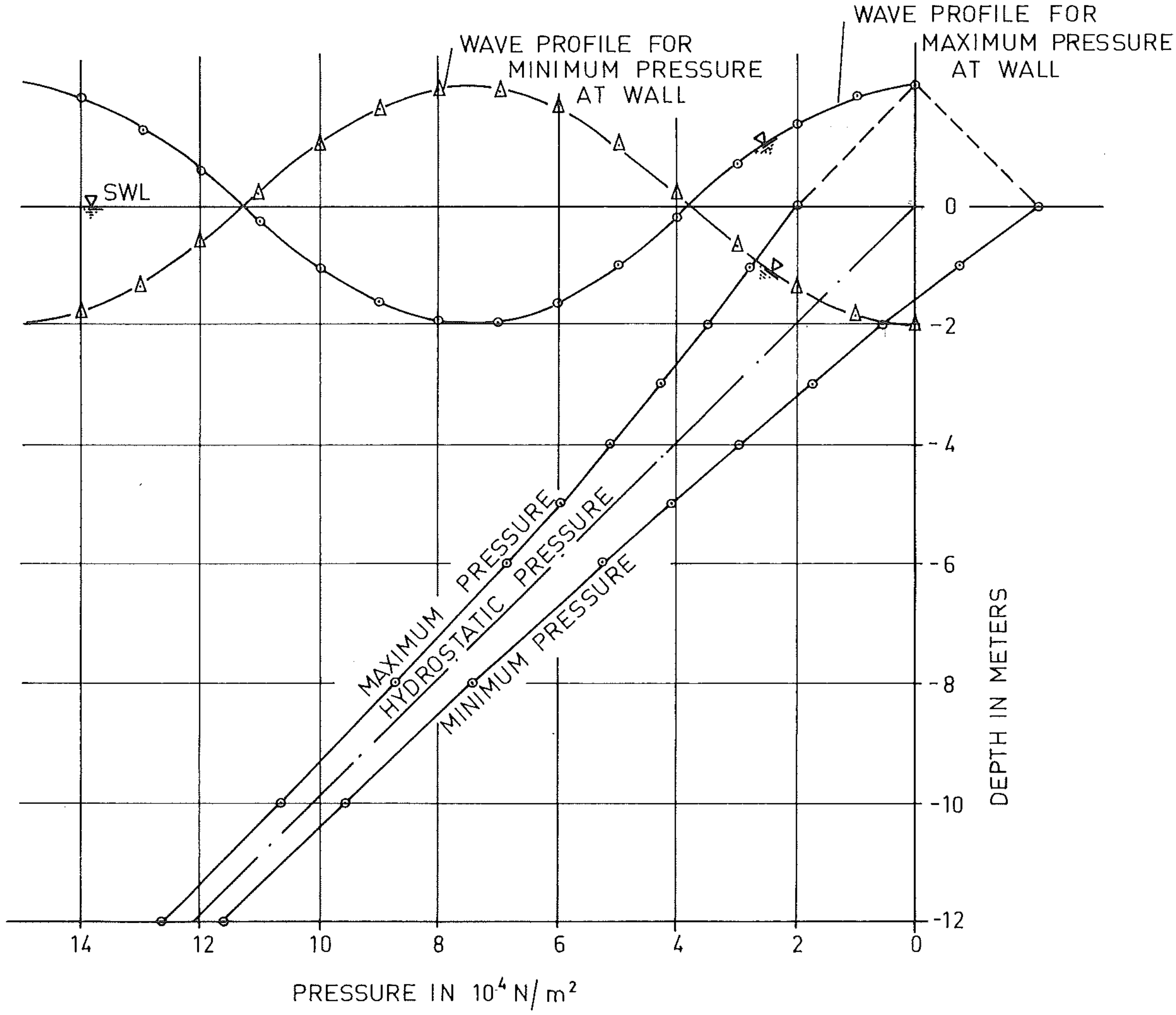

Figure 15.1
PRESSURE DIAGRAM FOR STANDING WAVE
WAVE LENGTH DISTORTION 1 : 2.5

the mean hydrostatic pressure. The period of this pressure fluctuation is the same as the wave period. The magnitude of this dynamic fluctuation is always less than the hydrostatic pressure resulting from a static head equivalent to the on-coming wave height.

Such pressure fluctuations usually do not cause serious difficulties. This is in contrast to the impact forces of breaking waves described in the following section. Other consequences of these standing waves will be discussed in chapter 17.

15.3. Breaking Waves - Impact

Waves breaking against a structure can cause extremely high, short duration, local pressures. (These relative terms will be better defined quantitatively later in this section). What theoretical model is most suited to describe this phenomona?

Continuous water jet

We are well aware of the influence of a continuous water jet impinging perpendicularly on a flat plate. This was the classis example used to illustrate the momentum equation in elementary steady flow fluid mechanics. The resulting pressure on the plate was found to be:

$$p = \tfrac{1}{2} \rho V^2 \tag{15.02}$$

where V is the velocity of the approaching flow.
Equation 15.02 yields pressures considerably less than observed impact pressures, even though the velocity of the approaching flow is the celerity of the wave. This follows from shallow water wave theory, volume I, chapter 5.

Water hammer

A second approach is based upon an assumption that a horizontally oriented block of water having length L hits a rigid wall with velocity V. Continuing the analogy to water hammer in rigid pipelines, a shockwave propagates through the length L at the speed of sound in water, c.
The time during which this occurs is $\delta = L/c$. After this time, δ, the shock wave returns at the same speed to the starting point.
Thus, the total time duration of the impact is:

$$\Delta t = 2\,\delta = 2L/c \tag{15.03}$$

where: c is the velocity of sound in sea water (about 1543 m/s - Sverdrup et al (1942)),
L is the length of the water mass, and
Δt is the total duration of the impact pressure.

This water mass causes a pressure maximum given approximately by:

$$p = \rho\, V\, c \tag{15.04}$$

when this strikes a rigid surface and is contained in a rigid pipe - see Heerema (1974). The rigid surface assumption is not too bad, but the oncoming water mass is certainly not rigidly contained in the directions normal to the flow. Further, no allowance has been made for the effects of air which may either be entrained in the breaking wave or trapped between it and the vertical wall. Führböter (1969) and summarized by Heerema (1974) attempted to correct for these deficiencies in an experimental study. He found that the effective length, L, of the approaching water mass was of the same order as the hydraulic radius, R, of the impact area. This was explained by the fact that sidewards escape of water develops (via a sidewards shock wave) just as fast as the shock wave travels back through the approaching water. The duration of this maximum pressure is then of order

$$\Delta t = \frac{R}{c} \qquad (15.05)$$

where: R is the hydraulic radius of the impact area.
The maximum pressure found was about ten percent of that given by 15.04, above, during his laboratory work. Entrained air and the air cushion between the water mass and the wall tended to increase the values of Δt given by equation 15.05 and decrease the maximum pressures. Führböter explained this by reasoning that the entrapped air must first be compressed before the sidewards shock wave and water escape can be initiated. Thus, the effective length becomes longer.

Another reason for the lower observed maximum pressures is the sidewards escape of the shock wave. Equation 15.04 usually predicts pressures much greater than those experienced in practice. Even when hydroelectric power station penstocks bored through solid rock are considered - a nearly ideal case - the measured water hammer pressures are usually less severe than predicted by equation 15.04.

15.4. Comparative Results

For illustrative purposes we shall let a wide jet of water 1 m thick strike a vertical rigid wall with a velocity of 10 m/s. The hydraulic radius of this jet is 0.5 m; the velocity of sound in sea water is 1543 m/s.

The continuous jet approach (equation 15.02) yields:

$$p = (\tfrac{1}{2})(1030)(10)^2 = 5.5 \times 10^4 \text{ N/m}^2 \qquad (15.06)$$

with

$$\Delta t = \infty \qquad (15.07)$$

The water hammer approach (equations 15.04 and 15.05) yields:

$$p = (1030)(10)(1543) = 1589 \times 10^4 \text{ N/m}^2 \qquad (15.08)$$

and

$$\Delta t = \frac{0.5}{1543} = 0.3 \text{ ms.} \qquad (15.09)$$

Führböter (1969), in contrast, found a value less than 10 % of that in (15.08) and about ten times that in (15.06). The impact duration was about 4 ms.

The largest wave impact forces measured on prototype vertical breakwaters is in the order of magnitude of 100×10^4 N/m^2. One can conclude that these results are not in conflict with each other.

15.5. Other Wave Forces

Two extremes of wave forces have just been described: the clapotis with a period equal to the wave period and and impact force lasting only milliseconds. These two theoretical models are not sufficient to describe the total force on a vertical wall in breaking waves. Prototype measurements carried out on the Haringvliet Sluice gates reported in an anonymous report by the Service of the Delta Works (Nota W-644) and model studies - van de Kreeke (1963)-have shown that additional force components are present.

As might be expected, these additional components lie between the extremes already described, both with regard to period and to magnitude of the total resulting force. For design purposes an "average" loading period in the order of 1 second is often used. Attempts to relate this dynamic force to the wave properties have not yet succeeded.

In the model work reported by van de Kreeke (1963), a model caisson was subjected to a random wind wave having a given significant wave height and period. This yielded a scattering of values of the maximum dynamic force on the model caused by each wave. These values were statistically analyzed; they did not fit any of the usual statistical models used for waves such as the Rayleigh Distribution which one might possibly expect. The force peaks measured having a low frequency of exceedance (less than, say, a few percent) were somewhat higher than the Rayleigh Distribution would predict. This is most likely the influence of the wave impacts already described. *

Not enough is known about the physical background of these forces to make a correct theoretical derivation possible. The best that can be done now is to attempt to evaluate the necessary forces and frequencies of exceedance from model or prototype testing. The importance of these particular forces in design will be pointed out in the following chapter.

15.6. Additional Comments

There is some question about the validity of the model scale laws for this process. Also, it must be remembered that instrumentation used to measure impact forces, must often be rather sophisticated in order to achieve an accurate response to the short duration forces involved.

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* It is important to distinguish between the *local* wave impact *pressures* described in section 15.3 and the *resulting* dynamic *force* on a large caisson being discussed here.

A study of breaking wave impact forces on slopes is being carried out by the Hydraulic Structures Group within the Civil Engineering Department of the Delft University of Technology.

The effects of hydrodynamic forces on a monolithic breakwater are discussed in chapter 16.

16. MONOLITHIC BREAKWATER FOUNDATIONS

E.W. Bijker
W.W. Massie

16.1. Failure Types and Causes

Three specific types of breakwater foundation failure in addition to those associated with rubble mound breakwaters must be considered. Only these three additional possible failures, settling in quicksand, horizontal sliding, and overturning will be discussed in this chapter. Other types of foundation failure such as excessive soil consolidation and foundation soil slip failures, common to other types of structures, will not be discussed in this chapter.

As is pointed out in the following section, quicksand can temporarily result when a short duration impulse load is applied to a soil mass. Such failures are thus caused by the short duration wave impacts described in the previous chapter.

The clapotis forces having a period equal to the wave period (several seconds) act over a sufficiently long time to possibly cause horizontal sliding or overturning of the breakwater. The true impact forces do not act for a long enough time to cause significant displacements - a few decimeters in this case. The shorter period dynamic forces having periods of about one second can also cause horizontal sliding or overturning. The calculation involved for sliding is discussed in section 5 of this chapter; overturning is considered in section 7.

16.2. Types of Foundations

Some indication of foundation types has already been given in chapter 13. The types mentioned there will be discussed in more detail here. Special attention will be paid later in this chapter to dynamic effects resulting from wave impact and other dynamic forces explained in the previous chapter.

Just as with any other structure, the purpose of the foundation is to transmit the necessary static and dynamic loads to the underlying soil layers. When the breakwater is constructed on a very hard clay or rock bottom this purpose is easily fulfilled; the foundation then serves primarily to form a smooth horizontal construction surface.

Unfortunately, not all monolithic breakwaters are founded upon such ideal materials. When the soil does not have sufficient bearing capacity then one alternative is to construct an underlayer of loose material in such a way that the loads are spread over a greater area. See figure 16.1. Such a construction results, in fact, in a composite breakwater.

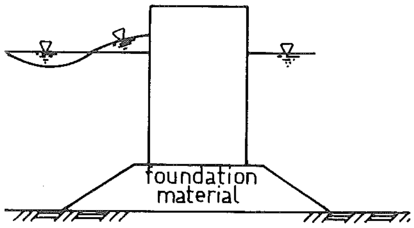


Figure 16.1
COMPOSITE BREAKWATER CROSS SECTION ON MODERATELY STIFF SOIL

The foundation just proposed is adaptable only when the surface of the underlying ground still has a reasonable bearing capacity. When a relatively weak surface layer of limited thickness covers a layer with higher bearing capacity, another solution is to replace the soft layer with higher quality foundation material such as coarse sand or fine gravel. Such soil improvement operations are expensive, especially when the poor quality layer is thick.

A remaining alternative is to construct a pile foundation or to use an open caisson or sheet pile cell type of breakwater. Such breakwaters, although very expensive, can still be the best alternative in deep water or where bottom conditions are too poor for even a rubble mound structure.

Fine sand soils can present some of the most troublesome problems for monolithic breakwater foundations, especially if it is loosely packed. When soil loadings vary very quickly - as a result of wave impact forces, for example - the changing packing of the soil grains decreases the void ratio and results in an excess of pore water which cannot escape during the short time interval involved. This water will not be able to bear the extra load resulting in loss of stability of the soil mass - a quicksand condition. Even though this loss of stability is of short duration, repeated occurrences can - and usually do - lead to failure of the structure involved. If it occurs evenly under the entire structure, the breakwater can sink vertically into the ground.

This phenomonon can be easily observed. Ships washed up on a sandy beach usually experience the deleterious effects of this quicksand condition. Automobiles parked on beaches have experienced the same thing. In this case the varying force between the tires and the sand comes from the infinitesimal vibration of the beach caused by surf in the vicinity.*

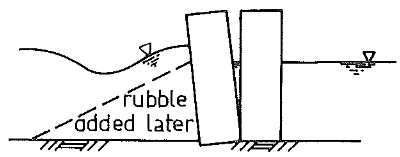


Figure 16.2
CROSS SECTION SHOWING RESUL OF QUICKSAND CONDITION

Uneven settlement of a monolithic structure is most likely. When a through longitudinal joint exists in a vertical breakwater the embarassing condition shown in figure 16.2 may result. This has happened with the Manora breakwater near Karachi, Pakistan. Once such a settlement has taken place here there is little to be done to effect a repair. Placement of rubble against the sagging side, forming a composite breakwater, may prevent further deterioration.

Obviously, this problem can best be avoided. One method is to place a porous but sand-tight filter layer under the monolithic structure. This will lock much like the filter used under a rubble mound breakwater even though this one is built for an *entirely different reason*. Filters under monolithic structures tend to be relatively thick in order to guarantee the water sufficient space to escape - see figure 16.3.

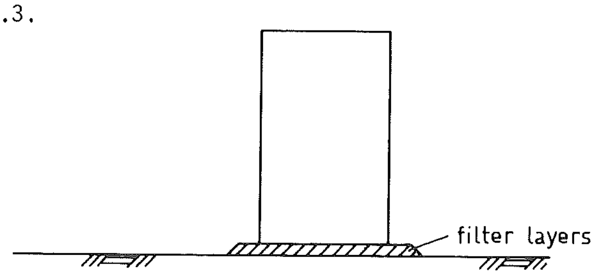


Figure 16.3
FILTER LAYER UNDER MONOLITHIC BREAKWATER

* A cheaper experiment than sacrificing one's auto at the beach is to stamp one's feet on the saturated sand.

Sometimes such a filter is insufficient to provide complete stability-quicksand may still develop in deeper layers. Vertical drainage can then be provided with vertical sand drains consisting of coarse sand.

Still more comprehensive subsoil improvement schemes can be used. Artificial compaction or even grouting can sometimes be worth considering. Such solutions are relatively expensive for common use, however.

Another entirely different approach is to adapt the basic monolithic structure to the difficult soil conditions. This can be done by constructing a more or less conventional pile foundation under the monolithic construction. This has been done in the past, at IJmuiden, The Netherlands, for example, but is rather expensive today.

Another adaption alternative is to construct the breakwater by forming cells of driven steel sheet piles. These cells are then filled and capped. This technique is often used for constructing temporary building pits.

Whatever the foundation chosen, it must be evaluated using all the classical foundation analysis criteria such as total and differential settlement and slip circle analyses. The influence of wave loads on the structure which are transmitted to the foundation is the topic of the following sections.

16.3. Impact Load Response

The "normal" wave loads caused by the pressure fluctuations resulting from a clapotis - see section 15.2 - may usually be treated as static loads on the structure. The foundation analysis is reasonably straightforward.

Wave impact loads described in the previous chapter can, however, cause significant analysis problems. Since the duration of an impact force is not long (a few tenths of a second) relative to the natural period of vibration of the structure, these loads can no longer be treated as static. Inertia effects of movements of the breakwater must be included.

The combination of the breakwater, surrounding water, subsoil and foundation may be schematized as a mass-spring system. The spring is formed by the soil. Although this may not be a nice linear spring, linearity is assumed in the further analysis. The mass consists of the breakwater mass plus an effective (virtual) water mass for those motions which excite water movements (waves). Additionally, the soil forming the spring also has a mass which must be included. A virtual mass of soil is also involved. How large is this virtual soil mass?

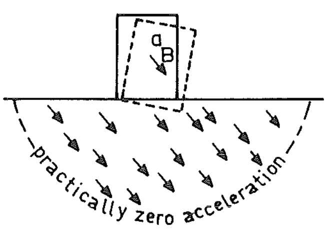


Figure 16.4
SCHEMATIC REPRESENTATION OF SIGNIFICANCE OF EQUATION 16.01

An extremely large mass of soil can be excited by the vibrating breakwater. However, as shown schematically in figure 16.4, the influence of this breakwater motion decreases with distance from the breakwater. In the same way as is done for hydrodynamic forces on piles - see volume IV - the virtual mass is defined as an equivalent mass which would have the same influence as the total soil mass if the virtual mass moved with the breakwater. In equation form:

$$m_s\, a_B = \int_{-\infty}^{-h} \left[\int_{-\infty}^{\infty} \left(\int_{-\infty}^{\infty} \rho_s\, a(x,y,z) dx \right) dy \right] dz \qquad (16.01)$$

where:

$a(x,y,z)$ is the acceleration of the soil at point (x,y,z),
a_B is the acceleration of the breakwater,
m_s is the virtual soil mass, and
ρ_s is the mass density of soil.

The calculations in equation 16.01 can* be carried out for each of the motion component directions. An analogous equation should be used to determine the virtual soil moment of inertia for rotation vibrations.

With this information, the equations of motion of the breakwater can be written. In the following the time-dependent dynamic force, $F(t)$, includes only the wave impact force; the normal wave loads are excluded.

For the vertical component of motion:

$$F_z(t) = (m_B + m_{sz})\ddot{z} + c_z\; z \qquad (16.02)$$

For the horizontal component:

$$F_x(t) = (m_B + m_{sx} + m_w)\ddot{x} + c_x\; x \qquad (16.03)$$

And for rotation about the y axis:

$$M_y(t) = (I_B + I_{sy} + I_{wy})\ddot{\phi} + c_\phi\; \phi \qquad (16.04)$$

where:

m_w is a virtual water mass,
I is a virtual inertia,
c is the spring constant, and
subscripts x, z, ϕ refer to items evaluated in those directions.

Accelerations are denoted in the above equations using the Newtonian notation: $\ddot{z} = \frac{d^2z}{dt^2}$. No damping has been included; this ommission is not serious when only the short term behavior is important and the damping is not too great.

Equations of the form of (16.03) and (16.04) are treated in dynamics of undamped single mass-spring systems; Bouma and Esveld (1976) treat the problem thoroughly. Such dynamic systems have a natural frequency given by:

$$\omega_n = \sqrt{\frac{c}{M}} \qquad (16.05)$$

where: M is the total mass, and
ω_n is the natural frequency

* This is true in theory. The practical execution is often nearly impossible.

Applying this to the horizontal motions (equation 16.03) yields:

$$\omega_{nx} = \sqrt{\frac{c_x}{m_B + m_s + m_w}} \tag{16.06}$$

Further properties of the response are dependent upon the characteristics of the applied force, F(t). For example if F(t) is a block function:

$$\text{for } t \leq 0 \quad : \quad F(t) = 0 \tag{16.07}$$

$$\text{for } 0 < t < t_1 \quad : \quad F(t) = F = \text{constant} \tag{16.08}$$

$$\text{for } t \geq t_1 \quad : \quad F(t) = 0 \tag{16.09}$$

then, again using (16.03) as an example:

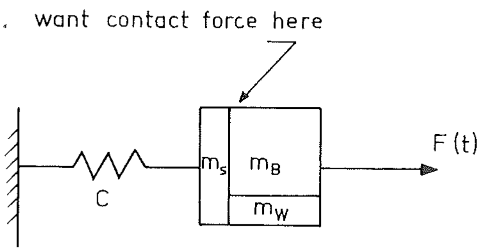


Figure 16.5a
PROBLEM SCHEMATIZATION

$$x = \frac{F}{c_x} \left[\cos\left[\omega_{nx}(t - t_1) \right] - \cos(\omega_{nx} t) \right] \tag{16.10}$$

for $t \geq t_1$

Responses to other types of loads are also given by Bouma and Esveld (1976).

Our primary interest, however, is in the contact force between the breakwater and ground. This can be better visualized using the schematized model shown in figure 16.5.

This contact force can be exposed by separating the mass as shown in figure 16.5b. From that figure, it follows that:

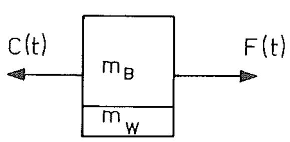


Figure 16.5 b
EXPOSURE OF CONTACT FORCE

$$C_x(t) = F_x(t) - (m_B + m_w)\,\ddot{x} \tag{16.11}$$

By once again restricting F(t) to a block function, we can evaluate (16.11) by substituting the second derivative of (16.10) for $\ddot{x}$:

$$C_x(t) = 0 - (m_B + m_w)\,\frac{F}{c_x}\,\omega_{nx}^2 \left[\cos \omega_{nx} t - \cos\left[\omega_{nx}(t - t_1) \right] \right] \tag{16.12}$$

Since this is valid only for $t \geq t_1$, $F(t) = 0$.* Substituting for ω_{nx} from equation 16.06 yields:

$$C_x(t) = -\frac{m_B + m_w}{m_B + m_w + m_s}\, F \left[\cos \omega_{nx} t - \cos\left[\omega_{nx}(t - t_1) \right] \right] \tag{16.13}$$

This is a nice neat result, but why are we spending so much effort on a block function response? Even though most dynamic loads on breakwaters are not block functions, any loading function can be approximated by the sum of several of these block functions. Since the system has been assumed to be linear, the response (contact force, in this case) will be the sum of the contact forces caused by each of the block functions. This will be illustrated with an example in the next section.

Before proceeding to that example, however, it is useful to examine

* Note: Even though $F(t) = 0$ for $t \geq t_1$ - equation 16.09, $F \neq 0$; it comes from the derivative of equation 16.10.

the response of the breakwater under a few limiting conditions.

When the bottom material is very hard - rock, for example - a stiff spring results, c is large and hence ω_n is large. Since the stiff spring limits displacements and hence accelerations the contact force approaches a value F. One can argue that these contact forces can be carried easily by the hard (rock) soil. This is sometimes not the case, since local stress concentrations can occur in the rock or on its contact surface as a result of, say, jointing in the rock. These locally concentrated stresses can result in local rock failure. The breakwater effectively "grinds" itself slowly into the rock layer.

If, on the other hand, the soil is very soft mud for example, the spring constant, c, is very small and the natural frequency is also small. The virtual soil mass can be large. Under these conditions, the term in brackets in 16.13 approaches zero since the arguments of both cosine terms are nearly zero. Thus, C(t) approaches zero; the applied force is absorbed by momentum changes of the breakwater.

The contact force, C(t), is also strongly dependent upon the duration of the block force, t_1, relative to the natural period, $\frac{2\pi}{\omega_n}$, of the breakwater. For example, when t_1 is equal to the natural period, the two cosine terms in equation 16.13 cancel out and C(t) = 0 for all $t \geq t_1$. Another extreme example occurs when t_1 is one half of the natural period. The maximum value of the term in brackets in equation 16.13 is then two, and C(t) undergoes its maximum variation. In general, the force duration for impact forces on breakwaters will be shorter than the natural period of the construction.

16.4. Example of Impact Response

Consider a single caisson of a monolithic breakwater having dimensions of 15 x 10 x 30 m and a mass of 9×10^6 kg. A wave impact pressure having a maximum value of 5×10^5 N/m^2 acts over an area 1.5 m high and 8 m wide for a total time of 20 ms. The resulting actual and schematized force diagrams are shown in figure 16.6. The soil spring constant is 3×10^{11} N/m and the virtual soil mass is equal to the mass of the breakwater. The virtual water mass is 11 percent of the breakwater mass.

Using (16.06) and the above data:

$$\omega_n = \sqrt{\frac{3 \times 10^{11}}{(1. + 1. + 0.11)(9 \times 10^6)}} = 125.69 \quad \text{rad/sec.} \tag{16.14}$$

or

$$T_n = 50 \text{ ms} \tag{16.15}$$

also,

$$\frac{m_B + m_w}{m_B + m_w + m_s} = \frac{1. + 0.11}{1. + 1. + 0.11} = 0.526 \tag{16.16}$$

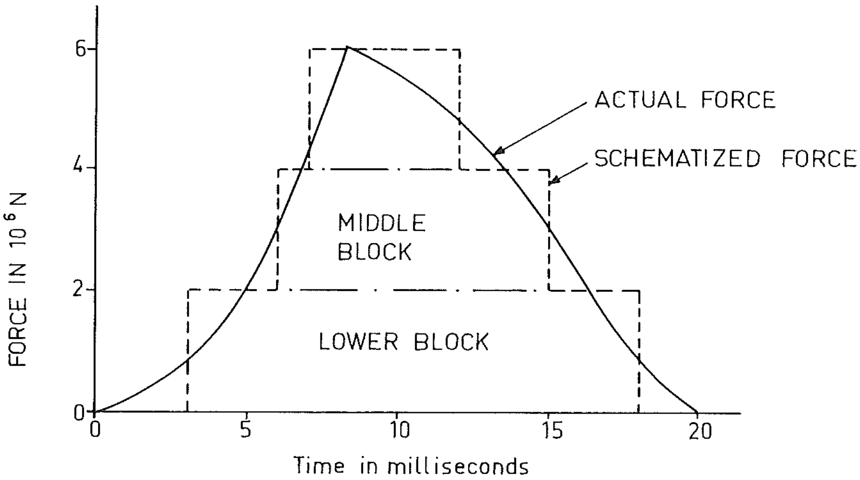


Figure 16.6
ACTUAL AND SCHEMATIZED FORCE DIAGRAM

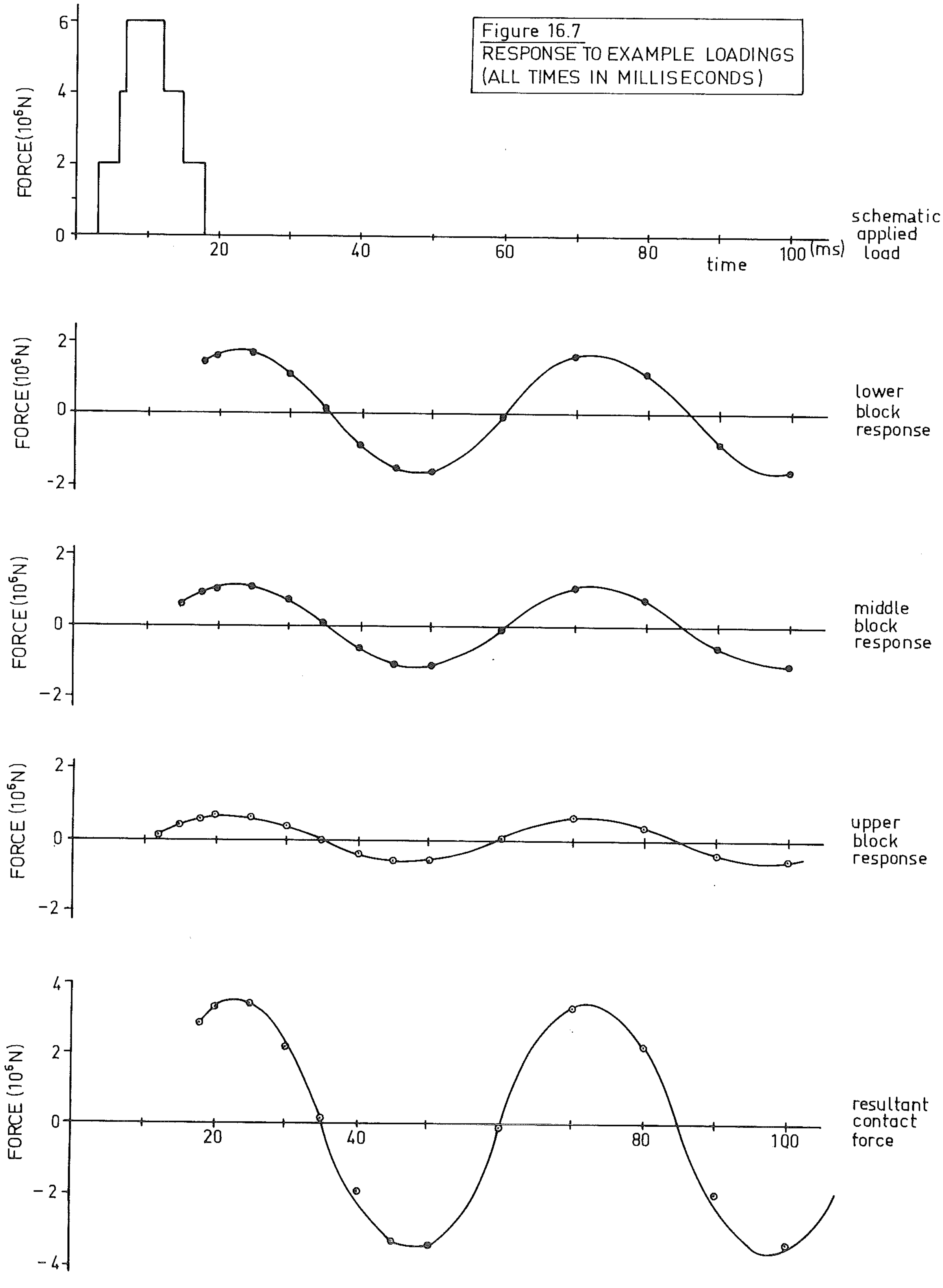
Table 16.1 shows the response computations for each of the three applied schematic block forces. The resulting contact response is ob-

Table 16.1
Response to Schematized Forces

	Lower Block t_1 = 15 ms		Middle Block t_1 = 9 ms		Upper Block t_1 = 5 ms		
Absolute time (ms)	Relative Time (ms)	Contact Force (10^6N)	Relative Time (ms)	Contact Force (10^6N)	Relative Time (ms)	Contact Force (10^6N)	Total Contact Force (10^6N)
0	-	-	-	-	-	-	-
3	0	-	-	-	-	-	-
6	3.	-	0	-	-	-	-
7	4.	-	1.	-	0	-	-
12	9.	-	6.	-	5.	0.20	-
15	12.	-	9.	0.60	8.	0.41	-
18	15.	1.38	12.	0.91	11.	0.57	2.86
20	17.	1.58	14.	1.05	13.	0.63	3.26
25	22.	1.65	19.	1.09	18.	0.60	3.34
30	27.	1.08	24.	0.72	23.	0.35	2.15
35	32.	0.11	29.	0.07	28.	-0.04	0.14
40	37.	-0.91	34.	-0.60	33.	-0.41	-1.92
45	42.	-1.58	39.	-1.05	38.	-0.63	-3.26
50	47.	-1.65	44.	-1.09	43.	-0.60	-3.34
60	57.	-0.10	54.	-0.07	53.	0.04	-0.13
70	67.	1.58	64.	1.05	63.	0.63	3.26
80	77.	1.08	74.	0.72	73.	0.35	2.15
90	87.	-0.92	84.	-0.61	83.	-0.42	-1.95
100	97.	-1.65	94.	-1.09	93.	-0.60	-3.34

tained by adding the responses to each block function at a given time. Table 16.1 and figure 16.7 show the results of such computations. In table 16.1 the absolute time is measured from the start of the rise of the force (t = 0 in fig. 16.6). Equation 16.13, used in computing the contact force components, has a time origin corresponding to the start of each block.

Figure 16.7
RESPONSE TO EXAMPLE LOADINGS
(ALL TIMES IN MILLISECONDS)



16.5. Breakwater Sliding

When horizontal dynamic forces on a vertical monolithic breakwater exceed the horizontal foundation friction force, displacement of the breakwater is inevitable. The object of this section will be to predict this displacement given the loadings.

The force equilibrium for a unit length of vertical caisson resting on a horizontal bottom is shown in figure 16.8. The derivation parallels that of van de Kreeke (1963). In this figure the wave force, F_W, acts horizontally and is assumed to be of form:

$$F_W = \hat{F}_W \sin \omega t \qquad (16.17)$$

where:

$\hat{F}_W$ is the dynamic force amplitude,

t is time, and

ω is the frequency of the loading.

This load frequency is less than that of impact forces treated earlier. It is also much lower than the natural frequency of the structure so that the forces may be considered to be static; the mass-spring analogy used earlier in this chapter can be neglected.

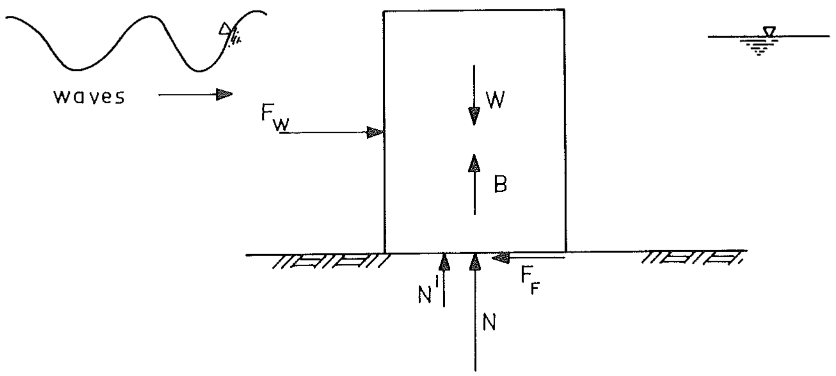


Figure 16.8
FORCES ON BREAKWATER

Since there is no vertical motion of the structure, vertical equilibrium yields:

$$N = W - B - N' \qquad (16.18)$$

where:

B is the bouyant force with (assumed) still water,

N is the resulting upward normal force,

N' is the instantaneaous resultant vertical dynamic force caused by propagation of wave pressures under the structure,

W is the weight of the caisson.

N' is of form:

$$N' = \hat{N}' \sin \omega t \tag{16.19}$$

thus,

$$N' = \varepsilon F_W = \varepsilon \hat{F}_W \sin \omega t \tag{16.20}$$

in which ε is a constant.

The horizontal friction force, F_F, is related to the normal force, N, by the Coulomb friction coefficient as follows:

$$F_F \leq \mu N \tag{16.21}$$

where μ is the friction coefficient.
This friction coefficient is related to the underlying soil properties by:

$$\mu = \tan \phi \tag{16.22}$$

where ϕ is the angle of internal friction of the soil. Examination of forces in the horizontal direction yields:

$$F_W = F_F < \mu N \tag{16.23}$$

if no motion is to take place. The more interesting case with motion is:

$$F_W - \mu N = m_B \frac{dv}{dt} \tag{16.24}$$

where $m_B = W/g$

Substituting 16.20, 16.18, and 16.17 into 16.24 yields:

$$\hat{F}_W \sin \omega t - \mu[W - B - \varepsilon \hat{F}_W \sin \omega t] = m_B \frac{dv}{dt} \tag{16.25}$$

Motion starts when the static friction force is first exceeded, thus when $\frac{dv}{dt} = 0$. Using this fact and a bit of algebra we can solve for the corresponding time t, or equivalently, phases ωt that this occurs:

$$\omega t = \sin^{-1} \left[\frac{W - B}{\hat{F}_W} \left(\frac{\mu}{1 + \mu\varepsilon} \right) \right] \tag{16.26}$$

Call this root ωt_1.

For computational ease, equation 16.25 can be rewritten as:

$$m_B \frac{dv}{dt} = \hat{F}_W (1 + \mu\varepsilon) \sin \omega t - \mu(W - B) \tag{16.27}$$

or:

$$\frac{dv}{dt} = \frac{\hat{F}_W}{m_B} (1 + \mu\varepsilon) \sin \omega t - \mu\left(\frac{W - B}{m_B}\right) \tag{16.28}$$

This can be integrated to determine the velocity at any time $t_2 > t_1$. Doing this:

$$v|_{t=t_2} = \int_{t_1}^{t_2} \frac{\hat{F}_w}{m_B} (1 + \mu\varepsilon) \sin \omega t \, dt - \int_{t_1}^{t_2} \mu \frac{W - B}{m_B} \, dt \qquad (16.29)$$

$$= - \frac{\hat{F}_w}{\omega \, m_B} (1 + \mu\varepsilon) \; [\cos \omega t_2 - \cos \omega t_1] - \mu \frac{(W - B)}{m_B} (t_2 - t_1) \qquad (16.30)$$

The horizontal displacement has an extreme value at a time t_2 such that $v|_{t=t_2} = 0$, thus:

$$\cos \omega t_2 - \cos \omega t_1 + \omega(t_2 - t_1) \sin \omega t_1 = 0 \qquad (16.31)$$

where 16.26 has been substituted. Obviously, $t = t_1$ is also a solution to this equation.

The displacement follows from an integration of equation 16.30. It should be noted that t_1 is a constant in this process.

$$x|_{t=t_2} = \int_{t_1}^{t_2} v(t) \, dt \qquad (16.32)$$

$$= \int_{t_1}^{t_2} - \frac{\hat{F}_w}{\omega \, m_B} (1 + \mu\varepsilon) \; [\cos \omega t - \cos \omega t_1] \, dt - \int_{t_1}^{t_2} \mu \frac{(W - B)}{m_B} (t - t_1) \, dt \qquad (16.33)$$

$$= - \frac{\hat{F}_w}{\omega^2 \, m_B} (1 + \mu\varepsilon) \; [\sin \omega t_2 - \sin \omega t_1] + \frac{\hat{F}_w}{\omega \, m_B} (1 + \mu\varepsilon) \cos \omega t_1 (t_2 - t_1)$$

$$- \mu \frac{W - B}{2 \, m_B} (t_2 - t_1)^2 \qquad (16.34)$$

Using 16.26 in the last term:

$$x|_{t=t_2} = \frac{\hat{F}_w(1 + \mu\varepsilon)}{m_B \, \omega^2} \; [- \sin \omega t_2 + \sin \omega t_1 + \omega(t_2 - t_1) \cos \omega t_1$$

$$- \frac{1}{2} \, \omega^2 (t_2 - t_1)^2 \sin \omega t_1] \qquad (16.35)$$

This is the objective! Now, there remains only a problem of evaluating equation 16.35 in view of the fact that neither t_1 nor t_2 is exactly known. (In a given physical problem, all of the other coefficients are known.) Luckily, t_1 can be solved easily using known parameters in equation 16.26; indeed, this is simply an inverse sine function. Then, given a value for ωt_1, ωt_2 can be solved using equation 16.31. The solution of this non linear equation must be done by trial. A valid but trivial solution is:

$$\omega t_2 = \omega t_1 \tag{16.36}$$

From the physical problem statement and equation 16.17 we can conclude that

$$0 < \omega t_1 < \frac{\pi}{2} \tag{16.37}$$

and

$$\omega t_2 > \frac{\pi}{2} \tag{16.38}$$

Solutions of 16.31 for given values of ωt_1 are listed in table 16.2.

Once the values of ωt_2 are known the terms in brackets in equation 16.35 can be evaluated so that equation 16.35 becomes:

$$x\big|_{t=t_2} = \frac{\hat{F}_w(1 + \mu\varepsilon)}{m_B\,\omega^2}\, f(\omega t_1) \tag{16.39}$$

Values of $f(\omega t_1)$ are included in table 16.2 and are plotted in figure 16.9

Table 16.2 Breakwater sliding parameters

ωt_1 (rad)	ωt_2 (rad)	$f(\omega t_1)$ (-)
0.2	4.7822	3.6014
0.3	4.4407	2.6812
0.4	4.1451	1.9513
0.5	3.8771	1.3802
0.5236	3.8168	1.2658
0.6	3.6276	0.9427
0.7	3.3913	0.6167
0.7854	3.1974	0.4115
0.8	3.1648	0.3823
0.9	2.9458	0.2212
1.00	2.7324	0.1169
1.0472	2.6332	0.0830
1.1	2.5233	0.0545
1.2	2.3176	0.0211
1.3	2.1144	6.0202×10^{-3}
1.4	1.9129	9.5548×10^{-4}
1.5	1.7124	2.8252×10^{-5}
1.5708	1.5708	0.0000

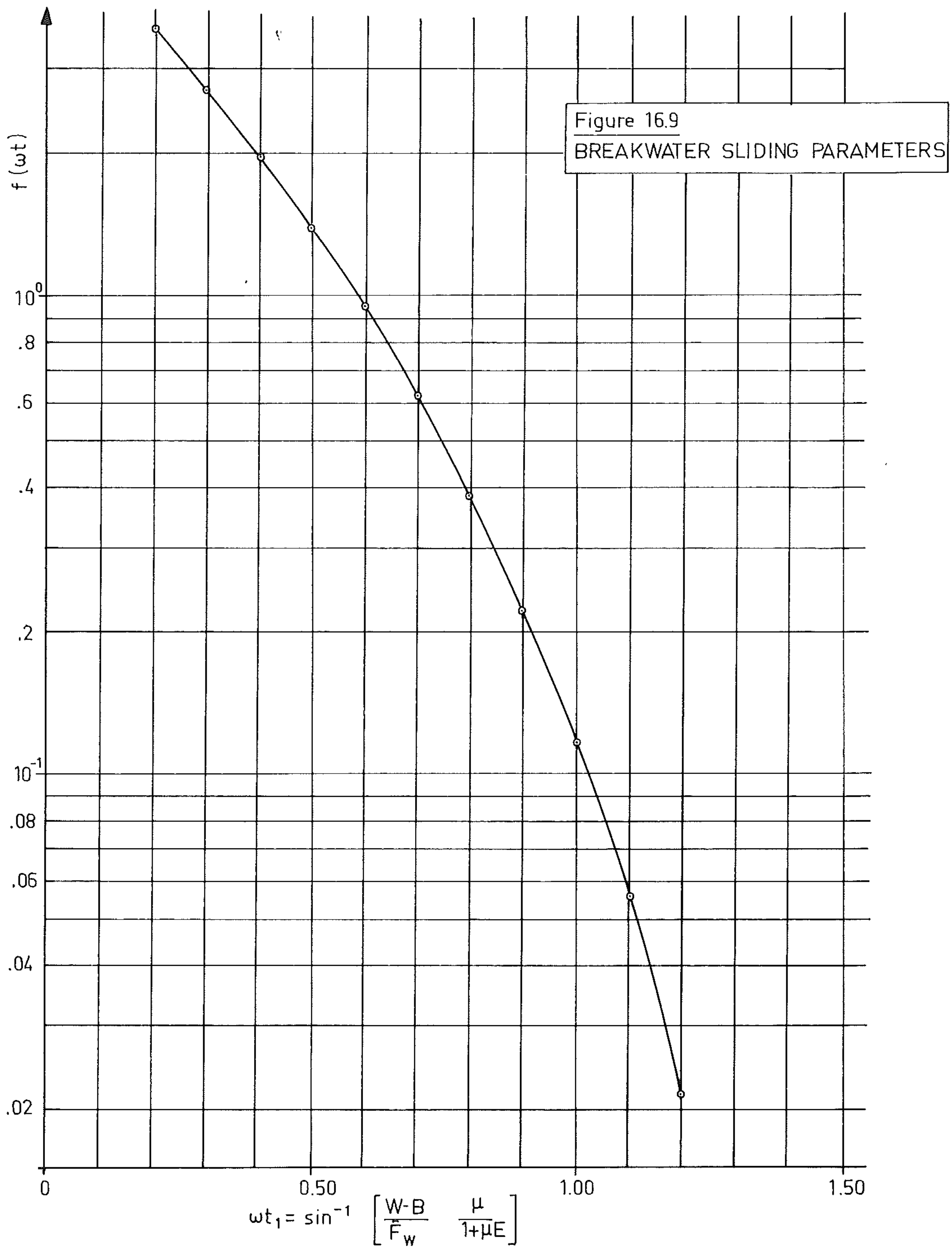


Figure 16.9
BREAKWATER SLIDING PARAMETERS

How does equation 16.39 behave in practice? For a clapotis force, $\hat{F}_w$ is relatively low, and ω is also low. Displacements can be in the order of meters - both positive and negative. Theoretically, equation 16.39 will yield equal positive and negative values so that our breakwater simply dances around its original position. This is, of course, sufficient to be considered a failure, but in practice a permanent displacement will result. This is caused by asymmetry in F_w. Since the clapotis forces are low, it is not difficult to dimension a foundation to resist them.

Wave impact forces are very large and work only in the positive direction; their frequency is very high. The resulting horizontal displacements are of the order of millimeters and are not too serious when compared to the other dynamic forces having periods of, say, one second - see chapter 15.5.

This last type of dynamic forces can cause significant problems. Displacements in the order of decimeters can be expected. Perhaps because these types of forces are not adequately explained and are not yet described theoretically, designers have not considered them in the past. This could be a strong constribution to the seemingly high percentage of failures with monolithic breakwaters.

16.6. Example of Sliding

The following type of problem is one of many that can be attacked using methods described in the previous section.

A caisson 16 m high is to be used to form the initial closure of an estuary. The water depth is 12 m. For a design significant wave of 5.2 m determine the necessary width of the caisson in order to prevent sliding of more than 0.2 m as a result of a single dynamic load cycle having a period of 1 second. The angle of internal friction of the sea bottom is taken as $\mu = \tan\phi = 0.5$.

The coefficient ε can be found from a foundation model for a specific case, but must now be estimated. One plausible idea is to assume that the dynamic wave pressure on the base of the monolith decreases linearly across the width, b, with a maximum at the front lower corner equal to $\frac{F_w}{h}$.

Thus, ε becomes a function of b, namely:

$$\varepsilon = \frac{b}{2h} = \frac{b}{24} \qquad (16.40)$$

when this above assumption is satisfied.

The mass density of the entire caisson is assumed to be 1800 kg/m^3. The amplitude of the applied force, found from model tests, is 1.25×10^6 N/m caisson length.

Work with a unit length of caisson 16 m high with unknown width, b. The following parameters can be evaluated:

$$m_B = (16)(b)(1800) = 2.88 \times 10^4 \text{ b kg/m} \qquad (16.41)$$

$$\frac{\hat{F}_w(1 + \mu\varepsilon)}{m_B \omega^2} = \frac{1.25 \times 10^6(1 + 0.5 \times \frac{b}{24})(1)^2}{2.88 \times 10^4 \times b \times (2\pi)^2} = \frac{1.10}{b} + 2.29 \quad 10^{-2} \qquad (16.42)$$

$$W = m_B g = (2.88 \times 10^4 \times b)(9.81) = 2.83 \times 10^5 \text{ b N/m} \qquad (16.43)$$

$$B = \rho \text{ g b } (12) = (1030)(9.81)(12)(b) = 1.21 \times 10^5 \text{ b N/m} \qquad (16.44)$$

$$\frac{W - B}{\hat{F}_w} \frac{\mu}{1 + \mu\varepsilon} = \frac{(2.83 - 1.21) \times 10^5 \times b}{1.25 \times 10^6} \frac{0.5}{1 + (0.5)(\frac{b}{24})} \qquad (16.45)$$

$$= 6.48 \times 10^{-2} \left(\frac{b}{1 + \frac{b}{48}}\right) \qquad (16.46)$$

Since the parameter evaluated in equation 16.46, used to determine ωt_1, involves b, a direct solution is impossible. A trial and error solution seems practical if not elegant.

Thus, let us initially guess that b = 10 m. Then

$$\omega t_1 = \sin^{-1} 0.536 = 0.566 \qquad (16.47)$$

Using figure 16.9, yields $f(\omega t_1) = 1.08$. This, combined with b = 10 m and equation 16.39 yields;

$$x|_{t=t_2} = \left(\frac{1.10}{10} + 2.29 \times 10^{-2}\right)(1.08) = 0.14 \text{ m} \qquad (16.48)$$

This is too small, since the allowable movement is 0.20 m; b must be reduced. Table 16.3 shows the computation. As shown in the table, a width of 8.8 m is sufficient.

Another interesting question is "How far will this caisson be moved by an 8 second clapotis caused by an individual wave having a 10% chance of exceedance in this design storm?"

The actual oncoming wave height must first be found from the Rayleigh distribution - see volume I chapter 10. Using table 10.1 from that book yields, for 10% exceedance:

$$\frac{H}{H_{sig}} = 1.07 \qquad (16.49)$$

Thus,

$$H = (1.07)(5.2) = 5.56 \text{ m} \qquad (16.50)$$

Its wave length in water 12 m deep follows using its deep water wave length λ_o of:

$$\lambda_o = 1.56 \, T^2 = 100 \text{ m} \qquad (16.51)$$

Table 16.3 Sliding computation

b (m)	ωt_1 (-)	$f(\omega t_1)$ (-)	$x\|_{t=t_2}$ (m)
10.	0.566	1.08	0.14
7	0.407	1.90	0.342
9	0.513	1.30	0.189
8.9	0.508	1.33	0.195
8.8	0.503	1.35	0.200

Using table 6.2 in volume I yields:

$\lambda = 76$ m, thus,

$$kh = \frac{2\pi}{\lambda} h = \frac{(2\pi)(12)}{76} = 0.99 \qquad (16.52)$$

The dynamic clapotis force follows from an integration of the second term of equation 15.01 from z = -12 to z = 0. (The triangular extra pressure will be added later - see figure 15.1).

$$\hat{F}_{w1} = \frac{\rho \; g \; H}{\cosh kh} \int_{-12}^{0} \cosh k(z + h) \; dz \qquad (16.53)$$

$$= \frac{\rho \; g \; H}{k} \left. \frac{\sinh k(z + h)}{\cosh kh} \right|_{-12}^{0} \qquad (16.54)$$

$$= \frac{(1030)(9.81)(5.56)(76)}{2\pi} \frac{\sinh (0.99)}{\cosh (0.99)} \qquad (16.55)$$

$$= 5.16 \times 10^5 \text{ N/m} \qquad (16.56)$$

The pressure above the still water level drops off linearly, we will assume, over the height of the caisson since the standing wave spills over the top. This adds a force of:

$$\hat{F}_{w2} = \frac{1}{2} \rho \; g \; H \; z_c \qquad (16.57)$$

$$= (\tfrac{1}{2})(1030)(9.81)(5.56)(4) = 1.12 \times 10^5 \text{ N/m} \qquad (16.58)$$

yielding a total $\hat{F}_w$ of 6.28×10^5 N/m.
Other parameters remain the same.
Using 16.26:

$$\sin \omega t = \left[\frac{[(16)(8.8)(1800) - (12)(8.8)(1030)] \; 9.81}{6.28 \times 10^5} \; \frac{0.5}{1 + (0.5)(\frac{8.8}{48})} \right]$$

$$= 1.04 \qquad (16.59)$$

Since this is greater than 1.00 no motion can be initiated; the structure is stable.

16.7. Breakwater Rotation

In addition to a possible sliding failure, it is conceivable that a breakwater section may be overturned by a rotation about a corner of its base. As is shown in figure 16.10, an equilibrium of moments is considered at the instant that rotation about point 0 is incipient. The soil supporting force is assumed to act at 0 (a very idealized assumption) and the dynamic vertical wave pressure force, N', is assumed to have a triangular distribution over the base. Equilibrium of moments about point 0, yields:

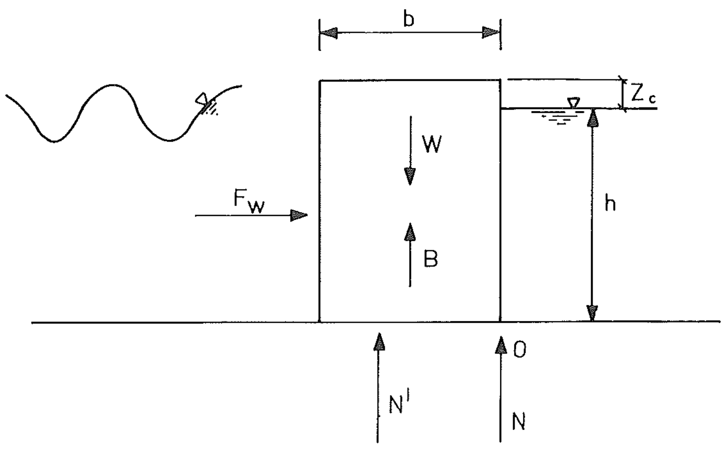


Figure 16.10
FORCES IMPORTANT TO ROTATION

$$(\hat{F}_W \frac{h}{2}) + (N' \frac{2b}{3}) + (B \frac{b}{2}) = W \frac{b}{2} \tag{16.60}$$

where it has been assumed that the wave force, $\hat{F}_W$, acts at an elevation $\frac{h}{2}$ above the bottom. By assuming that the horizontal and vertical dynamic wave pressures are the same at the lower exposed corner of the breakwater, N' can be evaluated in terms of F_W, b, and h:

$$N' = \frac{F_W}{h} \frac{b}{2} \tag{16.61}$$

also:

$$B = \rho \; g \; b \; h \tag{16.62}$$

and

$$W = \rho_B \; (h + z_c)(b) \; g \tag{16.63}$$

Substitution in 16.60 yields:

$$\hat{F}_W \frac{h}{2} + \frac{\hat{F}_W}{h} \frac{b}{2} \frac{2b}{3} + \rho \; g \; b \; h \frac{b}{2} = \rho_B \; g \; b \; (h + z_c) \frac{b}{2} \tag{16.64}$$

In most problems, the water depth, h, and the wave force, $\hat{F}_w$, are known Unknowns are b and z_c, both directly related to the breakwater dimensions.*
The simplest handy solution, then, is to solve for z_c in terms of b:

$$z_c = \frac{1}{\rho_B g} \left[\frac{\hat{F}_w h}{b^2} + \frac{2 \hat{F}_w}{3h} - (\rho_B - \rho) gh \right] \tag{16.65}$$

Alternatively, with a bit of algebra, b can be determined in terms of z_c. An especially simple quadratic equation results in:

$$b = \left[\frac{\hat{F}_w h}{\rho_B g z_c - \frac{2 \hat{F}_w}{3h} + (\rho_B - \rho) gh} \right]^{\frac{1}{2}} \tag{16.66}$$

Obviously the positive root of (16.66) will be the one of interest. Depending upon the problem, either of equations 16.65 or 16.66 may be useful.

16.8. Example of Rotation

Let us check the breakwater used in the sample calculation of section 16.6 against rotation. In other words, find the minimum width required to prevent the breakwater from tipping over. Putting values from that section in equation 16.66 yields:

$$b = \left[\frac{(1.25 \times 10^6)(12)}{(1800)(9.81)(4) - \frac{2(1.25 \times 10^6)}{(3)(12)} + (1800 - 1030)(9.81)(12)} \right]^{\frac{1}{2}} \tag{16.67}$$

$$= \left[\frac{1.50 \times 10^7}{7.06 \times 10^4 - 6.94 \times 10^4 + 9.06 \times 10^4} \right]^{\frac{1}{2}} \tag{16.68}$$

$$= (163)^{\frac{1}{2}} = 12.8 \text{ m} \tag{16.69}$$

This is wider than was required to prevent sliding.

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* Within rather narrow practical limits, the density of the breakwater, ρ_B, may also be varied.

17. INFLUENCE OF BREAKWATER ON WAVES

E.W. Bijker

17.1. Introduction

In the previous two chapters the effects of waves on monolithic breakwaters and their foundations have been discussed in detail. Here, we shall examine the influence which the breakwater has on the nearby wave patterns and bottom morphology. In principle each phenomenon discussed in the following sections occurs for both monolithic and rubble mound breakwaters. Usually, since the phenomena depend upon wave reflection they are most pronounced near vertical monolithic breakwaters.

17.2. Standing Waves

One may remember from short wave theory that the resultant of an incident and directly reflected travelling wave is a standing wave. Since the reflection is greatest from vertical smooth monolithic breakwaters, standing wave problems are most often found near these structures. What are the standing wave problems?

Since the wave height of the standing wave is twice as much as that of the incident wave, these waves can make for pretty choppy going for smaller ships approaching a harbor entrance or navigating within the harbor near a reflecting breakwater exposed to sea waves. For this reason, it is often rewarding to avoid the construction of vertical reflecting walls (breakwaters or quays) where sea waves penetrate into the harbor.

Standing cross waves can form in narrow canals and harbor basins having reflecting surfaces on both sides. The effect can be appreciable when the width and depth of the basin enhances a reasonance - see volume I chapter 19.

When longer waves such as swell and tidal components are involved, both rubble mound and monolithic structures are effective reflectors. The resulting seiches can cause problems for both cargo handling and ship moorings. These topics are discussed more fully in volume II.

17.3. Local Morphological Changes

In areas where short standing waves are found (near vertical breakwaters) the water motions are essentially different from those under a travelling wave. Therefore, morphological changes in an erodable bottom can be expected. Figure 17.1 shows the mass transport under a standing wave as well as the expected bottom changes. These results were obtained in a model study carried out by de Best (1971) and Wichers (1972), and are also reported in de Best, Bijker, Wichers (1971).

As is shown in figure 17.1a, coarse material moves as bedload; the resultant of forces on the grains tends to move them toward the nodes resulting in deposition there. Since there is little water motion near the antinodes, the bottom remains stable there. Erosion is most severe midway between the nodes and antinodes.

A different pattern develops with fine sand which is transported largely in suspension. Erosion takes place at the nodes where bottom velocities are high and material is deposited near the antinodes where the bottom water is relatively quiet and the mass transports converge.

What bottom protection is required? In order to protect the toe of the breakwater regardless of the soil grain size, the revetment should extend at least 3/8 of a wave length before the breakwater. When one is certain that all of the bottom material is relatively coarse, this revetment may be a bit shorter. This can be dangerous, however, since the more severe waves can still cause suspended transport of even coarse material giving a bed form as shown in figure 17.1 b for fine sand.

Aspects of the construction of monolithic breakwaters will be discussed in the following chapter.

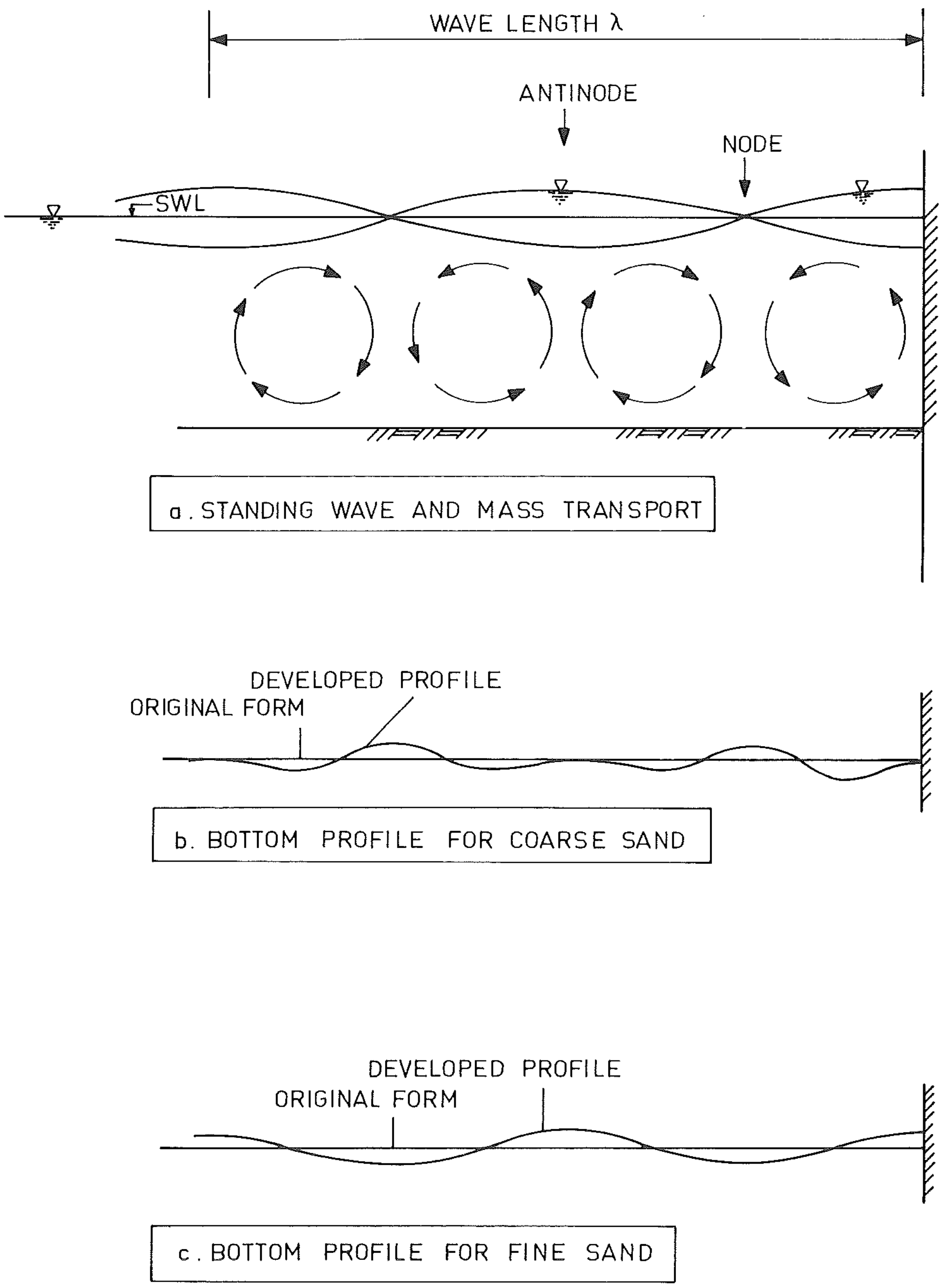

Figure 17.1

STANDING WAVE AND RESULTING BOTTOM CHANGES

18.1 CONSTRUCTION OF MONOLITHIC BREAKWATERS

18.1. Introduction

Just as with rubble mound breakwaters, the method of construction can influence the design of a monolithic breakwater. The construction methods described in the following sections will apply to the construction of only the massive monolithic part of the structure. Construction methods for bottom preparation - laying filters - are essentially the same as for rubble mound breakwaters; one is referred to chapter 10. The only exception to this remark would be the use of a separate pile foundation for a monolithic top construction. This is no longer common practice, however, and will not be discussed here except incidentally in section 18.4.

18.2 Construction Over Crest

One of the principal methods of placing the large elements of a monolithic breakwater is to set them in place using a special crane mounted on the crest of the already completed breakwater. Advantages of this method are that the entire construction activity is concentrated on one site near the breakwater and this method is the most independent of the sea conditions; elements can be placed - perhaps not so easily - even in rather bad weather. Also, the use of a rigidly mounted crane increases the placement precision of the work; the elements can be joined neatly without too much difficulty.

Among the disadvantages of the method are that construction progresses rather slowly and large monolithic units must be moved overland. Also, large and specialized construction equipment is needed.

The construction elements have many forms, but usually have a mass of a few hundred tons. Concrete is the almost universal building material. Early breakwaters were built up of massive blocks piled upon each other such as was done at Algiers, Morocco in 1927. As is shown in figure 18.1, the elements were locked together by their shape and by a concrete key cast after placement. The superstructure was also cast in place.*

In more recent times it has been more common to place elements which extend over the full height of the structure in one unit. This has been done, for example, at Hanstholm, Denmark. Figure 18.2 shows the elements and crane used for a secondary breakwater there while figure 18.3 shows a similar plan for the outer breakwater. Here, the elements were more circular in plan in order to reduce wave forces. Figure 18.4 shows a plan of the construction yard. The project is more completely described by Elbro (1964). In contrast to the earlier described elements, these construction elements were hollow concrete boxes which were allowed to fill with water during placement and were later filled with sand. This makes it possible to place a larger unit with a crane of limited capacity.

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* It is interesting to note that this breakwater failed; considerable effort was invested investigating the failure since the design has been "perfect".

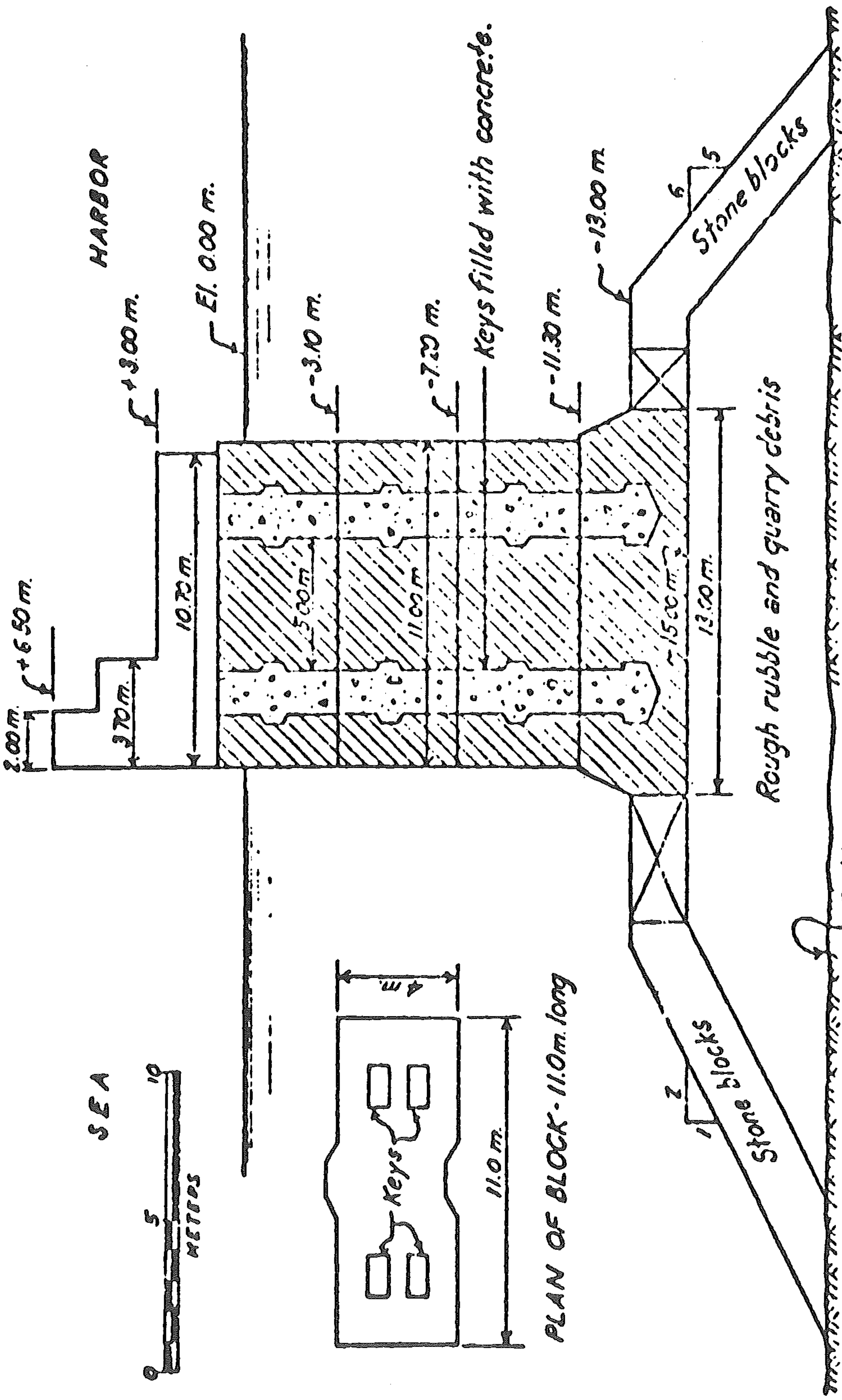

Figure 18.1
BREAKWATER FROM ALGIERS, MOROCCO

Since the keying joints of these units slide along each other during placement, special precautions must be taken to prevent damage to the keys. The usual method is to face the contact surfaces with hardwood - greenhart is excellent. This is shown in cross-section c-c of figure 18.2.

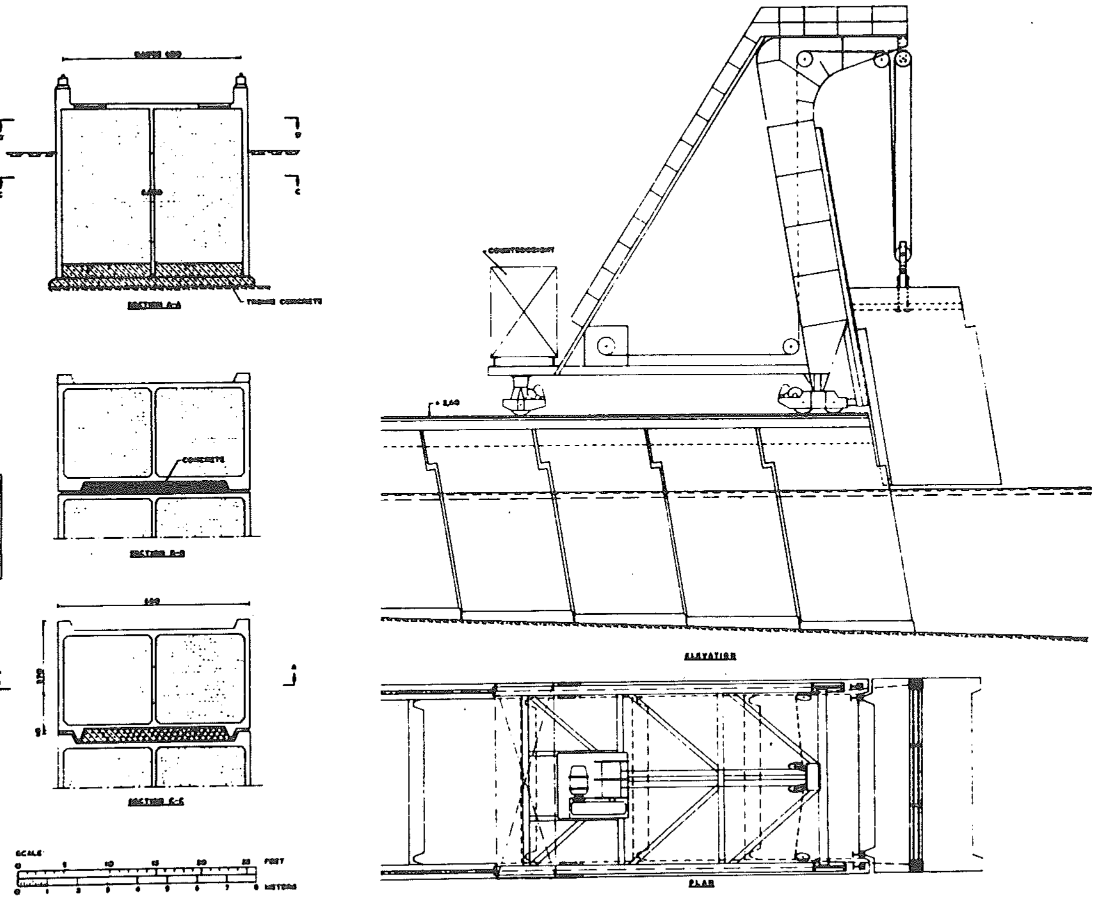

Figure 18.2
ELEMENTS AND CRANE FOR SECONDARY BREAKWATER

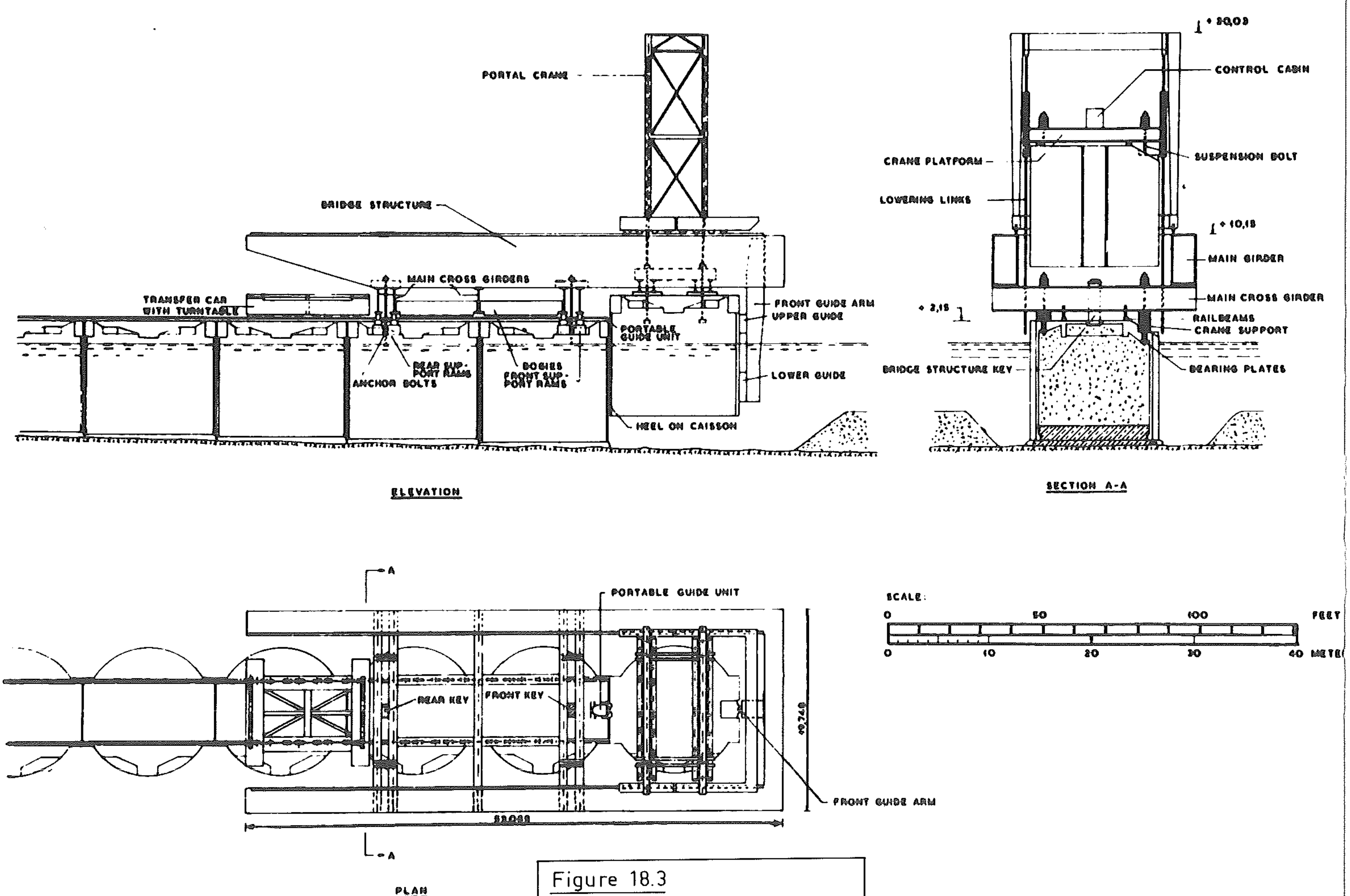

Figure 18.3
CONSTRUCTION CONSISTING OF CYLINDRICAL CAISSONS

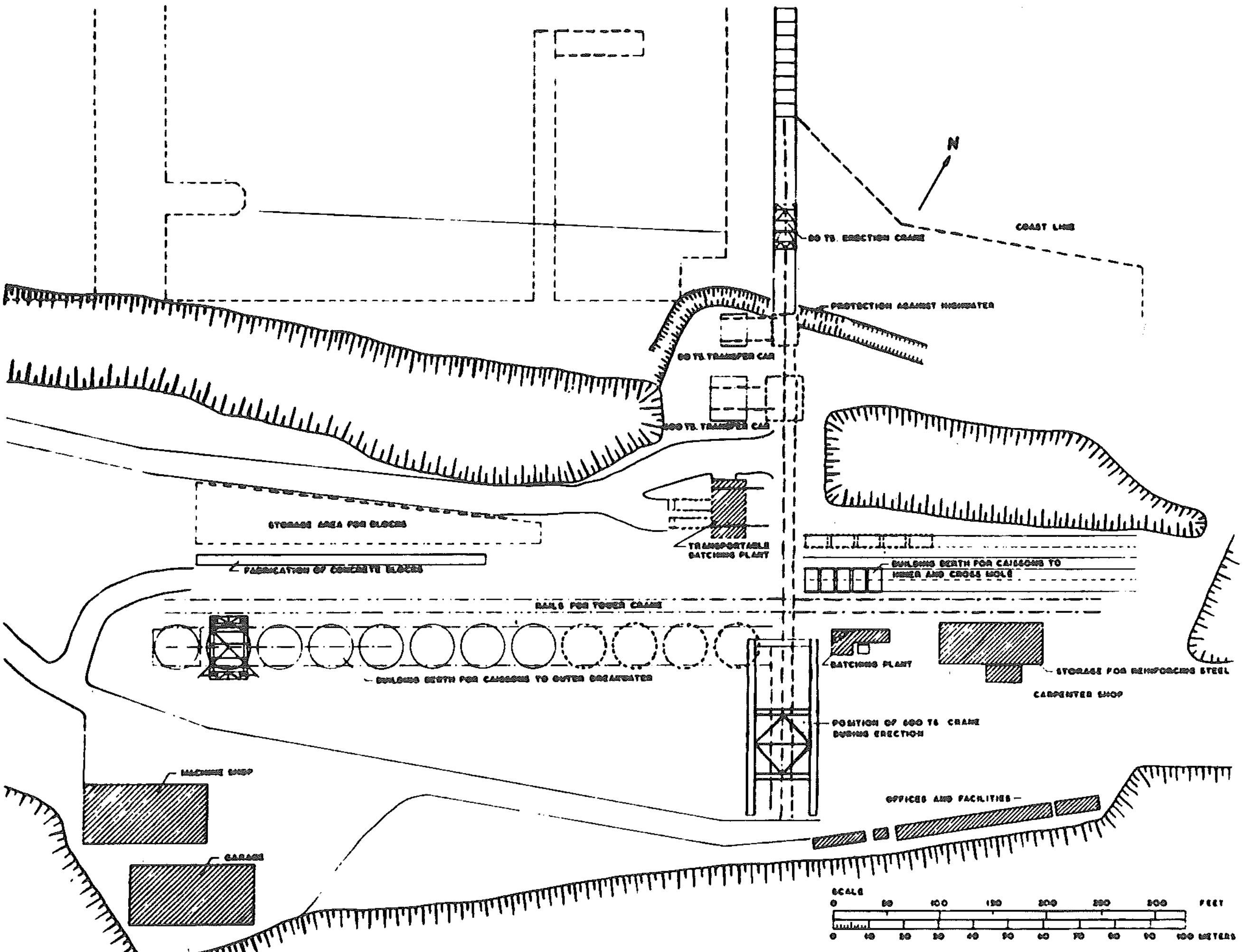

Figure 18.4
PLAN OF CONSTRUCTION YARD

18.3. Use of Floating Caissons

In order to avoid the problems of overland movement of heavy construction elements and shorten the working time at the construction site, large caissons or even old ships* are sometimes floated into position and sunk. This technique was used to construct the beachheads on the French coast in World War II. Additionally, this method of construction is well suited to deep water for which elements placed over the crest would become too heavy. Structures to be moved over water are more often limited by available water depth rather than total mass - consider the heavy structures for the offshore oil industry - see volume I chapter 32.

A separate construction site is now needed to fabricate the caissons but little major specialized equipment is needed. The accurate placement of the caissons can present a problem. For temporary structures or those which will soon be concealed - the closure of an estuary or the

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* In the north polar sea icebergs are sometimes towed near shore and sunk by adding ice on top of them in order to form a breakwater.

core of a composite breakwater, for example - precise alignment is not so important. For permanently exposed structures careful aligment is needed in order to assure that the key constructions transmit loads effectively between the adjacent caissons. Location of the caissons using only tug boats is usually insufficient for permanently exposed structures. They are certainly adequate for the placement of other caissons such as that forming the initial closure of the Brouwershavense Gat in The Netherlands.When placement accuracy is more critical an auxillary temporary caisson can provide the necessary additional guidance - see chapter 20.

18.4. Construction in Place

Occasionally, usually when soil conditions are too poor to support a concentrated surface load, cells are made from driven sheet pile. These interconnected cells are then filled and capped to complete the breakwater. Generally, the driving accuracy required makes it advantageous to drive the sheet piles using a driving rig situated on the crest of the completed portion of the breakwater. Care must be taken that the uncompleted cells are not severly damaged in a storm; completed cells derive much of their strength and stability from the pressure of the internal fill material.

Information presented in this and the previous five chapters will be combined in an optimum design in chapter 19.

19. OPTIMUM DESIGN

W.W. Massie
A. Paape

19.1. Introduction

The objective of this chapter is to use the information presented in the previous six chapters in order to make an economically optimum design of a vertical monolithic breakwater. In order to make a comparison with rubble mound breakwaters possible, an attempt will be made to design a monolithic breakwater for the same problem treated in chapter 11.

The general discussion included in sections 1 and 2 of that chapter is also valid for a monolithic breakwater; it will not be repeated here. However, important additional information will be presented in the following sections.

19.2. Design Data

While all of the data presented in chapter 11 remains valid for the problem at hand, the data presented there must be supplemented for the present problem. The additional information will be presented here; for completeness other strongly related data will be repeated from chapter 11.

Storm conditions

In addition to the data presented in table 11.1, data on the number of waves in the individual storms will also be needed. Table 19.1 repeats table 11.1 and adds this additional necessary data.

Data on the frequency of occurrence of short period dynamic loads is presented in figure 19.1. This curve represents the results of model and prototype tests carried out with various wave and water level conditions.

TABLE 19.1 Storm Data

Recurrence Interval (yrs)	H_{sig_0} (m)	Period T (s)	No. of Waves N (-)	Water level h' (m)
0.1	4.5	7.4	3000	
0.5	5.5	9	2500	
1.0	6.0	10	2000	3.2
5	7.0	11	1000	
20	8.0	12	1000	
100	9.0	13	800	4.6

Cost of materials

The cost data provided in table 11.2 must be augmented. Further, since the monolith will be fabricated from concrete elements, costs of armor stone are no longer relevant. Table 19.2 gives the relative cost figures necessary for this design.

All other data remains as presented in chapter 11 section 3.

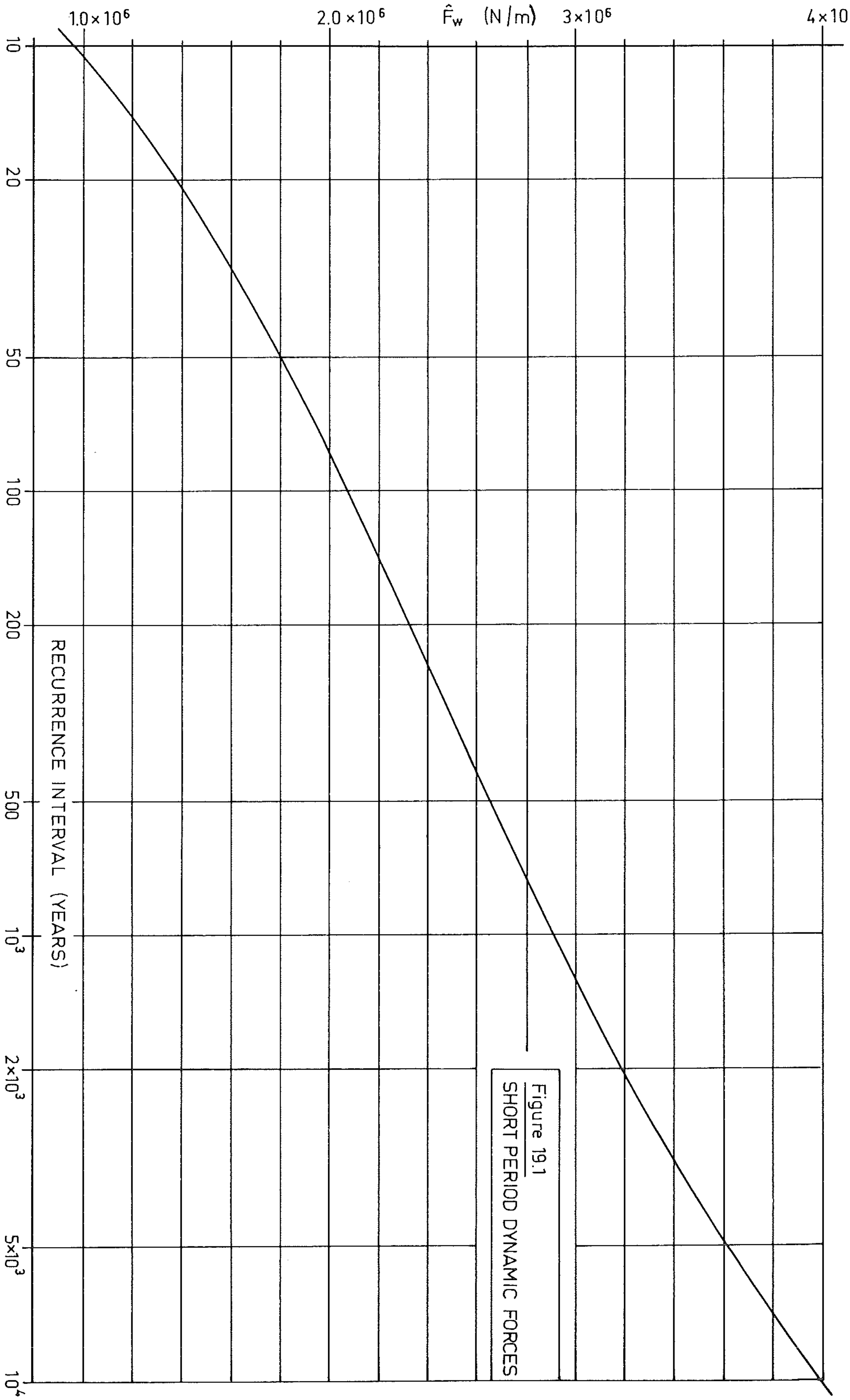

Figure 19.1
SHORT PERIOD DYNAMIC FORCES

TABLE 19.2 Costs of Materials in Place

Material	Use	Unit	Placement Method barge dumped	Over Crest
Sand	caisson fill	m^3	6*	10
Gravel	filter layer	m^3	40	50
Concrete	Caissons	m^3	350**	-
(ρ = 2400 kg/m^3)	Large Elements	m^3	-	400
	cap Const.	m^3	-	150

19.3. Preliminary Computations

Since the face of this breakwater is to be vertical, we can expect a standing wave to form before it. Also, since the breaking criteria for standing waves differ from those for travelling waves, the wave breaking computations of section 11.4 must be revised.

From Wiegel (1964), the appropriate breaking criterium for standing waves is:

$$H_x = 0.109 \lambda \tanh kh \qquad (19.01)$$

where:

H_x is the maximum progressive wave component,
k is the wave number = $2\pi/\lambda$,
h is the water depth, and
λ is the wave length.

It should be noted that the height of the standing wave at the breakwater will be twice the value of H_x provided that the wall is at least as high (there is no overtopping).

The necessary computations and extrapolations are carried out in table 19.3 which parallels the work presented in table 11.3. In table 19.3 values of λ are computed from the water depth and the values of h/λ taken from the *Shore Protection Manual* tables. Values of H_x follow from equation 19.01. We see that the standing wave breaking criterium is never a governing factor for the significant wave, since the higher of these break from shoaling long before reaching the breakwater. As can be seen by comparison of the two tables mentioned, the results for H_{sig} are identical. Figure 11.2 can still be used.

The wave computations are not yet completed, however. The clapotis force, one of the design loads, results from a single wave in a storm and can not be related, therefore, to only a significant wave height. The frequency of exceedance of various individual design wave heights is needed now. This computation is the same as that shown in chapter

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* price for sand from hydraulic dredge pipeline.

** price for completed floating caisson sunk in position.

TABLE 19.3 Wave Computations

Recurrence Interval (yrs)	H_{sig_o} (m)	T (sec)	h' (m)	Wave length λ_o (m)	Total depth h (m)	h/λ_o (-)	h/λ (-)	λ (m)	H_x (m)	$\frac{H}{H_o}$	H_{sig} (m)	Note	$P(H_{sig})$ $(\frac{\text{storms}}{\text{year}})$	No. of Waves N (-)
0.1	4.5	7.4	2.8	85.	12.8	0.1506	0.1838	70.	6.2	0.9133	4.1	(1)	10	3000
0.5	5.5	9	3.0	126.	13.0	0.1028	0.1434	91.	7.0	0.9308	5.1	(1)	2	2500
1	6.0	10	3.2	156.	13.2	0.0845	0.1273	104.	7.5	0.9487	5.7	(1)	1	2000
5	7.0	11	3.7	189.	13.7	0.0725	0.1163	118	8.0	0.9667	6.7	(2)	0.2	1000
10	7.5	11.5	3.9	207.	13.9	0.0673	0.1114	125.	8.2	0.9766	6.8	(2)	0.1	1000
20	8.0	12	4.2	225.	14.2	0.0631	0.1074	132.	8.5	0.9858	7.0	(2)	0.05	1000
50	8.5	12.5	4.4	244.	14.4	0.0590	0.1033	139.	8.7	0.9958	7.1	(2)	0.02	900
100	9.0	13	4.6	264.	14.6	0.0553	0.0996	147.	8.9	1.006	7.2	(2)	0.01	800
500	10.0	14	5.1	306.	15.1	0.0493	0.0934	162.	9.3	1.025	7.4	(2)	0.002	600
1000	10.5	15	5.3	351.	15.3	0.0436	0.0873	175.	9.5	1.048	7.5	(2)	0.001	500
5000	11.5	16	5.8	399.	15.8	0.0396	0.0827	191.	9.9	1.066	7.7	(2)	0.0002	500

Notes: (1) Significant wave not broken, larger waves break at breakwater

(2) Significant wave broken by shoaling before reaching breakwater; larger waves break at breakwater.

11 of volume I except that the annual probability of exceedance of our design wave will be $1-E_3$ using the notation of that chapter. This corresponds to equation 11.16 in volume I with a life, ℓ, of one year.

A calculation such as described in that chapter must be repeated for each of a whole series of chosen design wave heights. A sample of such a calculation for H_d = 8.0 m is shown in table 19.4. The breaking criteria influence the computation, however.

TABLE 19.4 Statistical Calculation for H_d = 8.0 m

H_{sig} (m)	P exceedance $(\frac{storm}{year})$	$P(H_{sig})$ $(\frac{storm}{year})$	Char. H_{sig} (m)	N $(\frac{waves}{storm})$	H_x (m)	H_d/H_{sig} (-)	* $P(H_o)$ (-)	E_1 (-)	E_{2i}
4.1	10								
		8.	4.5	3000	6.8	-	0.0	0.0	0.0
5.1	2								
		1.	5.4	2000	7.3	-	0.0	0.0	0.0
5.7	1								
		0.8	6.1	1500	7.7	-	0.0	0.0	0.0
6.7	0.2								
		0.15	6.8	1000	8.2	1.18	6.28×10^{-2}	1.00	1.50×10^{-1}
7.0	0.05								
		0.04	7.1	900	8.7	1.13	7.89×10^{-2}	1.00	4.00×10^{-2}
7.2	0.01								
		0.008	7.3	700	9.1	1.10	9.05×10^{-2}	1.00	8.00×10^{-3}
7.4	0.002								
		0.0018	7.5	500	9.5	1.07	1.03×10^{-1}	1.00	1.80×10^{-3}
7.7	0.0002								

$E_{2i} = P(H_{sig}) * E_1$
$= P(H_{sig})$

$$P(H > H_D) = 1.92 \times 10^{-1}$$

In column 6 of table 19.4 values of H_x, the maximum possible individual wave height, corresponding to H_{sig} in column 4 are listed. These H_x values are interpolated from values in table 19.3. Obviously, since H_x is the maximum wave that will not break, the chance of H_d occurring in a storm in which $H_x < H_d$ must be zero, irrespective of the Rayleigh distribution. (It is quietly assumed that up until breaking of individual waves occurs, the Rayleigh distribution can still be used. This assumption is reasonably supported in the literature - Battjes (1974)). Thus, we can conclude that $P(H_d) = 0$ for $H_d > H_x$; this is shown in column 8. The non-zero values in that column follow from the Rayleigh distribution:

* $P(H_D) = 0$ when $H_D > H_x$ (table 19.3) corresponding to the other conditions in the row.

$$P(H_d) = e^{-2\left(\frac{H_d}{H_{sig}}\right)^2} \qquad (19.02)$$

Since the ratios $\frac{H_d}{H_{sig}}$ are never extremely large, the chance that H_d occurs in a storm consisting of N wave characterized by H_{sig}:

$$E_1 = 1 - [\,1 - P(H_d)]^N \qquad (19.03)$$

is one. Occurrence of the wave at least once in the storm is guaranteed, and the chance that both the wave *and* the storm occur is the same as the chance of the storm alone. Indeed, values of E_{2i} in table 19.4 are identically equal to values of $P(H_{sig})$. This also results in the fact that

$$P(H > H_d) = 1 - \left[\prod_{i=1}^{4} (1 - E_{2i})\right] \qquad (19.04)$$

$$= 0.192$$

for the data in table 19.4. This value is the same as (within reasonable computational accuracy) the chance of exceedance of the lowest H_{sig} in which $H_x \geq H_d$. That chance is 0.2 in this example corresponding to H_{sig} = 6.7 m.

The conclusion of this is that *for this problem with breaking waves,* computations such as just outlined are unnecessary and *in this special case* the frequency of exceedance of a given design wave H_d is the same as the frequency of exceedance of H_{sig} corresponding to $H_d = H_x$ in table 19.3. Figure 19.2 can then be obtained by plotting H_x versus $P(H_{sig})$ from table 19.3.

19.4. Optimization Variables and Philosophy

The breakwater elements to be placed must be dimensioned to withstand loads which can lead to various types of failures; see section 16.1. Both the applied loads and the ability to resist these loads are related to the dimensions (crest elevation and width) of the structure. This interrelationship of structure dimensions and applied loads as well as the diversity of applied loads makes an optimization computation somewhat more complex than for a rubble mound structure. While the construction costs remain easy to determine, annual damage costs will be more difficult.

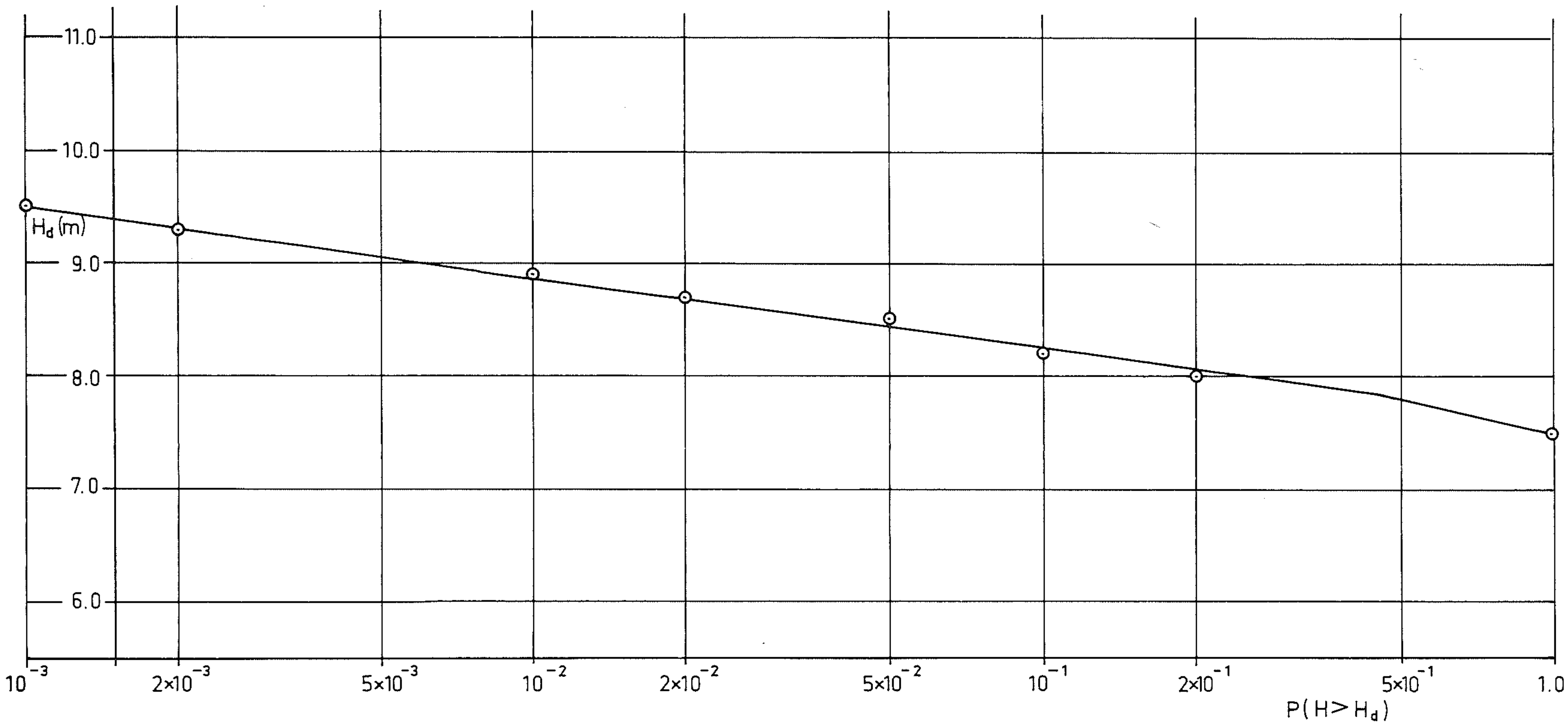
In contrast to the damage to rubble mound breakwaters, damage to a monolithic structure can result from several somewhat independent sources: clapotis, impact forces, or short period dynamic forces.

The effects of impact forces, the formation of quicksand, are essentially impossible to predict. Therefore, in order to assure that problems will not occur a reasonably thick porous filter layer will be placed on the sand bottom to support the monolithic elements.*

* Possible subterranean failure is not being considered - see chapter 16.

Figure 19.2

DESIGN WAVE HEIGHT AS FUNCTION OF ANNUAL FREQUENCY OF EXCEEDANCE



A thickness in the order of 1.5 m should be sufficient. This eliminates one source of possible damage.

Since the short period dynamic forces can cause either a sliding or tipping failure of the breakwater, these forces must be used in both criteria. For a given design, however, failure will occur either by sliding or tipping and not, in general, by both forms simultaneously. The condition (sliding or tipping) which happens to be important in a given design will be that which occurs with the lower applied force, $\hat{F}_w$.

Failure will be considered to have occurred when any of the following occur:

a. Tipping is initiated by a short period dynamic force,
b. A displacement of more than a small amount - say 0.1 m - results from the same force as in a.
c. Any displacement occurs caused by a clapotis.

Since failures a and b are caused by the same force, they are mutally exclusive; occurrence of either one will prevent the other. On the other hand, the clapotis force - c, above - is independent of a or b. This will have consequences for the statistics in section 19.7.

What optimization parameters are available? Since not all failure conditions can be related to a single characterizing wave - as was done for a rubble mound breakwater - another parameter must be chosen. The simplest parameter, then, is related to the weight of the breakwater since this plays an important role in its stability. Unfortunately, weight, itself, is not sufficient, since the stability of the design depends also upon its geometry. Indeed, two parameters must now be optimized: the height and width of the proposed design; within certain limits, these two parameters can be varied independently.

Obviously, the *minimum* height of the breakwater is determined by other considerations such as visibility to mariners or overtopping. However, since the weight of the breakwater increases with increasing height, it can be economical to construct the breakwater with a crest somewhat higher than would otherwise be needed.

Further, a very high narrow breakwater would be uneconomical just as would be a very wide low one. However, except for these limitations, the width, b, and crest elevation, z_c, of the breakwater are completely independent variables. The optimization must be carried out using both variables; this can most easily be done by fixing one value - the height - and then varying the other - the width. This process will be repeated using various heights.

19.5. Minimum Crest Elevation

The overtopping criterium used in chapter 11 will be used here in a somewhat adapted way. In fact the largest waves of the irregular wave field, as used in the calculations of chapter 11, produce slight overtopping. The maximum wave height criterium of standing waves as applied in section 19.3 gives formally an absolute maximum of the wave height irrespective of the irregular wave field. In the case of a vertical breakwater it seems logical to take a less stringent overtopping criterium as in the case of a rubble mound one. In chapter 11 overtopping was allowed to occur no more than 5 times per year; in our case we can allow 10 times per year. Thus our breakwater must now be at least high enough to reflect the heighest wave to be expected with that frequency. Using table 19.3 directly yields a design approaching wave height of 6.2 m with a water level 2.8 m above MSL. Since the reflected standing wave will be twice as high as the approaching wave, the breakwater crest must be 6.2 m higher than the still water level; the minimum crest elevation is thus 6.2 + 2.8 = 9.0 m above a MSL datum. This is somewhat higher than was needed for the rubble mound breakwater.

19.6. Construction Costs

Initially, let us assume that the breakwater is to be built from concrete elements placed from a crane mounted on the crest of the completed breakwater. A proposed design of an element is shown in figure 19.3. An element with an overall length of 6.0 m has been chosen resulting in an effective length of 5.5 m after mating with adjoining elements.

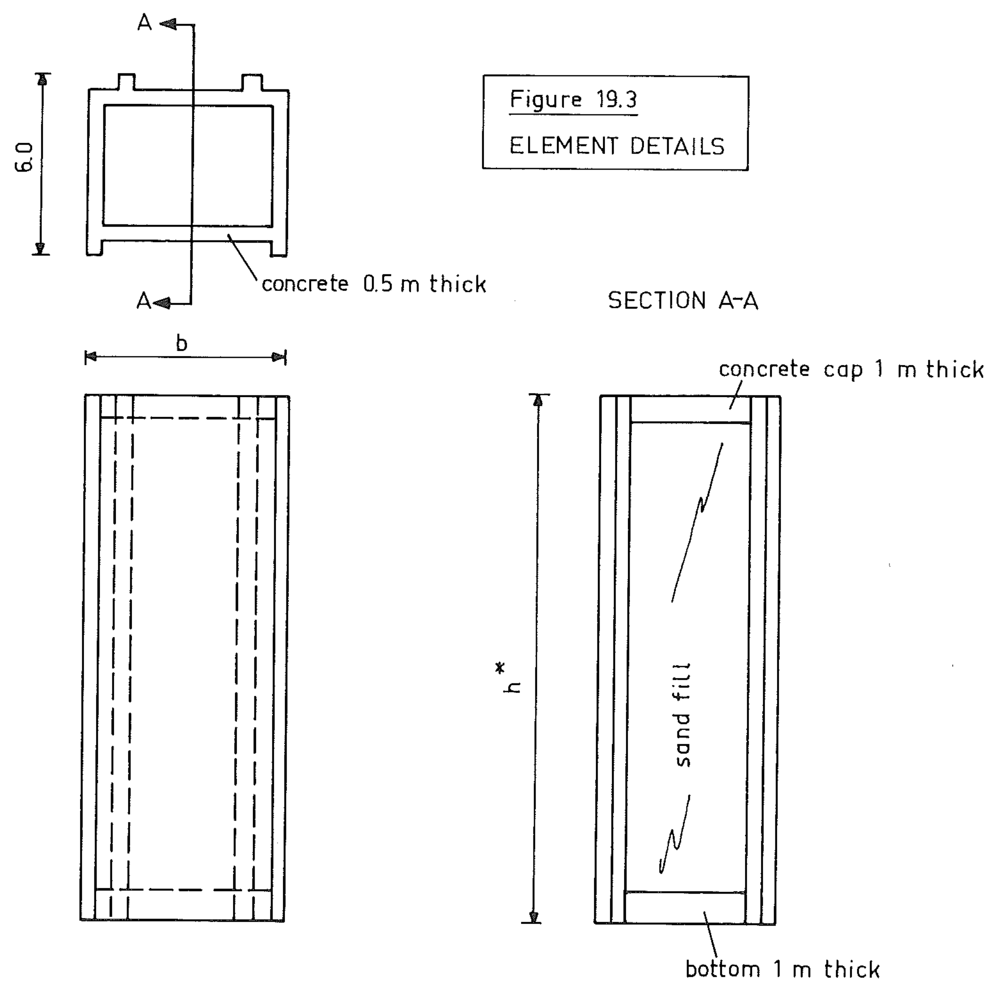


Figure 19.3
ELEMENT DETAILS

The total height, h^*, and the width, b, have been left as variables; the construction costs will be expressed in terms of these variables.

A wall thickness of 0.5 m has been chosen for the element. Such a choice must, of course, be based upon a detailed structural concrete design; such details are beyond the scope of these notes.

The bottom has been chosen to be 1.0 m thick and a 1.0 m thick cap covers the structure after placement and filling. With this background, the cost of an element can be determined - see table 19.5 for material quantities for a single breakwater element. In addition to the quantities listed there, the filter layer and bottom protection must be dimensioned. A layer 20 m wide and 1.5 m thick under the elements will be chosen irrespective of the actual breakwater width, b. Further, a bottom protection 1.0 m thick will be extended 70 meters out in front of the breakwater. This represents about 3/8 of the wave length of the longest wave to be expected - see chapter 17 and table 19.3. Such an apparent overdesign is justified by the low unit cost of the filter as compared to the total structure. For a 5.5 m effective length of breakwater,

$$5.5(1.5 \times 20 + 1.0 \times 70) = 550 \text{ m}^3 \tag{19.05}$$

of gravel costing $40/\text{m}^3$ will be needed. Thus the cost per meter of breakwater length will be:

$$\frac{(550)(40)}{5.5} = 4000/\text{m} \tag{19.06}$$

Other costs, in terms of the dimensions h^* and b, can be determined from data in table 19.5. Reducing everything to a unit length of 1.0 m yields:

$$\text{Cast Concrete: } \frac{400}{5.5} \times (5h^* + bh^* + 4b - 4) \tag{19.07}$$

$$= 363.64\ h^* + 72.73\ bh^* + 290.91b - 290.91 \tag{19.08}$$

$$\text{Cap Concrete: } \frac{150}{5.5}(4b - 4) \tag{19.09}$$

$$= 109.09b - 109.09 \tag{19.10}$$

$$\text{Sand fill: } \frac{6}{5.5}(-4h^* + 4bh^* - 8b + 8) \tag{19.11}$$

$$= -4.36h^* + 4.36bh^* - 8.73b + 8.73 \tag{19.12}$$

Adding all of these costs (19.06, 19.08, 19.10, 19.12) yields:

$$C = 359.28h^* + 81.09bh^* + 391.27b + 3608.73 \tag{19.13}$$

TABLE 19.5 Element Quantities

Item (-)	dimensions (m)	number (-)	volume (m^3)
Concrete: ρ = 2400 kg/m^3; costing 400/m^3			
Ribs	$0.5 \times 0.5 \times h^*$	4	h^*
Side Walls	$0.5 \times 4 \times h^*$	2	$4h^*$
End Walls	$0.5 \times h^* \times b$	2	h^*b
Bottom	$4 \times (b-1) \times 1$	1	$4b-4$
			$5h^* + bh^* + 4b - 4$
Concrete: ρ = 2400 kg/m^3; costing 150/m^3			
Cap	$4 \times (b-1) \times 1$	1	$4b-4$
Sand Fill, wet: ρ = 2400 kg/m^3; costing 6/m^3			
	$(h^*-2)(b-1)4$	-	$-4h^* + 4bh^* - 8b + 8$

Equation 19.13 gives the relationship between the breakwater dimensions, h^* and b, and the construction cost. Thus, with $h^* = 17.5$ and b = 9 m, for example, the cost is:

$$(359.28)(17.5) + (81.09)(9)(17.5) + (391.27)(9) + 3608.73$$

$$= 26189.24/\mathrm{m} \tag{19.14}$$

of which only 4000 is associated with the filter.

19.7. Determination of Damage

Since two optimization variables are involved, they will be varied independently with one, the crest elevation being held constant while the width is varied. This process will be repeated with various (fixed) crest elevations. The steps below are numbered for easier reference. The order shown is not the only one possible; other sequences of the first steps, especially, are conceivable.

1. Choose a crest elevation. Initially, we shall work with the lowest crest which satisfies the overtopping condition - 6.2 m above SWL and 9 m above MSL - see section 19.5. Since the breakwater is located on a filter 1.5 m thick placed in 10 m waterdepth, the total height, h^*, of the monolith will be:

$$h^* = 9 + 10 - 1.5 = 17.5 \text{ m} \tag{19.15}$$

This establishes the first of our optimization variables.

2. Choose a design wave for a clapotis calculation. Using Table 19.3 we choose, initially, the maximum wave occurring in a storm with recurrance interval of 50 years. This yields $H_x = H_d = 8.7$ m with a SWL 4.4 m above MSL. This wave has a period of 12.5 sec. and a length, λ, of 139 m. Other values will be chosen later when a new condition is needed.

3. Compute the clapotis force. The clapotis force is computed using the methods described in section 15.2. The integration of the dynamic part of equation 15.01 extends from the SWL (MSL + 4.4 m) to the bottom of the monolith (MSL - 8.5 m). Thus:

$$\hat{F}_{w1} = \frac{\rho\ gH}{\cosh\ kh} \int_{-h_t}^{0} \cosh\ k(z + h)\ dz \tag{19.16}$$

where:

g is the acceleration of gravity,

H is the approaching wave height,

h is the water depth to SWL.

h_t is the depth above the structure toe,

k is the wave number = $2\pi/\lambda$,

z is the vertical coordinate,

ρ is the mass density of water, and

λ is the wave length.

$$\hat{F}_{w1} = \frac{\rho\ gH}{k \cosh\ kh} \sinh\ k(z + h) \Big|_{z = -h_t}^{0} \tag{19.17}$$

Since the structure is placed on a narrow, very porous filter, the water depth will be considered to extend to the sand bottom, 1.5 m deeper than the toe of the structure: Thus $h - h_t = 1.5$ m and:

$$\hat{F}_{w1} = \frac{\lambda\ \rho\ gH}{2\pi\ \cosh\ kh} [\sinh(kh) - \sinh(1.5k)] \tag{19.18}$$

$$= \frac{(1030)(9.81)(\lambda)(H)}{(2)(\pi)(\cosh\ kh)} [\sinh\ kh - \sinh\ (1.5\ k)] \tag{19.19}$$

$$= \frac{1608.62\ \lambda\ H}{\cosh\ kh} [\sinh\ kh - \sinh\ (1.5\ k)] \tag{19.20}$$

This is independent of the crest elevation and can, therefore, be evaluated once for each wave condition. This has been done with the results listed in table 19.6.

An additional force component results from the wave above the SWL. When there is overtopping:

$$\hat{F}_{w2} = \tfrac{1}{2}\ \rho\, g\, H_d\ z_c \tag{19.21}$$

where z_c is the crest height above SWL.

and

$$\hat{F}_{w2} = \tfrac{1}{2}\ \rho g H^2 \qquad (19.22)$$

when there is no overtopping ($z_c > H$). Since this force is dependent upon the crest elevation, it must be computed separately for each design case.

TABLE 19.6 Wave Force Computation on lower portion of breakwater.

Rec.Int. (yrs)	H (m)	h' (m)	λ (m)	$\hat{F}_{w1}$ (N/m)
10	8.2	3.9	125.	8.959×10^5
20	8.5	4.2	132.	9.586×10^5
50	8.7	4.4	139.	1.005×10^6
100	8.9	4.6	147.	1.053×10^6
500	9.3	5.1	162.	1.157×10^6
1000	9.5	5.3	175.	1.212×10^6
5000	9.9	5.8	191.	1.321×10^6

Using the data above for the problem at hand:

$$\hat{F}_{w2} = (\tfrac{1}{2})(1030)(9.81)(8.7)(4.6) \qquad (19.23)$$

$$= 2.022 \times 10^5 \text{ N/m} \qquad (19.24)$$

The total force is, now:

$$\hat{F}_w = \hat{F}_{w1} + \hat{F}_{w2} \qquad (19.25)$$

$$= 1.207 \times 10^6 \text{ N/m} \qquad (19.26)$$

4. Determine the width, b, necessary to withstand this clapotis. Two criteria must be examined - sliding and overturning. No movement is to be allowed in either case.

Based upon the discussion in chapter 16, sliding will not occur if:

$$\frac{W - B}{\hat{F}_w} \left(\frac{\mu}{1 + \mu\varepsilon}\right) \geq 1.00 \qquad (16.26)\ (19.27)$$

where:

B is the bouyant force on the breakwater section,
W is the weight of the section,
μ is the coefficient of sliding friction, and
ε is a pressure coefficient.

Substituting from (16.40),(16.44) and using:

$$W = bh^* \rho_B\, g \qquad (19.28)$$

yields, when $\mu = 0.5$:

$$\frac{(b)(h^*)\rho_B\, g - b(h_t)\, \rho g}{\hat{F}_w} \quad \frac{0.5}{1 + 0.5\, \frac{b}{2\, h_t}} \geq 1 \tag{19.29}$$

or, since $z_c = h^* - h_t$:

$$\frac{b}{4\, h_t + b} \geq \frac{\hat{F}_w}{2\, g\, h_t[\rho_B\, z_c + (\rho_B - \rho)\, h_t]} \tag{19.30}$$

Substituting numerical values for the problem at hand yields:

$$\frac{b}{(4)(12.9) + b} \geq \frac{1.207 \times 10^6}{(2)(9.81)(12.9)\ [(2400)(4.6) + (2400 - 1030)12.9]} \tag{19.31}$$

$$\frac{b}{b + 51.6} \geq 0.166 \tag{19.32}$$

or:

$$b \geq \frac{(0.166)(51.6)}{1 - 0.166} = 10.27\ \text{m} \tag{19.33}$$

The rotation check follows from equation 16.69:

$$b \geq \left[\frac{\hat{F}_w\, h_t}{\rho_B\, g\, z_c - \frac{2\, \hat{F}_w}{3\, h_t} + (\rho_B - \rho)\, g\, h_t}\right]^{\frac{1}{2}} \tag{16.66)(19.34}$$

Again substituting values yields:

$$b \geq \left[\frac{(1\ .207 \times 10^6)(12.9)}{(2400)(9.81)(4.6) - \frac{(2)(1.207 \times 10^6)}{(3)(12.9)} + (2400 - 1030)(9.81)(12.9)}\right]^{\frac{1}{2}} \tag{19.35}$$

$$\geq \left[\frac{1.557 \times 10^7}{1.083 \times 10^5 - 6.238 \times 10^4 + 1.734 \times 10^5}\right]^{\frac{1}{2}} \tag{19.36}$$

$$\geq 8.42\ \text{m}$$

Choose b = 10.3 m; this satisfies both conditions.

5. Determine short period dynamic force necessary to cause a given sliding displacement. For the problem at hand, a displacement limit of 0.1 m has been suggested. This is again an inverse problem much like that in section 16.6.

Using (16.39) yields:

$$x\Big|_{t=t_2} = \frac{\hat{F}_w\, (1 + \mu\varepsilon)}{m_B\, \omega^2}\ f(\omega t_1) \tag{16.39)\ (19.37}$$

Again using (16.40) and assuming that the dynamic force has a period of one second:

$$x\Big|_{t=t_2} = \frac{\hat{F}_w(1 + 0.5\,\frac{b}{2h_t})}{bh^* \rho_B(\frac{2\pi}{T})^2}\, f(\omega t_1) \tag{19.38}$$

Thus:

$$\hat{F}_w \cdot f(\omega t_1) = \frac{4\pi^2\, bh^*\, \rho_B}{1 + \frac{b}{4h_t}}\, x\Big|_{t=t_2} \tag{19.39}$$

Since the right hand side of (19.39) can be evaluated, this equation is of the form:

$$\hat{F}_w\, f(\omega t_1) = J \tag{19.40}$$

where J is a constant.

A second relation involving $\hat{F}_w$ which must be satisfied results from equation 16.26:

$$\sin(\omega t_1) = \frac{W - B}{\hat{F}_w}\left(\frac{\mu}{1 + \mu\varepsilon}\right) \tag{19.41}$$

or, using previously introduced relations such as were used in (19.29):

$$\hat{F}_w \sin(\omega t_1) = \frac{bg(h^*\, \rho_B - \rho\, h_t)}{2(1 + \frac{b}{4h_t})} \tag{19.42}$$

Again, since the right hand side of (19.42) can be evaluated it is of form:

$$\hat{F}_w \sin(\omega t_1) = K \tag{19.43}$$

where K is a constant.

Dividing (19.40) by (19.43) yields:

$$\frac{f(\omega t_1)}{\sin(\omega t_1)} = \frac{J}{K} \tag{19.44}$$

If we know the ratio $\frac{f(\omega t_1)}{\sin(\omega t_1)}$ as a function of ωt_1, then ωt_1 can be evaluated from known parameters. Once ωt_1 is known, then $\hat{F}_w$ follows directly from, for example, (19.43). The ratio $\frac{f(\omega t_1)}{\sin(\omega t_1)}$ is independent of the breakwater properties and can be evaluated from the data in table 16.2. This has been done; the results are listed in table 19.7 and shown in figure 19.4.

For the problem at hand,

$$\frac{f(\omega t_1)}{\sin(\omega t_1)} = \frac{4\pi^2\, bh^*\, \rho_B}{(1 + \frac{b}{4h_t})}\, x\Big|_{t=t_2}\, \frac{2(1 + \frac{b}{4h_t})}{bg(h^*\rho_B - h_t\, \rho)} \tag{19.45}$$

which with the known constants yields:

$$\frac{f(\omega t_1)}{\sin(\omega t_1)} = \frac{(0.1)(4)(\pi^2)(2400)\ h^* \ (2)}{(9.81)(2400\ h^* - 1030\ h_t)} \tag{19.46}$$

$$= \frac{1931.10\ h^*}{2400\ h^* - 1030\ h_t} \tag{19.47}$$

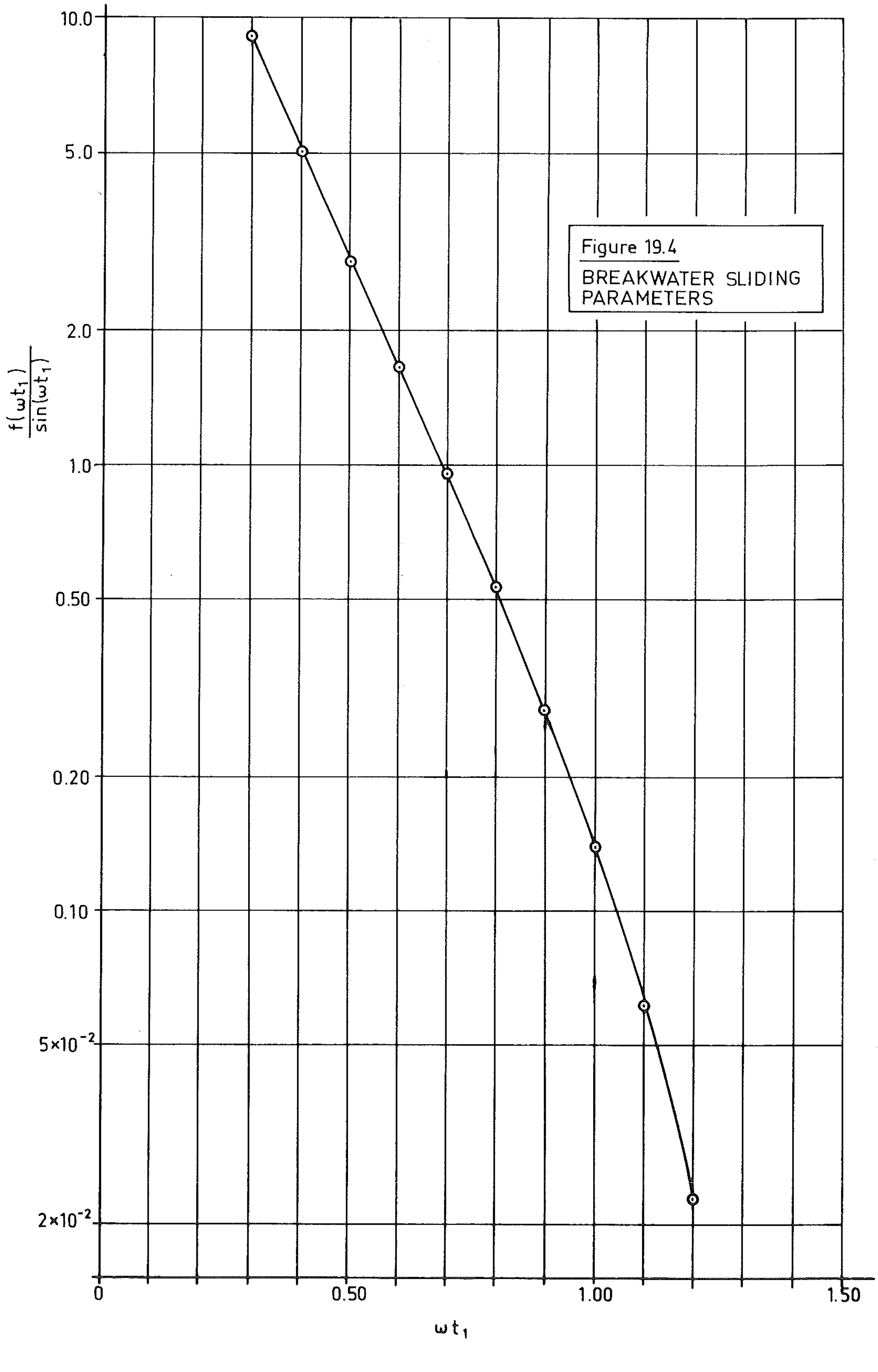


Figure 19.4
BREAKWATER SLIDING PARAMETERS

TABLE 19.7 Additional Breakwater Sliding Parameters

ωt_1	$\frac{f(\omega t_1)}{\sin(\omega t_1)}$
(rad)	-
0.2	18.1276
0.3	9.0728
0.4	5.0108
0.5	2.8789
0.5236	2.5316
0.6	1.6696
0.7	0.9573
0.7854	0.5819
0.8	0.5329
0.9	0.2824
1.00	0.1389
1.0472	0.0958
1.1	0.0612
1.2	0.0226
1.3	6.2479×10^{-3}
1.4	9.6959×10^{-4}
1.5	2.8323×10^{-5}
1.5708	0.0000

which is a function of the water depth and crest elevation. For the present problem:

$$\frac{f(\omega t_1)}{\sin(\omega t_1)} = \frac{(1931.10)(17.5)}{(2400)(17.5)-(1030)(12.9)} \tag{19.48}$$

$$= 1.177 \tag{19.49}$$

which yields:

$$\omega t_1 = 0.66 \tag{19.50}$$

$\hat{F}_w$ follows from (19.42):

$$\hat{F}_w = \frac{(9.81)(b)(2400\ h^* - 1030\ h_t)}{(2)(1 + \frac{b}{4h_t})\ \sin(\omega t_1)} \tag{19.51}$$

yielding:

$$\hat{F}_w = \frac{(9.81)(10.3)[(2400)(17.5) - (1030)(12.9)]}{2(1 + \frac{10.3}{(4)(12.9)})\ \sin(0.66)} \tag{19.52}$$

$$= 1.973 \times 10^6\ \text{N/m} \tag{19.53}$$

6. Determine the value of $\hat{F}_w$ necessary to cause tipping. This value of $\hat{F}_w$ can be determined by solving equation 16.67 for $\hat{F}_w$:

$$\hat{F}_w = \frac{g\, b^2(\rho_B\, h^* - \rho h_t)}{(h_t + \frac{2b^2}{3h_t})} \tag{19.54}$$

Substituting values for this problem yields:

$$\hat{F}_w = \frac{(9.81)(10.3)^2\,[(2400)(17.5) - (1030)(12.9)]}{(12.9) + \frac{(2)(10.3)^2}{(3)(12.9)}} \tag{19.55}$$

$$= 1.626 \times 10^6 \text{ N/m} \tag{19.56}$$

7. Choose the least of the forces $\hat{F}_w$ found in steps 5 and 6. This is done because the lower force has the greatest frequency of exceedance. In this calculation, the lower force is 1.626×10^6 N/m and is determined by a rotation criterium. This means that sliding will not be a problem; a short period force will cause failure by tipping before sliding becomes critical. Thus, the probability of failure by sliding is totally irrelevant.

From figure 19.1, with $\hat{F}_w = 1.626 \times 10^6$ N/m:

$$P(\hat{F}_w) = 1/34 \quad \text{per year} \tag{19.57}$$

This is the probability of failure by exceeding the short period dynamic force.

8. Determine the overall probability of failure. This is done by adding the probability of failure from step 7 to that chosen for the clapotis in step 2. These probabilities are added since the two events can occur independent of one another; that is, there is no relation between wave height or water depth and the magnitude of the forces depicted in figure 19.1 - see v.d. Kreeke (1963). For the current problem this overall probability, P(f), is:

$$P(f) = P(\hat{F}_w) + P(H_d) \tag{19.58}$$

$$= 1/34 + 1/50 = 4.941 \times 10^{-2} \tag{19.59}$$

9. Determine the construction cost. The preliminary work for this step was completed in section 6 of this chapter. The construction cost follows directly from the dimensions and unit prices as reflected in equation 19.13. For the current dimensions:

$$C = (359.28)(17.5) + (81.09)(10.3)(17.5) + (391.27)(10.3) + 3608.73 \tag{19.60}$$

$$= 2.854 \times 10^4 \text{ N/m} \tag{19.61}$$

10. Determine the capitalized damage cost.

Unlike a rubble mound breakwater, a monolithic breakwater does not suffer partial damage - there is either no damage or destruction. For this reason, the damage cost of the breakwater is related to the total construction cost. Also, since a destroyed breakwater must be cleaned away from the site before a new structure can be constructed, the damage cost will be greater than the construction cost alone. It is therefore assumed that damage costs - if they occur - will amount to twice the construction cost of the breakwater. Thus, the annual damage cost is:

$$\text{annual damage cost} = (2.)(C)(P(f)) \tag{19.62}$$

The capitalized damage cost is this amount in (19.62) times the present worth factor. Using the same factor as in chapter 11 section 7:

$$pwf = 12.2335 \qquad (11.31) \tag{19.63}$$

yields a capitalized damage cost of:

$$\text{cap.dam.} = (12.2335)(2)(C)(P(f)) \tag{19.64}$$

11. Determine the total cost by adding the construction cost to the capitalized damage cost:

$$\text{total cost} = [(12.2335)(2)(P(f)) + 1\ ]\ C \tag{19.65}$$

$$= [24.467\ P(f) + 1\ ]\ C \tag{19.66}$$

or in this case:

$$\text{total cost} = [(24.467)(4.941 \times 10^{-2}) + 1]\ \ 2.854 \times 10^{4} \tag{19.67}$$

$$= 6.305 \times 10^{4}/\text{m} \tag{19.68}$$

19.8. The Optimization

The procedure just outlined has determined the total cost of a single breakwater. Obviously the optimum design is that which has the lowest total cost. This optimum must be found by repeating steps 2 through 11 in the previous section for various design waves and then repeating the entire procedure - beginning with step 1 - for various crest elevations. This is done in table 19.8 in which each row is computed using the methods described in the previous section. Results of that specific computation are shown in the top row of figures in the table.

Step and equation numbers are listed on each column in order to make the computation more clear. The computations involved in a single row can be carried out on a programmable pocket calculator; seven programs were used in sequence.

Since it appears from the last column of the table that the total cost is still decreasing for the design with $P(H_d)$ = 1/5000, additional, wider breakwaters were also computed. For these, the chance of failure due to a clapotis is effectively zero; all damage is caused by the short period wave force.

The data presented in the table can be presented in various ways. Cost graphs showing costs versus width are plotted for each of the chosen crest elevations in figure 19.5. The overall optimum is not too obvious, however; various curves must be compared in order to reveal the optimum. A help for this visualization might be to plot the total cost of the best solution at each crest elevation as a function of crest elevation and of width. This results in the curves shown in figure 19.6. They indicate that an optimum solution must be about 13 m wide and 20 m high. Another more conventional visualization for an optimization function of two variables is to plot contour lines of constant parameter value (total cost, here) as a function of the two optimization parameters, height and width. This is shown in figure 19.7. The previous figures can, of course, be related to figure 19.7. The curves in figure 19.5 are profiles made by intersecting the optimization surface with planes h^* = constant. Figure 19.6 is a projection of points near the bottom of the "valley" seen running from the upper left to lower right in figure 19.7 on to planes perpendicular to the coordinate axes.

The optimum design appears to have a height of about 20.2 m and a width of about 13.0 m - figure 19.7. Examining and interpolating in table 19.8 yields the following conclusions:

a. The breakwater is heavy enough to withstand all clapotis forces - $P(H_d) \approx 0$.
b. The crest elevation is considerably higher than that needed to limit the overtopping to an acceptable degree.
c. All damage will result from the short period dynamic forces. Failure will occur by tipping with a frequency of occurrence of about 1/500 per year.
d. An incremental increase in height improves stability more than an equal incremental increase in width. This follows from the relative slopes of the two curves in figure 19.6.

The optimum design is sketched in figure 19.8.

TABLE 19.8 Optimization Computations

Step No.	1	2					3			4			5			6	7		8	9	11
Eq. No.	-	-	-	-	-	-	tab.19.6	(19.21/22)	(19.25)	(19.30)	(19.34)	-	(19.47)	fig. 19.4	(19.51)	(19.54)	-	fig. 19.1	(19.58)	(19.13)	(19.65)
										SLIP	TIP	chosen			sliding	tipping	critical				
	h^*	$P(H_d)$	H_d	h'	z_c	h_t	$\hat{F}_{w1}$	$\hat{F}_{w2}$	$\hat{F}_w$	b	b	b	$\frac{f(\omega t_1)}{\sin \omega t_1}$	ωt_1	$\hat{F}_w$	$\hat{F}_w$	$\hat{F}_w$	$P(\hat{F}_w)$	P(f)	C	'total cost'
	(m)	(-)	(m)	(m)	(m)	(m)	(N/m)	(N/m)	(N/m)	(m)	(m)	(m)	-	-	(N/m)	(N/m)	(N/m)	(-)	(-)	(-/m)	(-/m)
	17.5	1/50	8.7	4.4	4.6	12.9	1.005×10^6	2.022×10^5	1.207×10^6	10.27	8.42	10.3	1.177	0.66	1.973×10^6	1.626×10^6	1.626×10^6	1/34	4.941×10^{-2}	2.854×10^4	6.305×10^4
	17.5	1/100	8.9	4.6	4.4	13.1	1.053×10^6	1.979×10^5	1.251×10^6	10.79	8.71	10.8	1.185	0.66	2.043×10^6	1.714×10^6	1.714×10^6	1/41	3.439×10^{-2}	2.945×10^4	5.423×10^4
	17.5	1/500	9.3	5.1	3.9	13.6	1.157×10^6	1.833×10^5	1.340×10^6	11.89	9.34	11.9	1.207	0.65	2.216×10^6	1.994×10^6	1.894×10^6	1/62	1.813×10^{-2}	3.144×10^4	4.538×10^4
	17.5	1/1000	9.5	5.3	3.7	13.8	1.212×10^6	1.776×10^5	1.390×10^6	12.51	9.66	12.5	1.216	0.65	2.296×10^6	1.996×10^6	1.996×10^6	1/80	1.350×10^{-2}	3.253×10^4	4.327×10^4
	17.5	1/5000	9.9	5.8	3.2	14.3	1.321×10^6	1.601×10^5	1.481×10^6	13.72	10.33	13.8	1.239	0.65	2.458×10^6	2.199×10^6	2.199×10^6	1/140	7.343×10^{-3}	3.488×10^4	4.115×10^4
	17.5	0				14.3						14.5		0.65	2.558×10^6	2.334×10^6	2.334×10^6	1/205	4.878×10^{-3}	3.615×10^4	4.046×10^4
	17.5	0				14.3						16.0		0.65	2.764×10^6	2.611×10^6	2.611×10^6	1/450	2.222×10^{-3}	3.886×10^4	4.097×10^4
	19	1/50	8.7	4.4	6.1	12.9	1.005×10^6	2.682×10^5	1.273×10^6	9.51	8.08	9.5	1.135	0.67	2.048×10^6	1.629×10^6	1.629×10^6	1/34	4.941×10^{-2}	2.879×10^4	6.359×10^4
	19	1/100	8.9	4.6	5.9	13.1	1.053×10^6	2.654×10^5	1.318×10^6	9.96	8.34	10.0	1.143	0.67	2.130×10^6	1.732×10^6	1.732×10^6	1/43	3.326×10^{-2}	2.975×10^4	5.397×10^4
	19	1/500	9.3	5.1	5.4	13.6	1.157×10^6	2.238×10^5	1.411×10^6	10.93	8.93	11.0	1.161	0.66	2.313×10^6	1.921×10^6	1.921×10^6	1/67	1.693×10^{-2}	3.169×10^4	4.481×10^4
	19	1/1000	9.5	5.3	5.2	13.8	1.212×10^6	2.496×10^5	1.462×10^6	11.47	9.22	11.5	1.169	0.66	2.390×10^6	2.018×10^6	2.018×10^6	1/86	1.263×10^{-2}	3.265×10^4	4.274×10^4
	19	1/5000	9.9	5.8	4.7	14.3	1.321×10^6	2.351×10^5	1.556×10^6	12.52	9.83	12.5	1.189	0.66	2.534×10^6	2.193×10^6	2.193×10^6	1/137	7.499×10^{-3}	3.458×10^4	4.093×10^4
	19	0				14.3						14.0		0.66	2.779×10^6	2.533×10^6	2.533×10^6	1/360	2.778×10^{-3}	3.748×10^4	4.003×10^4
	19	0				14.3						15.0		0.66	2.936×10^6	2.750×10^6	2.750×10^6	1/650	1.538×10^{-3}	3.941×10^4	4.090×10^4
	21	1/500	9.3	5.1	7.4	13.6	1.157×10^6	3.478×10^5	1.505×10^6	9.97	8.50	10.0	1.114	0.67	2.429×10^6	1.930×10^6	1.930×10^6	1/69	1.649×10^{-2}	3.210×10^4	4.505×10^4
	21	1/1000	9.5	5.3	7.2	13.8	1.212×10^6	3.457×10^5	1.558×10^6	10.43	8.77	10.5	1.121	0.67	2.522×10^6	2.047×10^6	2.047×10^6	1/93	1.175×10^{-2}	3.314×10^4	4.267×10^4
	21	1/5000	9.9	5.8	6.7	14.3	1.321×10^6	3.352×10^5	1.659×10^6	11.36	9.33	11.5	1.137	0.67	2.699×10^6	2.262×10^6	2.262×10^6	1/170	6.082×10^{-3}	3.524×10^4	4.048×10^4
	21	0				14.3						12.5		0.67	2.891×10^6	2.534×10^6	2.534×10^6	1/360	2.778×10^{-3}	3.733×10^4	3.987×10^4
	21	0				14.3						13.5		0.67	3.078×10^6	2.798×10^6	2.798×10^6	1/750	1.333×10^{-3}	3.942×10^4	4.071×10^4
	23	1/5000	9.9	5.8	8.7	14.3	1.321×10^6	4.355×10^5	1.756×10^6	10.46	8.92	10.5	1.097	0.67	2.252×10^6	2.837×10^6	2.252×10^6	1/200	5.200×10^{-3}	3.556×10^4	4.009×10^4
	23	0				14.3						12.0		0.67	2.722×10^6	3.172×10^6	2.722×10^6	1/600	1.667×10^{-3}	3.895×10^4	4.054×10^4
	23	0				14.3						12.5		0.67	2.875×10^6	3.280×10^6	2.875×10^6	1/910	1.099×10^{-3}	4.008×10^4	4.115×10^4
	23	1/1000	9.5	5.3	9.2	13.8	1.212×10^6	4.417×10^5	1.654×10^6	9.67	8.42	9.7	1.084	0.68	2.063×10^6	2.639×10^6	2.063×10^6	1/97	1.131×10^{-2}	3.376×10^4	4.310×10^4
	20	1/1000	9.5	5.3	6.2	13.8	1.212×10^6	2.977×10^6	1.510×10^6	10.91	8.98	11.0	1.143	0.66	2.042×10^6	2.480×10^6	2.042×10^6	1/92	1.187×10^{-2}	3.294×10^4	4.250×10^4
	20	1/5000	9.9	5.8	5.7	14.3	1.321×10^6	2.852×10^5	1.606×10^6	11.88	9.55	11.9	1.161	0.66	2.212×10^6	2.623×10^6	2.212×10^6	1/145	7.097×10^{-3}	3.475×10^4	4.078×10^4
	20	0				14.3						13.0		0.66	2.488×10^6	2.820×10^6	2.488×10^6	1/320	3.125×10^{-3}	3.696×10^4	3.979×10^4
	20	0				14.3						14.0		0.66	2.730×10^6	2.995×10^6	2.730×10^6	1/620	1.613×10^{-3}	3.898×10^4	4.052×10^4
	22	1/1000	9.5	5.3	8.2	13.8	1.212×10^6	3.937×10^5	1.606×10^6	10.02	8.58	10.0	1.101	0.67	2.032×10^6	2.581×10^6	2.032×10^6	1/89	1.224×10^{-2}	3.327×10^4	4.322×10^4
	22	1/5000	9.9	5.8	7.7	14.3	1.321×10^6	3.852×10^5	1.706×10^6	10.87	9.11	10.9	1.116	0.67	2.237×10^6	2.754×10^6	2.237×10^6	1/155	6.652×10^{-3}	3.522×10^4	4.096×10^4
	22	0				14.3						12.0		0.67	2.560×10^6	2.984×10^6	2.560×10^6	1/390	2.564×10^{-3}	3.762×10^4	3.998×10^4
	22	0				14.3						13.0		0.67	2.847×10^6	3.186×10^6	2.847×10^6	1/825	1.212×10^{-3}	3.979×10^4	4.097×10^4

Figure 19.5 a
COST CURVES FOR VARIOUS CREST ELEVATIONS

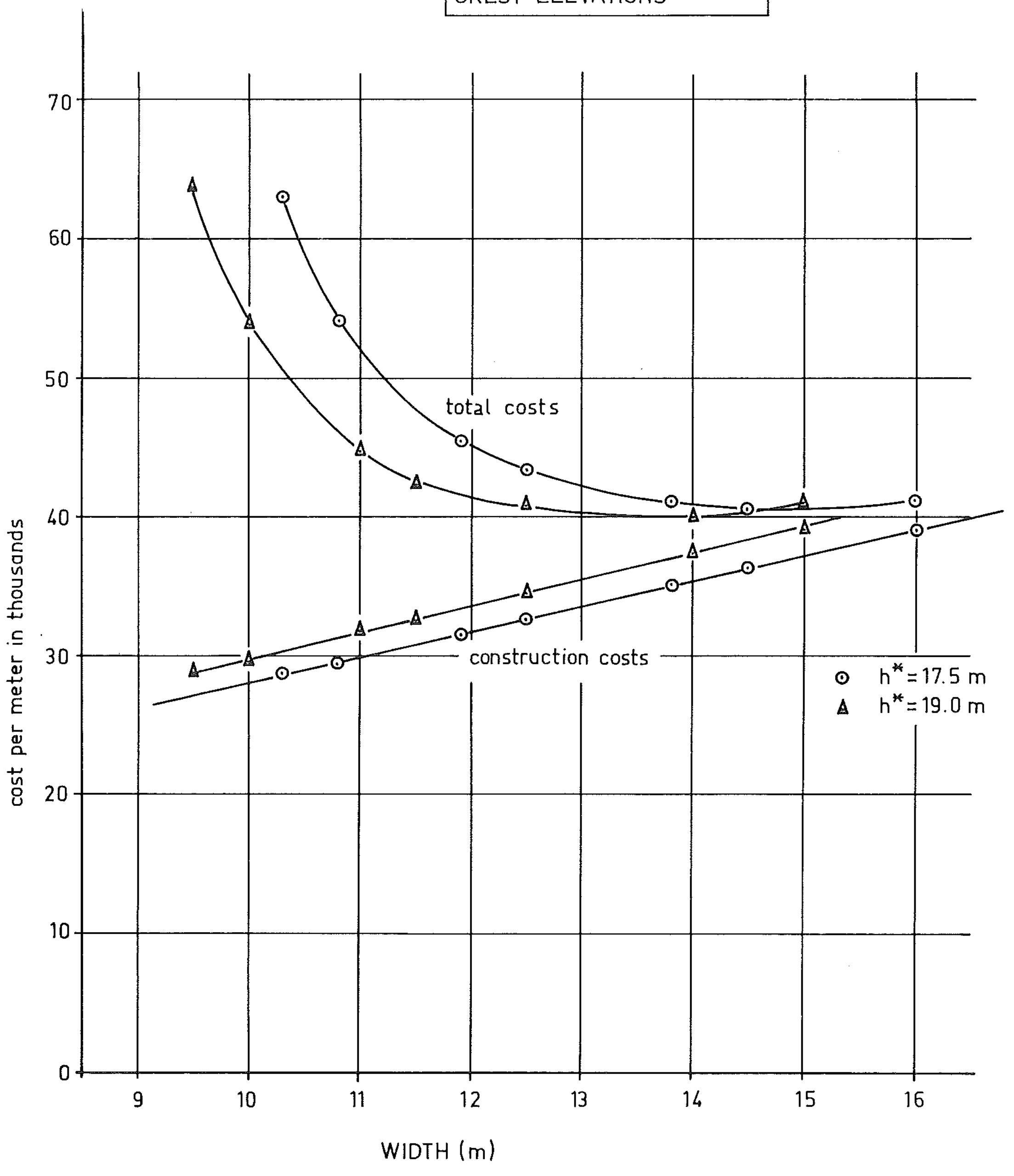


Figure 19.5 b
COST CURVES FOR VARIOUS CREST ELEVATIONS

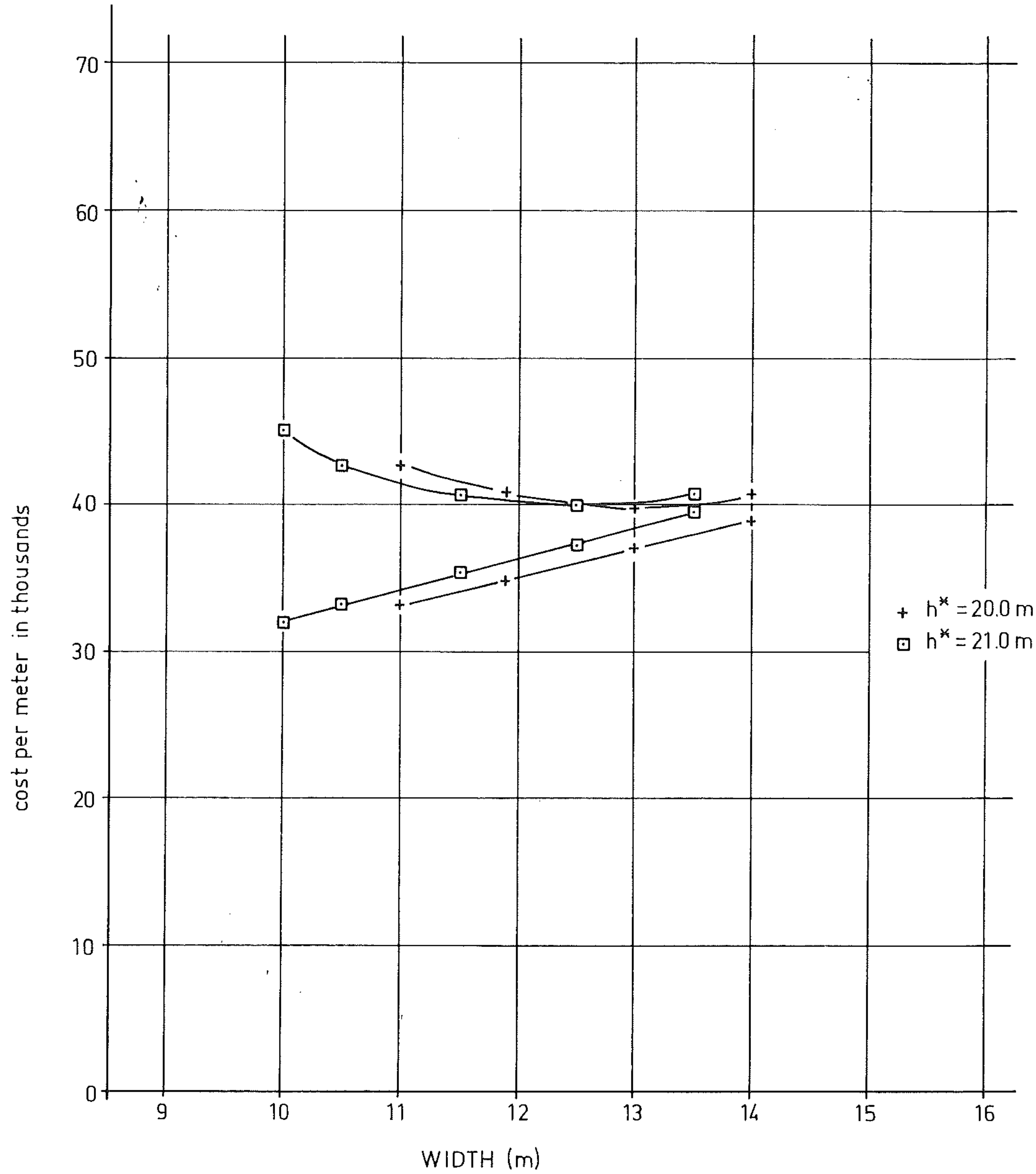


Figure 19.5c

COST CURVES FOR VARIOUS CREST ELEVATIONS

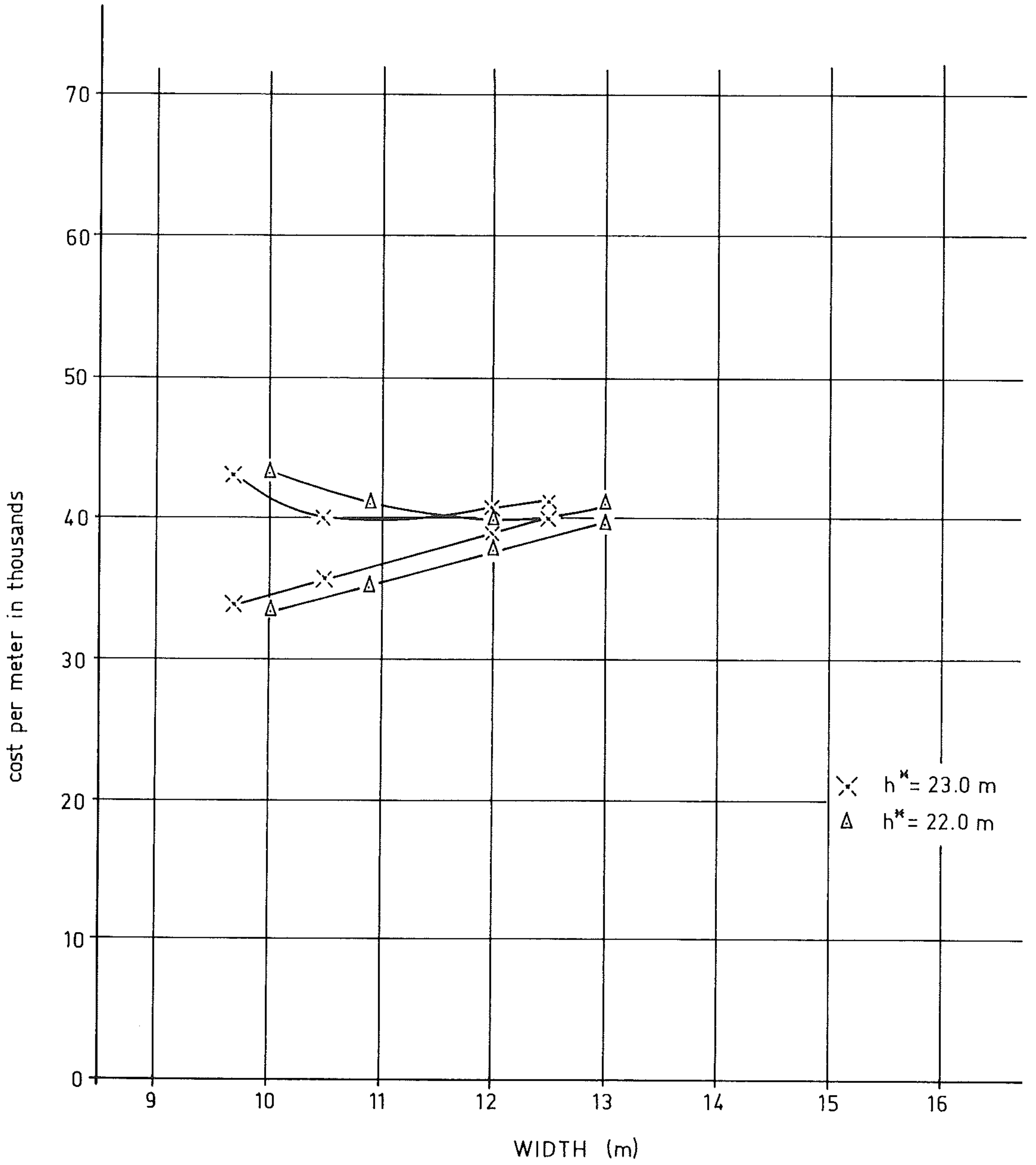
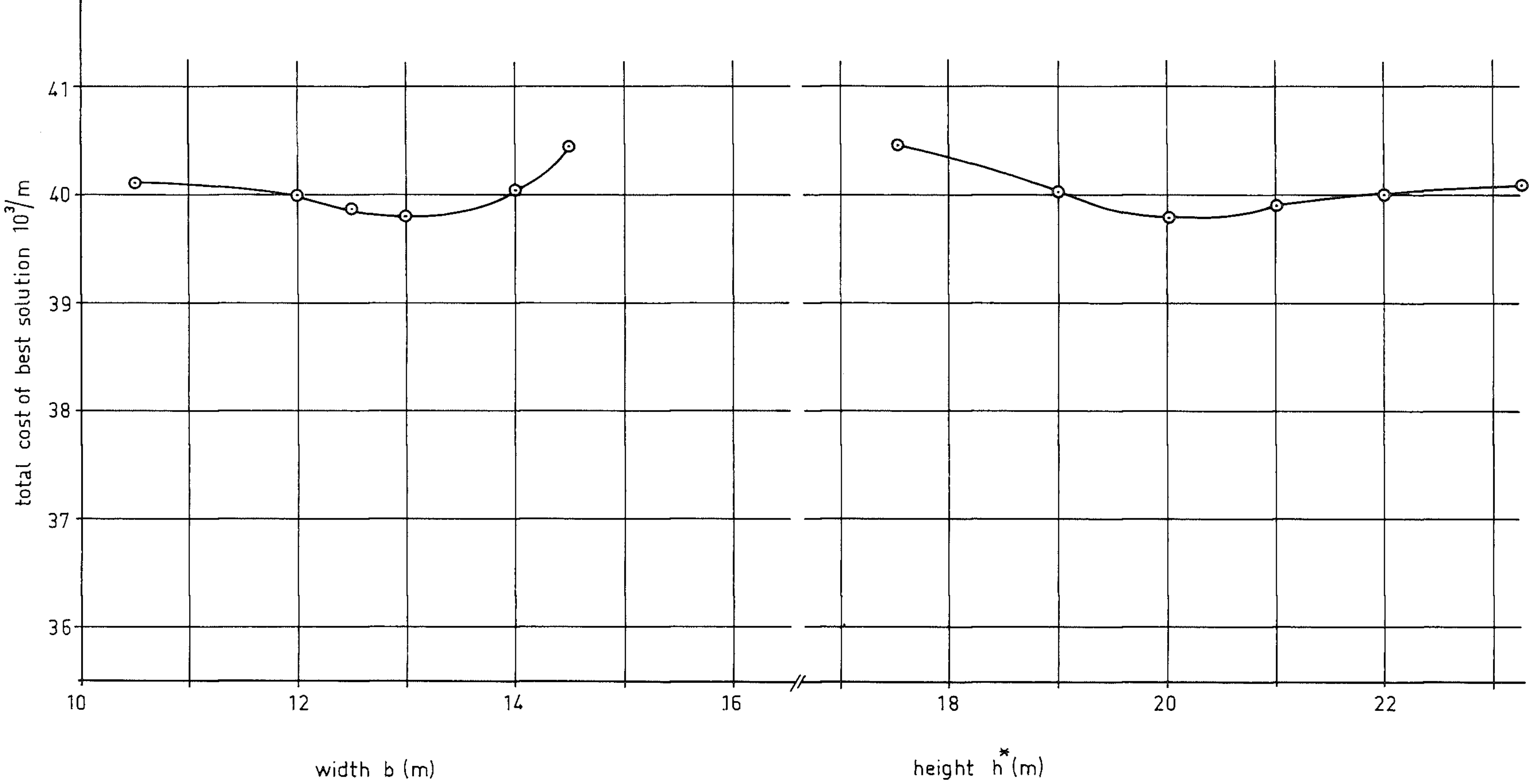


Figure 19.6
TOTAL COST VERSUS HEIGHT
TOTAL COST VERSUS WIDTH
FOR BEST SOLUTIONS



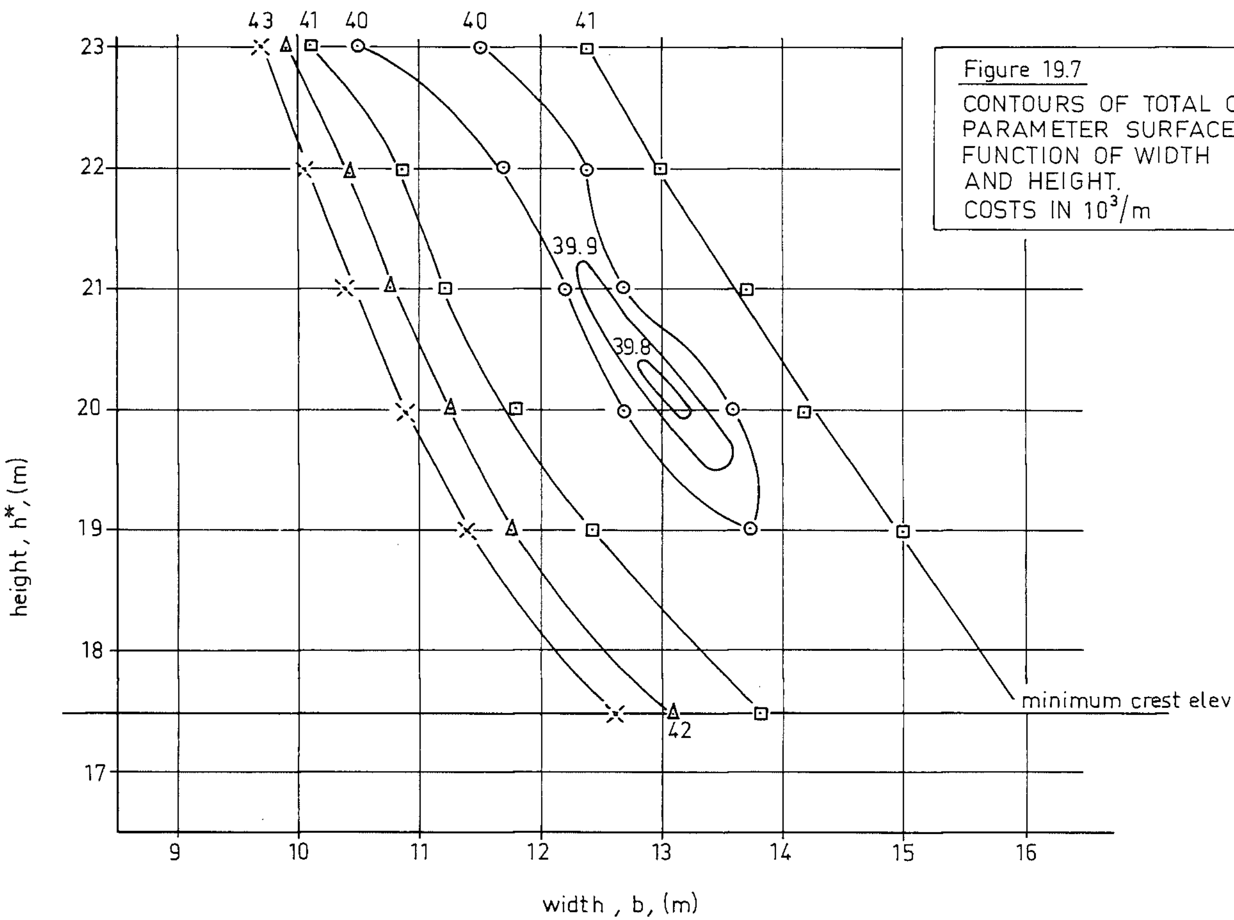


Figure 19.7
CONTOURS OF TOTAL COST PARAMETER SURFACE AS FUNCTION OF WIDTH AND HEIGHT.
COSTS IN 10^3/m

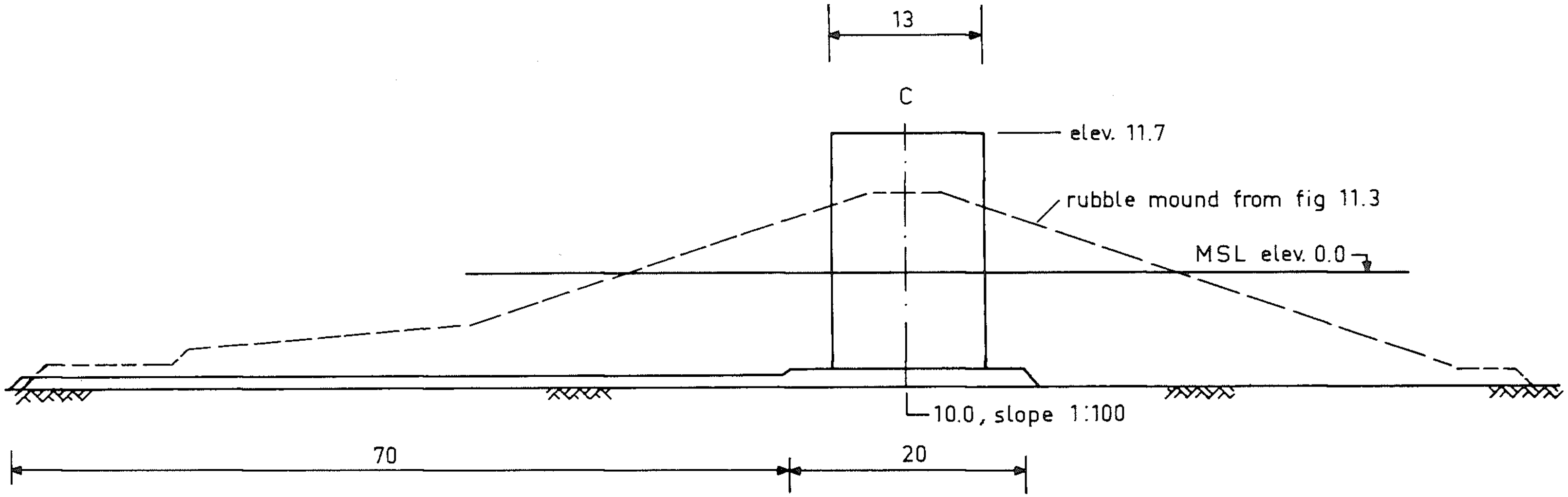


Figure 19.8

SKETCH OF MONOLITHIC BREAKWATER
ORIGINAL SCALE: 1:500

19.9. Additional Comments

The optimum stone rubble mound breakwater for this problem has been outlined in figure 19.8 for comparison purposes. Comparison of the two designs leads to the following conclusions:

a. The crest elevation of the monolithic structure is much higher than that of the rubble mound structure.
b. The unit price of materials (price per cubic meter) for the rubble mound structure is lower than for the monolith. However, since the monolith uses much less material, its total cost is lower even so.
c. The rubble mound breakwater will have to be repaired relatively frequently. the low frequency of repair for the monolithic structure results from the high cost of carrying out these repairs, if they occur.

The discussion of economics - interest rates and life of the structure - presented in section 11.7 remains equally valid for the monolithic structure.

The low frequencies of damage associated with the optimum monolithic design may not prove to be too dependable in practice. Such frequencies must obviously be based upon extrapolations.

Since the annual chance of damage is so small (about 1/500), what is the chance that no maintenance will be needed during the 50 year life of the structure? The chance that maintenance will be needed (failure *will* occur) in any one year is $P(f) \approx 1/500$. The chance that failure *will not* occur in one year is:

$$1 - P(f) \tag{19.69}$$

The chance that this *will not* occur in the life of the structure is:

$$[1 - P(f)]^{\ell} \tag{19.70}$$

where ℓ is the life of the structure. In this case this chance is:

$$[1 - 1/500]^{50} = 0.90475 \tag{19.71}$$

or a bit over 90%! By comparison, for the rubble mound breakwater of chapter 11 with a chance of damage of 1/26 per year, there is a chance of only

$$[1 - 1/26]^{50} = 0.14071 \tag{19.72}$$

or a bit more than 14% of not having to carry out any repairs.

This concludes the section on monolithic breakwaters. The subject of breakwater design concludes in the following chapter with a short discussion of the alternative designs for the northern breakwater at the entrance to Rotterdam.

20. ROTTERDAM - EUROPOORT ENTRANCE DESIGN

J.F. Agema
E.W. Bijker
W.W. Massie

20.1. Introduction

The purpose of this chapter will be to briefly summarize the application of breakwater design principles in a specific case. In order to put the breakwater design in proper perspective, general harbor layout considerations will first be discussed. Later the discussion becomes more specific resulting in construction details of the northern breakwater of the entrance.

A special feature of this particular design study was that both monolithic and rubble mound structures were considered. More important, the economically least expensive solution was not chosen.
The reasons for this appear in section 20.4.

20.2. Harbor Layout Considerations

This design problem involves the expansion of an existing, busy harbor complex. Ship traffic destined for the existing harbor facilities must be taking into consideration when planning the expansion.

One way to avoid conflicts between construction operations and existing shipping is to develop a second, new, separate harbor entrance. While such a plan has advantages during construction, it results in a more complex (dangerous) traffic pattern in the immediately adjacent sea after completion. Many more crossings occur in ship's paths entering and leaving from two adjacent harbor entrances than from a single entrance. Also, tidal current patterns become more complex as the number of entrances increases. Navigation becomes more difficult; wider dredged channels are needed.

All of these factors led to an early decision to use only a single main harbor entrance. The consequences - that an accident in the single harbor entrance could shut down the entire port and that construction activities could not be allowed to significantly hinder shipping - were accepted.

Four possible main purposes of breakwaters are listed in chapter 2: wave reduction, reduce dredging, provide quay facilities, and guide currents. Which of these are important for Europoort? Littoral transport of sand was effectively stopped by other features - the seaward industrial expansion to the south and the existing breakwater and groins to the north. The entrance width would not be varied appreciably - harbor currents and erosion or deposition would not be materially influenced; dredging would not be increased by the breakwater extensions. Adequate quay facilities were planned elsewhere further inland. Since ships would be entering with a reasonable speed, even tugboat assistance could be postponed until ships were well inside the harbor entrance.

The combination of longshore and harbor tidal currents did, however, present harbor layout problems. The layout of the harbor entrance breakwaters was to a great extent determined by the predicted current patterns. The result of the chosen layout on the current pattern has already been shown - figure 2.4. Concluding, the primary purpose of the breakwater is to guide tidal currents. How does this functional need reflect on the breakwater design?

Since wave action in the entrance is not detrimental to the harbor operation in this case, the breakwater crest need not be high; overtopping is of no consequence. Other navigational aids, buoys and fixed lights would guarantee visibility; the crest elevation could be low, only mass overtopping which could lead to substantial currents in the entrance must be prevented. Thus, the minimum crest elevation resulting from the harbor layout was a bit higher than the normal high tide level.*

Breakwater porosity was not a design factor since sand transport and wave transmission were not important. Low porosity was not considered detrimental, but it was not required. All of these design layout aspects are dealt with in more detail in an anonymous Dutch report, *Het Ontwerp van de Nieuwe Havenmond bij Hoek van Holland* (1964). Types of breakwaters which satisfy these harbor layout requirements are discussed in the following section. There, and for the rest of this chapter, the discussion will be restricted to the extension of the northern breakwater - see figure 20.1.

20.3. Proposed Designs

Many types of breakwater structures were considered, all of which met the harbor layout requirements expressed in the previous section. Rubble mound, monolithic and composite constructions were considered, twelve different concepts in all. These are each illustrated via sketches in figure 20.2. Table 20.1 lists the types along summary evaluations of the cross-sections. More detailed data is available in a report by van de Kreeke and Paape.

20.4. Evaluation of Designs

One is impressed by the variety of solutions suggested. However, construction methods were limited to use of floating equipment. Construction over the crest was apparently eliminated as uneconomical early in the design phase. The basis for this may have been the additional cost of raising the crest sufficiently to allow this type of construction operation. The crest elevation would now be determined based upon an overtopping criteria during the construction phase.

The optimizations presented in figure 20.3 and the costs listed in table 20.1 were determined for cross sections located in water 12 m deep as shown in figure 20.2 as well. As is indicated in the remarks in table 20.1, the most economical choice of cross section was a function of the water depth. Figure 20.2.1 shows the most economical solution for 8 m water depth, for example. Obviously, on the other hand, it is very uneconomical to use a multitude of different types of cross sections in the same breakwater. It is best, therefore, to choose a single breakwater form for which only details such as dimensions or weights will vary along the breakwater. The construction process is

* Construction techniques might dictate a higher level in this case.

simplified.

The final choice for the breakwater form was a rubble mound structure, constructed using concrete cubes for primary armor. More details of the design and construction are given in the following section.

20.5. Construction Details

Two cross sections of the northern breakwater are shown in figure 20.4. The locations of these cross sections are shown on figure 20.1. As is shown in figure 20.4, a broad portion of the sea bed was raised using a sand and gravel fill. A large quantity of inexpensive, easily placed material was used in order to reduce the size of the breakwater proper. In this way, a maximum portion of the structure could be built from moving ships; the hinderance to other shipping traffic was minimized. Further details of the construction phases are shown in figure 20.5.

TABLE 20.1 Overview of Breakwater Types

Type	fig. no. 20.2	Relative Costs at Optimum for 12m waterdepth Const.	Maint	Total	chance of Failure at optimum	Remarks
90^{o} Caisson	a	25200	1300	26500	1/1000	economical of material, but expensive to construct, caisson placement difficult and bothersome to shipping
60^{o} Caisson	b	13500	500	14000	1/1000	especially difficult to float into place
Hanstholm Caisson	c	11400	100	11500	1/5000	Very flat optimization curve - figure 20.3a Cheapest solution for 12m water depth. Rock asphalt difficult to place
Hanstholm Caisson with Cubes	d	12700	300	13000	1/3000	Ballasting will be slow with crane
Hanstholm Block Wall	e	14300	200	14500	1/5000	Use large concrete blocks very difficult to construct
Concrete Cube Rubble Mound	f	15400	600	16000	1/1500	Large volume of inexpensive material continuous construction cheapest solution for 10m dept see figure 20.3b
Stone Asfalt Rubble Mound	g	-	-	19000	-	Rock asphalt difficult to place under water
Concrete Cubes Retaining Wall	h	-	-	15000	-	Retaining wall difficult to place but is immediately above water cheapest solution for 8m depth
Caisson with cubes	i	-	-	17000	-	Uses much varied construction equipment
Retaining Wall on top of Rubble Mound	j	-	-	17000	-	Wall difficult to place Rock asphalt top used
Retaining Wall on top of Rubble Mound	k	-	-	17000	-	Retaining wall difficult to place
Concrete Cubes with Crest Struct.	l	-	-	15500	-	Crest structure too difficult to place cheapest solution for 8m water depth

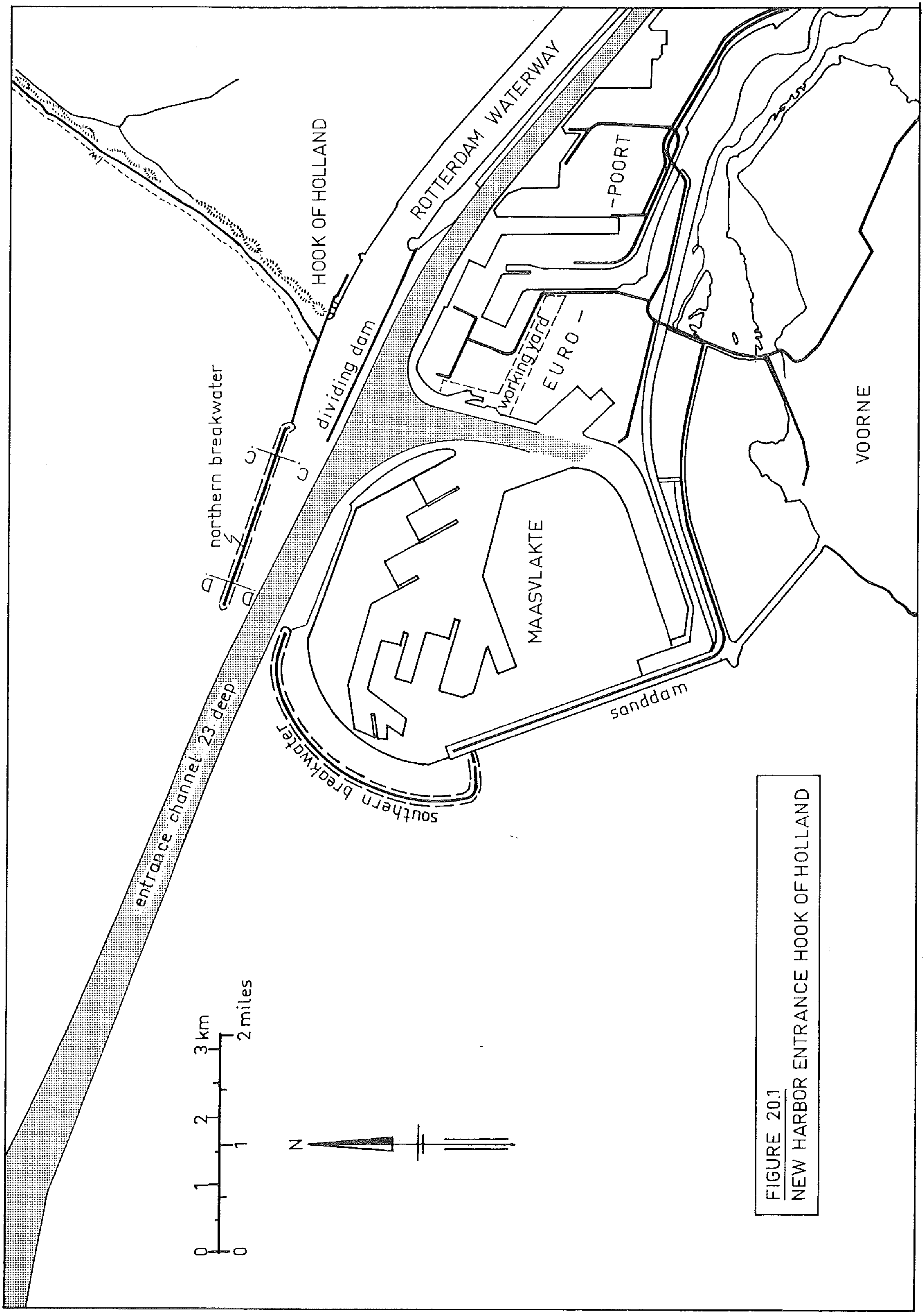
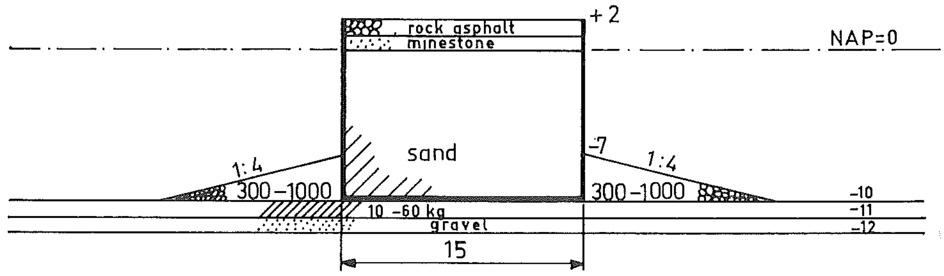


FIGURE 20.1
NEW HARBOR ENTRANCE HOOK OF HOLLAND

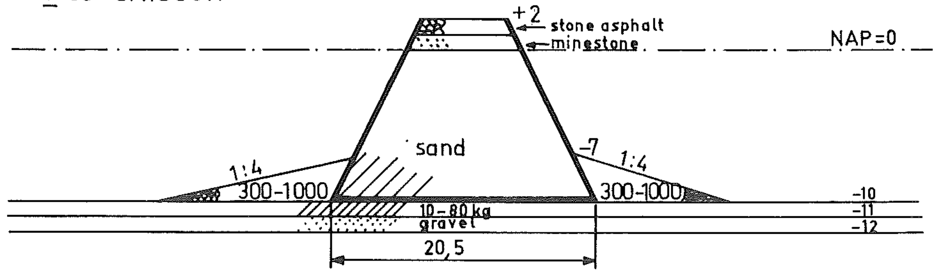
Figure 20.2

PROPOSED DESIGNS FOR NORTH BREAKWATER

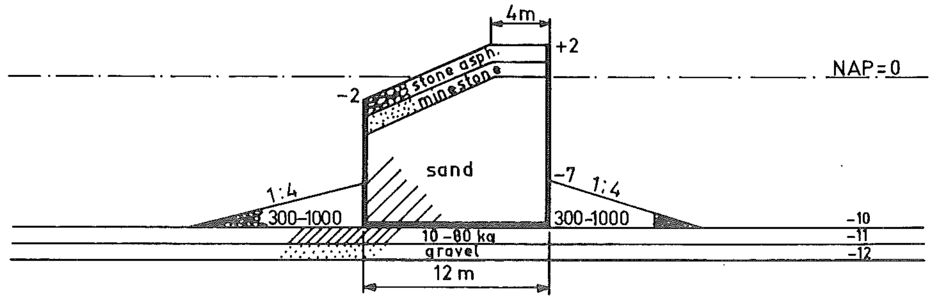
a 90° CAISSON



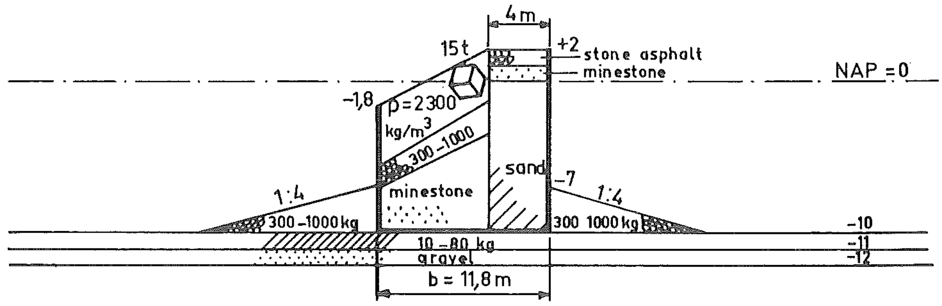
b 60° CAISSON



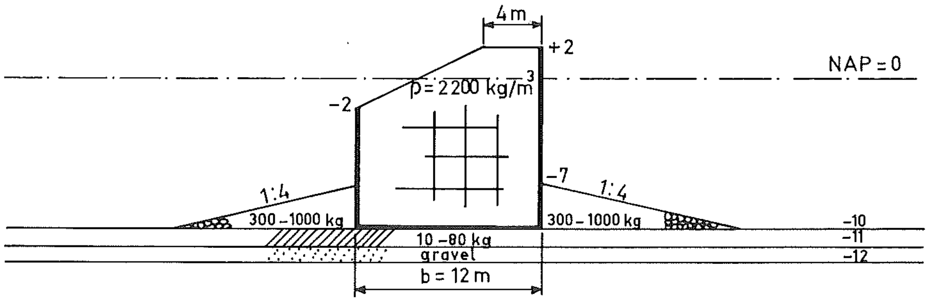
c „HANSTHOLM" CAISSON



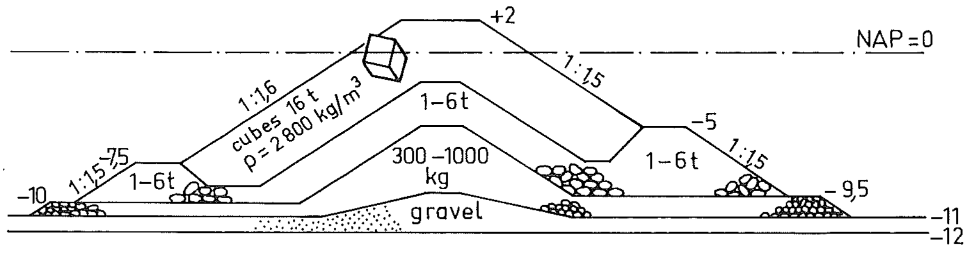
d „HANSTHOLM" CAISSON WITH CUBES



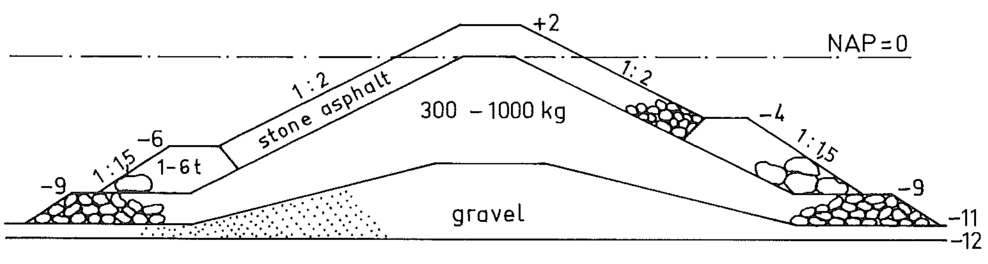
e „HANSTHOLM" BLOCK WALL



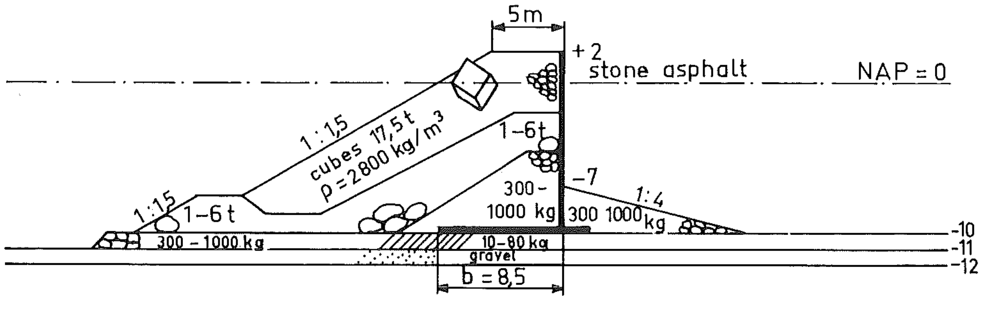
f CONCRETE CUBE RUBBLE MOUND



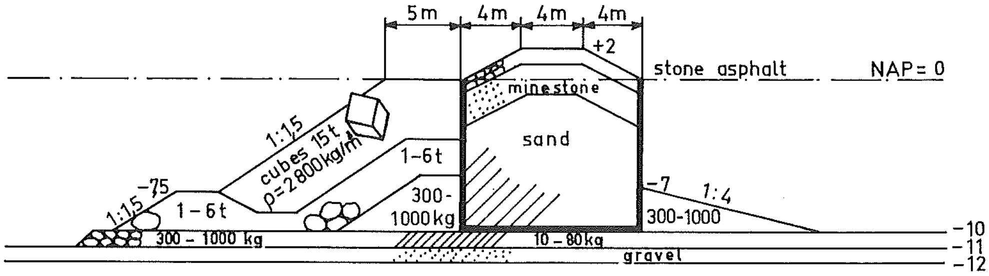
g STONE ASPHALT RUBBLE MOUND



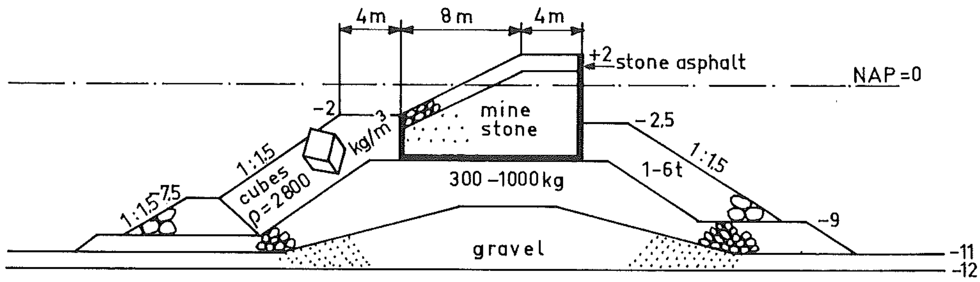
h CONCRETE CUBES WITH RETAINING WALL



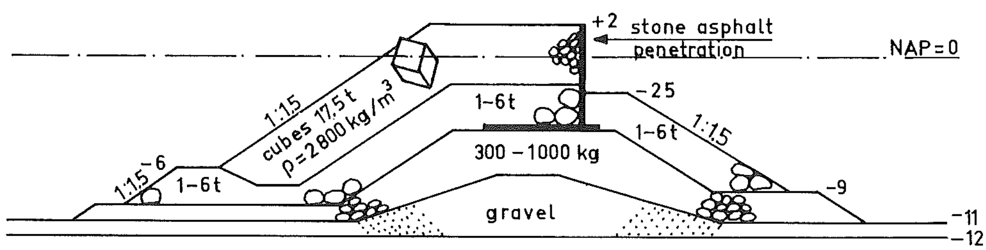
i CAISSON CUBES



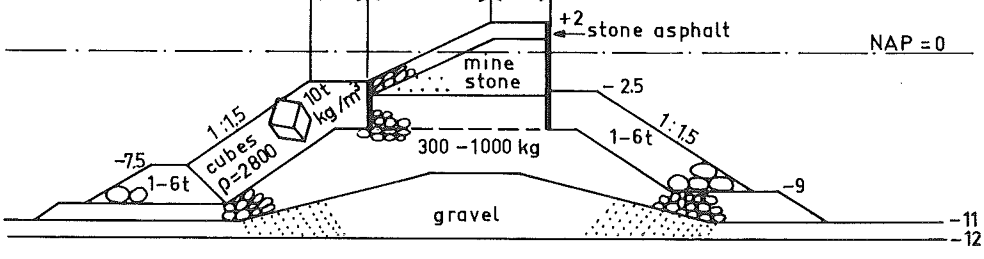
j. RETAINING WALL ON RUBBLE MOUND



k. RETAINING WALL ON RUBBLE MOUND



l. CONCRETE CUBES WITH CREST STRUCTURE



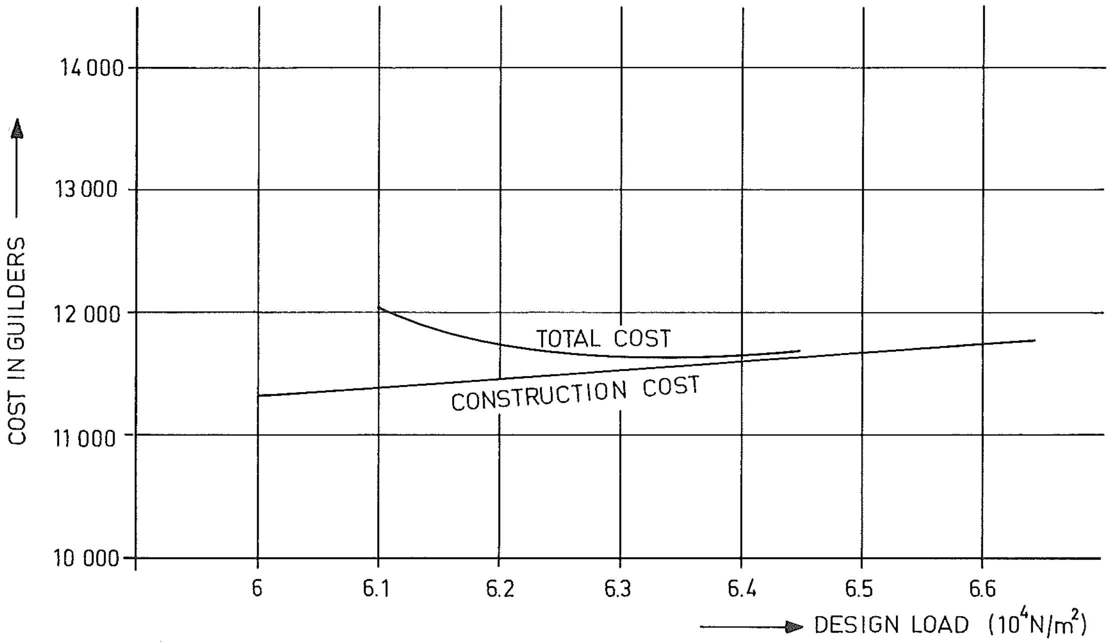


Figure 20.3a
OPTIMIZATION CURVE FOR „HANSTHOLM" CAISSON

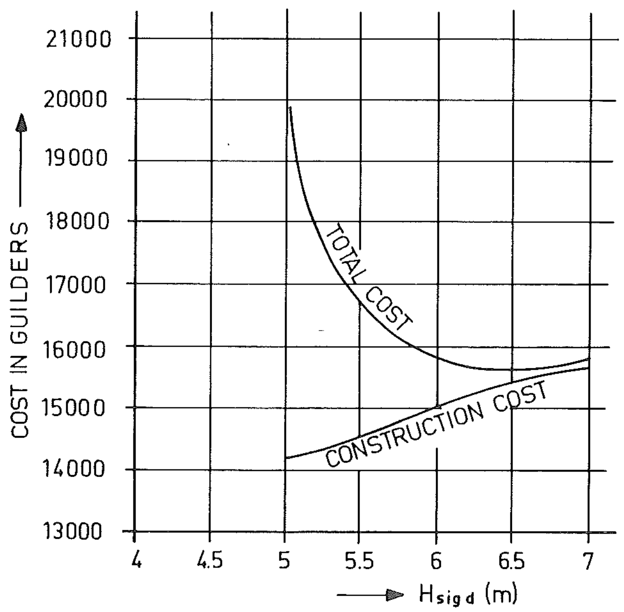
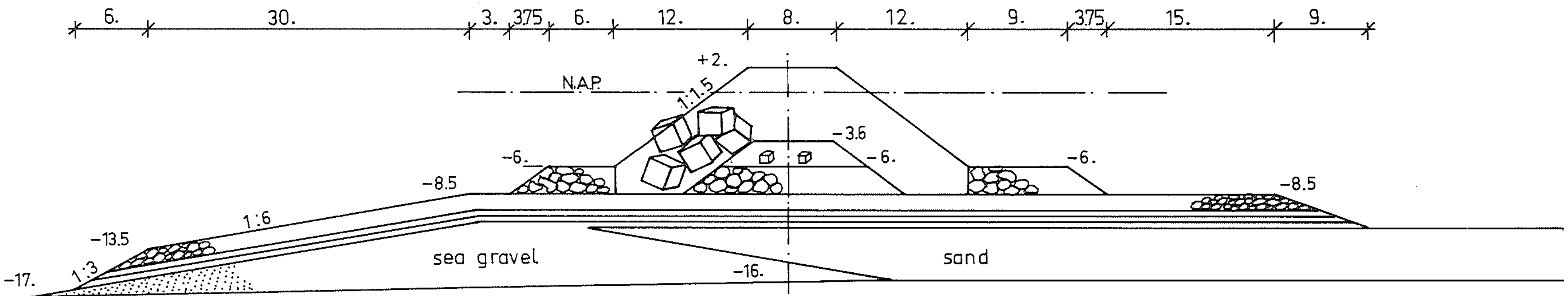
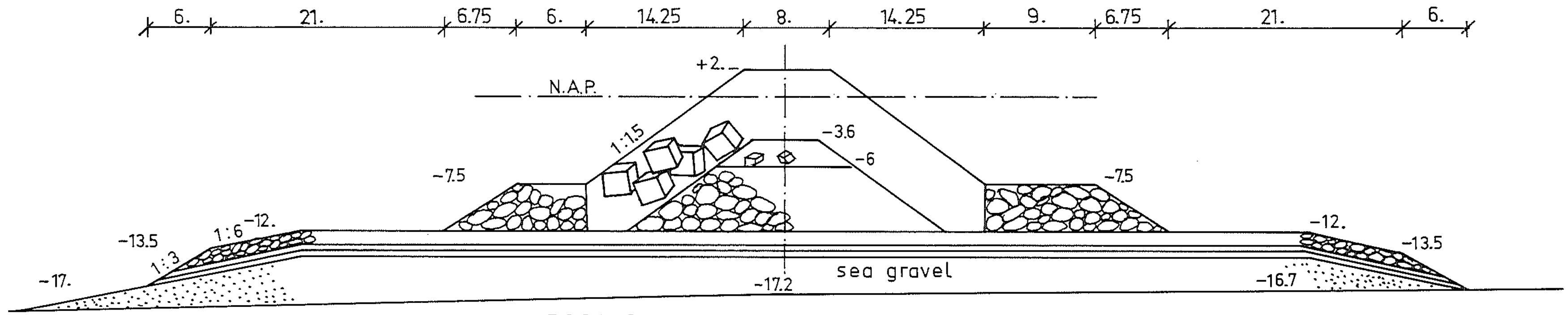


Figure 20.3b
OPTIMIZATION CURVE FOR CUBE RUBBLE MOUND



CROSS SECTION C–C IN FIGURE 20.1



CROSS SECTION D–D IN FIGURE 20.1

Figure 20.4

CROSS SECTIONS OF NORTH BREAKWATER

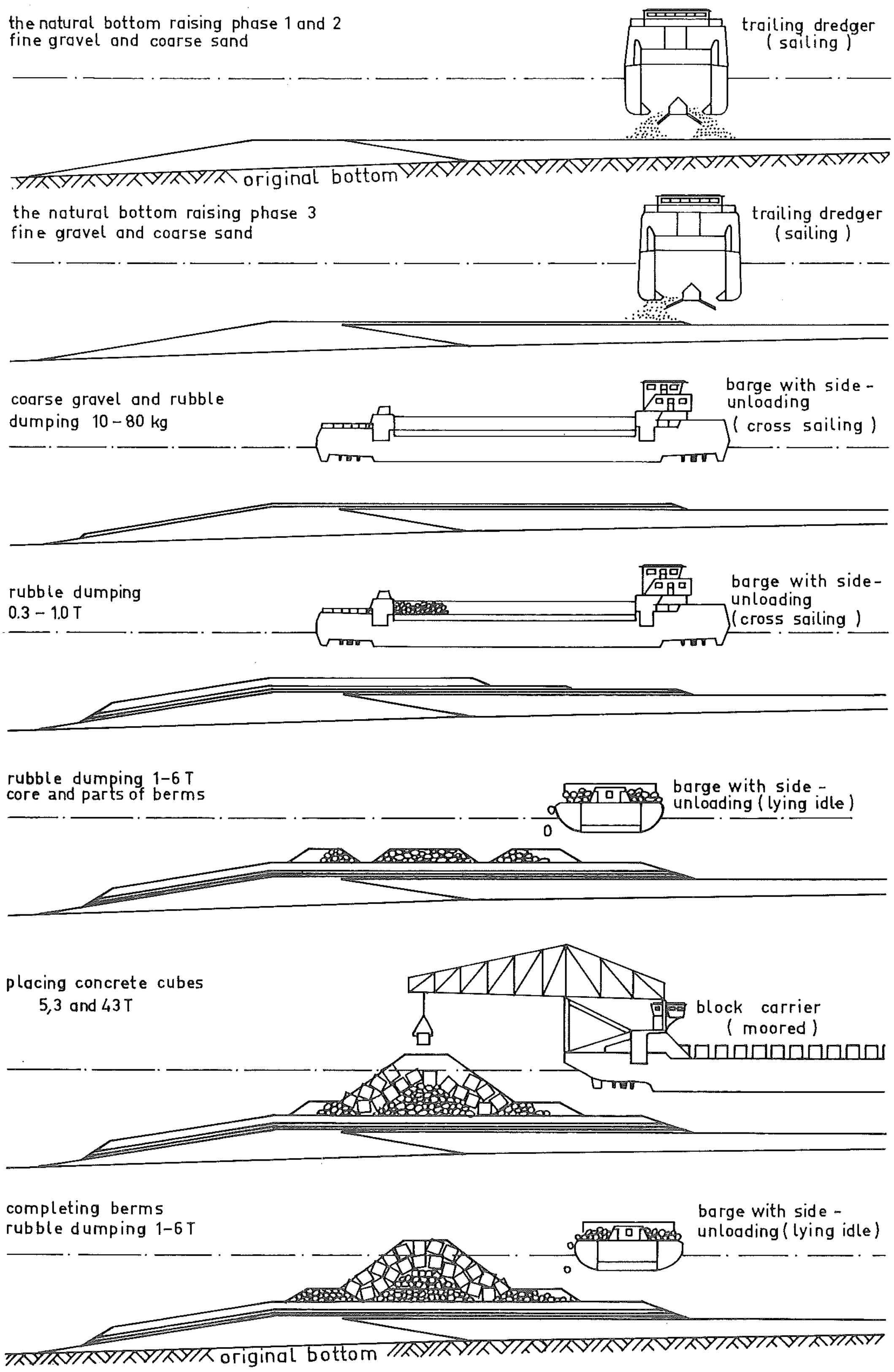


Figure 20.5
(CONSTRUCTION) PHASES OF NORTH BREAKWATER

SYMBOLS AND NOTATION W.W. Massie

The symbols used in this set notes are listed in the table. International standards of notation have been used where available except for occasional uses in which direct conflict of meaning would result. Certain symbols have more than one meaning, however this is only allowed when the context of a symbol's use is sufficient to define its meaning explicitly. For example, T is used to denote both wave period and temperature.

Functions are denoted using the British and American notation. The major discrepancy with European continental notation occurs with the inverse trigonometric functions. Thus, the angle whose sine is y is denoted by:

$\sin^{-1} y$ instead of arc sin y.

Possible confusion is avoided in these notes by denoting the reciprocal of the sine function by the cosecant function, csc, or by $\frac{1}{\sin}$. This same rule applied to the other trigonometric and hyperbolic functions as well.

In the table a meaning given in capital letters indicates an international standard. The meaning of symbols used for dimensions and units are also listed toward the end of the table.

Roman Letters

Symbol	Definition	Equation	dimensions	Units
A	CROSS SECTIONAL AREA	-	L^2	m^2
a	coefficient	7.02	-	-
a_B	acceleration	16.01	LT^{-2}	m/s^2
B	bouyant force	16.18	MLT^{-2}	N
b	coefficient	7.08	-	-
b	breakwater width	16.40		
C	number of armor units per unit surface area	7.22	L^{-2}	$1/m^2$
C_x	contact force	16.11	MLT^{-2}	N
c_ϕ	spring constant	16.02	ML^2T^{-2}	Nm/rad
c_x	spring constant	16.03	MT^{-2}	N/m
c_z	spring constant	16.02	MT^{-2}	N/m
d	block "diameter"	7.01	L	m
e	BASE OF NATURAL LOGS		-	-
F_x	force in x direction	16.03	MLT^{-2}	N
F_z	force in z direction	16.02	MLT^{-2}	N

Symbol	Definition	Equation	dimensions	Units
F_w	wave force	16.17	MLT^{-2}	N
F_F	friction force	16.21	MLT^{-2}	N
f	friction force	7.04	MLT^{-2}	N
f	failure in P(f)	19.58	-	-
g	ACCELERATION OF GRAVITY		LT^{-2}	m/s^2
H	WAVE HEIGHT	7.01	L	m
H_d	design wave height	19.02	L	m
H_i	incident wave height	15.01	L	m
H_{sig}	significant wave height	11.03	L	m
H_{sig_o}	significant wave height at deep water	11.04	L	m
H_t	transmitted wave height	5.04	L	m
H_x	maximum progressive wave component			
H_o	wave height at deep water	fig. 5.02	L	m
H^*	unknown wave height	7.19	L	m
h	WATERDEPTH	tab.11.3	L	m
h_a	depth to toe of armor	11.05	L	m
h'	waterlevel	tab.11.1	L	m
h^*	total height of breakwater	19.07	L	m
I_B	virtual inertia	16.04	ML^2	kgm^2/rad
I_{wy}	virtual inertia	16.04	ML^2	kgm^2/rad
I_{sy}	virtual inertia	16.04	ML^2	kgm^2/rad
i	subscript index	16.04	ML^2	kgm^2/rad
J	constant	19.40	-	-
K	constant	19.43	-	
k	WAVE NUMBER $2\pi/\lambda$	15.01	L^{-1}	1/m
L	Length of impacting mass	15.03	L	m
M	mass	16.05	M	kg
M_y	moment	16.04	ML^2T^{-2}	Nm
m	number of layer of armor units	7.21	-	-
m'	number of units across crest	7.23	-	-
m_B	breakwater mass	16.02	M	kg
m_s	virtual soil mass	16.01	M	kg
m_w	virtual water mass	16.03	M	kg
N	normal force	7.03	MLT^{-2}	N
N	number of waves	tab.19.3	-	-

Symbol	Definition	Equation	dimensions	Units
N'	dynamic normal force	16.18	MLT^{-2}	N
n	slope porosity	5.01	-	-
P( )	probability of ( )	19.02	-	-
p	pressure	15.01	$ML^{-1}T^{-2}$	N/m^2
R	run up	5.01	L	m
R	hydraulic radius	15.05	L	m
r	slope roughness	5.01	L	m
T	PERIOD (wave)	5.01	T	s
t	TIME	15.01	T	s;hr
t	Layer thickness	7.21	L	m
u	COMPONENT VELOCITY IN X DIRECTION		LT^{-1}	m/s
V	TOTAL VELOCITY	15.02	LT^{-1}	m/s
v	COMPONENT VELOCITY IN Y DIRECTION	16.24	LT^{-1}	m/s
W	breakwater weight	16.18	MLT^{-2}	N
W_{sub}	block weight	7.03	MLT^{-2}	N
w	COMPONENT VELOCITY IN Z DIRECTION		LT^{-1}	m/s
X	COORDINATE DIRECTION		L	m
x	COORDINATE DIRECTION	16.01	L	m
x	horizontal displacement	16.32	L	m
Y	COORDINATE DIRECTION		L	m
y	COORDINATE DIRECTION	16.01	L	m
Z	COORDINATE DIRECTION		L	m
z	COORDINATE DIRECTION	15.01	L	m
z_c	crest elevation above SWL	5.02	L	m

GREEK LETTERS

Symbol	Definition	Equation	dimensions	Units
α	breakwater slope	5.01	-	rad.
β	foreshore slope	5.01	-	rad.
γ	breaker index	11.02	-	-
Δ	RELATIVE DENSITY	7.11	-	-
ζ	time interval	15.03	T	s
ε	dynamic pressure coefficient	16.20	-	-
θ	slope angle	7.03	-	rad.
λ	WAVE LENGTH	fig.5.2	L	m
μ	friction coefficient	7.04	-	-
π	3.1415926536		-	-
ρ	DENSITY OF WATER	7.01	ML^{-3}	kg/m^3
ρ_a	density of armor	7.08	ML^{-3}	kg/m^3
ρ_B	density of breakwater	16.63	ML^{-3}	kg/m^3
ρ_s	density of soil	16.01	ML^{-3}	kg/m^3
ϕ	angular rotation	16.04	-	rad.
ϕ	angle of internal friction	16.22	-	rad.
ω	circular frequency	15.01	T^{-1}	rad/s
ω_n	natural frequency	16.05	T^{-1}	rad/s

Special symbols

ℓ	structure life	11.30	T	years
i	interest rate	11.29	-	-
p	wave breaking parameter	11.02	-	-
c	speed of sound in water	15.03	LT^{-1}	m/s
$\hat{}$	amplitude of	16.17		
pwf	present worth factor	11.29	-	-

Subscript

Symbol	Definition	Equation
a	armor	7.08
B	breakwater	16.02
c	crest	5.02
i	incident H_i	5.01
n	natural (frequency)	16.05
o	deep water	fig.5.2
s	soil	16.01
sub	submerged	7.03
t	toe of construction	5.01
t	transmitted	5.03
w	water	16.17
x	x component	16.03
y	y component	16.04
z	z component	16.02

Functions used

	Trigonometric functions
sin( )	sine of ( )
cos( )	cosine of ( )
tan( )	tangent of ( )
$\sin^{-1}$( )	angle whose sine is ( ) *
$\cos^{-1}$( )	angle whose cosine is ( )
$\tan^{-1}$( )	angle whose tangent is ( )

* The reciprocal of sin( ) would be denoted by csc( ) cosecant ( ).

<u>hyperbolic functions</u>

sinh( )	hyperbolic sine of ( )
cosh( )	hyperbolic cosine of ( )
tanh( )	hyperbolic tangent of ( )
$\sinh^{-1}$( )	argument whose hyperbolic sine is ( )
$\cosh^{-1}$( )	argument whose hyperbolic cosine is ( )
$\tanh^{-1}$( )	argument whose hyperbolic tangent is ( )

<u>logarithmic functions</u>

log( )	logarithm to base 10 of ( )
ln( )	logarithm to base e of ( )
exp( )	e raised to the power ( )
P( )	probability of exceedance of ( )
f( )	general function of ( )
Π( )	product of ( )
Σ( )	sum of ( )

Dimensions and units

Symbol	Definition
^oC	degree celsius
cm	centimeter = 10^{-2} m
ft	foot
g	GRAM
h	hour
hr	hour
kg	KILOGRAM
km	kilometer = 10^{3} m
kt	knot = nautical miles per hour
L	LENGTH DIMENSION
lb	pound force
M	MASS DIMENSION
m	METER
mg	milligram = 10^{-3} g
mm	millimeter = 10^{-3} m
μm	micrometer = 10^{-6} m
N	NEWTON
rad	radians
s	SECOND
T	TIME DIMENSION
yr	year
o	degree temperature degree angle
$^o/oo$ %o	parts per thousand

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