Breakwater design

Lecture notes CIE5308

H.J. Verhagen
J.P. van den Bos (Ed.)

DRAFT EDITION January 2017

Delft University of Technology

TU Delft Delft University of Technology

Challenge the future

BREAKWATER DESIGN

LECTURE NOTES CIE5308

by

H.J. Verhagen
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Lecture notes
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Department of Hydraulic Engineering
Faculty of Civil Engineering and Geosciences

Delft University of Technology

Preface

These lecture notes are a temporary edition of an intended textbook for "Breakwater design". Please read any reference to "this book" as "these lecture notes".

This book is primarily a study book for graduate students. It has been prepared for students in Coastal Engineering at the Delft University of Technology. The consequence is that, in addition to treating the latest insights into the subject matter, it places the developments in their historic perspective, at least when this contributes to better understanding. It also means that this book cannot replace comprehensive textbooks or original scientific publications. The book focuses on understanding of the design process, but is certainly not a design manual. The reader is strongly advised to consult the original references rather than blindly following this textbook.

In the curriculum of Delft University, the course on breakwaters is preceded by a variety of courses on subjects such as fluid mechanics, hydraulic engineering, coastal engineering and bed, bank and shore protection, design process, and probabilistic design. Therefore it is assumed that the reader is familiar with this knowledge and it will not be discussed in detail in this book.

Breakwaters, and specifically various kinds of rubble mound breakwaters, underwent a tremendous development in the period 1985-1995. After that, the pace of innovation seemed to slowing down, although monolithic breakwaters were gaining attention in the following decade. In the most recent years focus of research was on the effect of shallow water conditions, optimising the use of the quarries (the Icelandic breakwaters) as well as research on variations on the rubble mound breakwater, like the (semi-)submerged structures, breakwaters with a longer berm and new concrete elements. Therefore, the present study book does not represent a static subject. This necessitates that both the teacher and the student should continuously observe the latest developments.

The first edition of this book (2001) was written by Kees d'Angremond and Ferd van Rooden. The second edition has been updated by Henk Jan Verhagen (2009). That revision added the treatment of wave statistics, the spectral approach in the stability formula, the shallow water conditions and the Icelandic breakwaters, and brought the book in line with the Rock Manual (2007) and with the European Standard on Armourstone (EN 13383).

In the previous editions this book combined the design of breakwaters and closure works. Since 2016, the closure works have become the topic of a separate online course (with an accompanying textbook). Therefore the chapters on closure works have been removed, and the remaining text has been reworked into these present lecture notes. These are intended to be used for the TU Delft course CIE5308 "Breakwaters and Closure Dams" in the course year 2016-2017. The text of these lecture notes follows the text of the book to a large extent, with some re-written or additional material (most notably Appendix A).

Because some changes in the curriculum of the course are foreseen, this will have a drawback on the set-up of this book. Therefore this edition is a temporary edition.

Valuable contributions in the form of comments and/or text were received from: Marcel van Gent (Deltares), Jentsje van der Meer (independent consultant), Jelle Olthof (Delft University of Technology and Royal Boskalis Westminster), Gerrit Jan Schiereck, (Delft University of Technology), Sigurður Sigurðarson (Icelandic Maritime Administration) and Shigeo Takahashi (Japanese Port and Airport Research Institute). Many others contributed in a variety of ways, including correcting text and preparing figures. A special thanks to Margaret Boshek, who checked both the English spelling as well as the readability of the book.

Also, our fellow members of the teaching staff for CIE5308 (Bas Hofland, Coen Kuiper and Greg Smith) are acknowledged for their comments and contributions to these present lecture notes.

Henk Jan Verhagen	Delft, October 2016
Jeroen van den Bos	Delft, January 2017

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1
Introduction

1.1. Scope
For this book we have deliberately chosen that the text should follow a more or less logical design procedure for breakwaters. It follows the required design steps from the system level down to the individual cross-section level, and in time from conceptual design to construction. This systematic approach starts at the functional description of breakwaters, system analysis including side-effects and derivation of boundary conditions and continues to the actual design, first of the main armour layer and then of the rest of the cross section.It finishes with an overview of construction methods, materials and constructability issues. All existing breakwater types are discussed briefly but only the types that are frequently used all over the world (i.e. rubble mound breakwaters, berm breakwaters and monolithic breakwaters) are treated in detail. It is expected that the reader will possess basic knowledge of hydraulic engineering. Only in some cases, where they are deemed useful for a proper understanding of the actual design process, some aspects of basic hydraulic engineering are presented.

1.2. References
This book is an educational textbook, not a design manual neither a reference book. The focus of this book is on the transfer understanding of the basic principles. It is not an overview of all existing formulas. Also because the results of new research will change details of existing formulas, it is not useful to focus on details of such formulas, but more on the physical concepts behind the formulas. Although a study book has its own right to existence, there are some outstanding reference books in the field treated by this textbook and these are often far more com¬prehensive than any study book can be. Therefore a number of books and periodicals that should be available to anybody who will ever be in charge of design or construction of breakwaters are mentioned here. Such books include: Coastal Engineering Manual [US ARMY CORPS OF ENGINEERS, 2002]), The Rock Manual (CIRIA/CUR/CETMEF [2007]) and various PIANC/MarCom Working Group reports. Useful periodicals include the Journals of the ASCE, the journal "Coastal Engineering" (from Elsevier) as well ass the "Coastal Engineering Journal" (from World Scientific) and the proceedings of the international conferences on Coastal Engineering. Additional educational material (PowerPoint presentations, videos) is on-line availa-ble via the educational platform of TU Delft (http://blackboard.tudelft.nl). To have guest access to this website, one should not log-in, but click on "courses" and search for "ct5308".

1.3. Miscellaneous
To avoid misunderstandings, a glossary of the terms used in this book is added as Appendix 8. For Dutch students an English-Dutch glossary is available on the above mentioned "blackboard" site. The reader is also referred to a more general vocabulary on hydraulic engineering (http://www.waterdictionary.info). In this book, the metric (mks) system (based on the definition of mass [kg], length [m], and time [s] has been used, except for some widely accepted nautical and hydrographic terms such as knots, fathoms and miles.

2

POSITIONING THE SUBJECT

2.1. GENERAL

Breakwaters are widely used throughout the world. This type of structure is primarily designed for the protection of vessels harboured within ports and for port facilities from wave action, but sometimes breakwaters are also used to protect beaches from erosion or to protect valuable habitats that are threatened by the destructive forces of the sea. Although the threat is usually a product of wave action, protection against currents is also important. Additionally, breakwaters can prevent or reduce the siltation of navigation channels. In some cases, breakwaters also accommodate loading facilities for cargo or passengers.

Non-technical aspects, including environmental, social and cultural values, cannot be expressed in financial terms. The evaluation of such considerations is not within the scope of this book. Nevertheless, engineers must identify the consequential effects to the best of their ability and present them in such a way that they are understood by decision-makers.

2.2. TYPES OF BREAKWATERS

There are many different types of breakwaters that can be divided into categories according to their structural features:

2.2.1. MOUND TYPES

Mound types of breakwaters are simply large heaps of loose elements, such as gravel and quarry stone or concrete blocks. The stability of the exposed slope of the mound depends on the ratio between load and strength i.e. wave height (H) versus size and the relative density of the elements (Δd). On one extreme, for example, is a gravel beach that is subject to continuous changes in the equilibrium profile as the wave characteristics change and also due to longshore transport. On the other extreme, for example, is the 'statically stable breakwater', where the weight of the elements in the outer armour layer is sufficient to withstand the wave forces. Between these two extremes is the 'berm breakwater', where the size of the armour is not sufficient to guarantee stability under all conditions, but where some extra quantity of material is provided so that the slope of the structure can reshape between given limits. Typical values of $H/\Delta d$ for the three types of structures are given in Table 2.1.

Type of structure	$H/(\Delta d)$
Sandy Beach	> 500
Gravel Beach	$20 - 500$
Rock slope	$6 - 20$
Berm Breakwater	$3 - 6$
(Stable) Rubble Mound Breakwater	$1 - 4$
Caisson	< 1

Table 2.1: Characteristic values of $H/(\Delta d)$

2.2.2. MONOLITHIC TYPES

Monolithic breakwaters have a cross-section which acts as one solid block. Types of monolithic structures include caissons, a block wall, or a masonry structure. This type of structure can be categorized by a typical value of $H/\Delta d$ that is given (as caisson) in Table 2.1. The main differences between the mound and the monolithic types of breakwaters are caused by the interaction between the structure and the subsoil and also by the behaviour at failure. The mound-type structures can be considered flexible (i.e. they can follow uneven settlement of the foundation layers), whereas monolithic structures require a solid foundation that can cope with high and often dynamic loads. The behaviour of the structures when close to failure is also quite different. When a critical load value is exceeded, a monolithic structure will lose stability at once, whereas a mound type of structure will fail more gradually as elements from the armour layer are displaced. However, because of the sloped construction, the footprint of a rubble mound breakwater is much larger. Where construction restrictions related to depth or environmental issues are a concern a vertical wall breakwater may be the better option.

2.2.3. COMPOSITE TYPES

Composite types of breakwaters combine a monolithic element with a low-crested berm composed of stable loose elements. In fact, there are an abundance of composite breakwater designs that combine a rigid element and a flexible structure.

2.2.4. SPECIAL (UNCONVENTIONAL) TYPES

Many methods can be used to break wave action other than the traditional types defined above. These include:

- Floating breakwaters
- Pneumatic breakwaters
- Hydraulic breakwaters
- Pile breakwaters
- Horizontal plate breakwaters

All these unconventional breakwaters have been implemented or their use has been proposed, in exceptional cases under exceptional conditions. Under standard conditions their use usually appears to be either infeasible or uneconomic. Floating, pneumatic and hydraulic breakwaters require either large dimensions or a lot of energy to damp longer period waves that occur at sea. Usually they are only economic in case of relative small waves in very deep water (e.g. in 2016 edition 11 the Italian lakes). Pile breakwaters and horizontal plate breakwaters require very high structural strength to survive wave loads under extreme conditions.

Apart from a division between the categories described so far, there is also a distinction in terms of the freeboard of the crest above the still water level (SWL [1]). Traditional structures usually have a crest level that is only overtopped occasionally. It is also possible to choose a lower crest level that is overtopped more frequently, or even completely submerged. When a low crest level is combined with the design philosophy of a berm breakwater, (i.e. a reshaping mound) it is termed a reef-type breakwater. Examples of all types of breakwaters are shown in Figure 2.1 to Figure 2.4.

2.3. HISTORICAL BREAKWATERS

The first breakwaters that are described in traceable sources date back to ancient Egyptian, Phoenician, Greek and Roman cultures. Most of them were simple mound structures, composed of locally found rock. As early as 2000 BC, mention was made of a stone masonry breakwater in Alexandria, Egypt (TAKAHASHI [2002]). The Greeks also constructed breakwaters (mainly rubble mound) along some parts of the Mediterranean coast. The Romans constructed true monolithic breakwaters once they had mastered the technique of making concrete. The Roman emperor Trajan (AD 53 - 117) initiated the construction of a rubble mound breakwater in Civitavecchia, which still exists today (Figure 2.5). The very flat seaward slope and the complicated superstructure are proof of a history of trial and error, damage and repair (VITRUVIUS [27 BC]; SHAW [1974] BLACKMAN [1982]; DE LA PENA, PRADA AND REDONDO [1994]; FRANCO [1996])

In modern times similar breakwaters were constructed at Cherbourg (France) (1781/1789/1830), and at Plymouth (UK) (1812/1841). In both cases, the stability of the seaward slope was insufficient and during subsequent repair operations the final slopes were between 1:8 and 1:12 (See Figure 2.6 and Figure 2.7).

[1] SWL is the water level that would exist in the absence of sea and swell (instantaneous mean water level in the absence of waves).

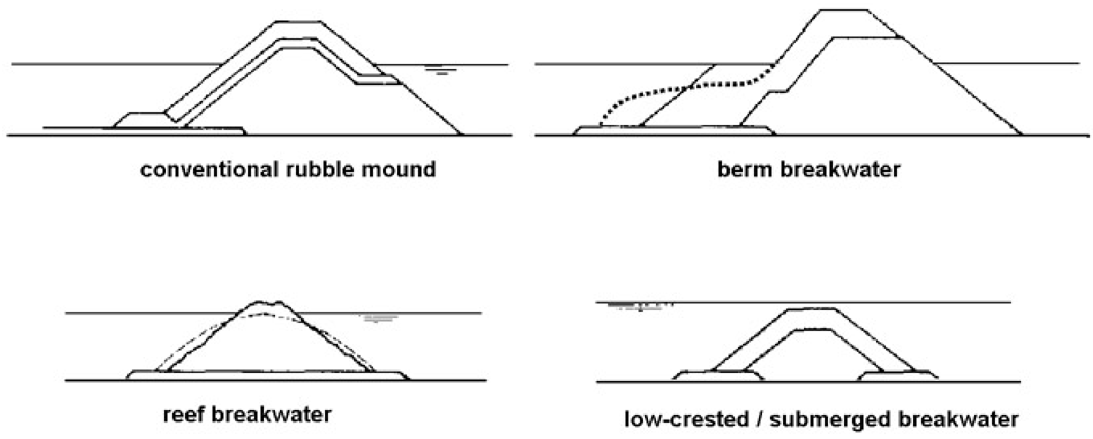


Figure 2.1: Mound breakwater types

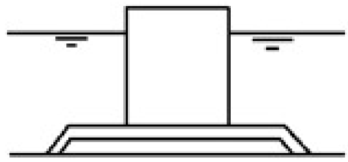


Figure 2.2: Monolithic breakwater type

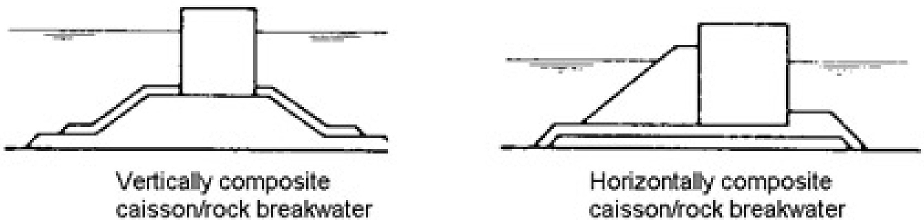


Figure 2.3: Composite breakwater types

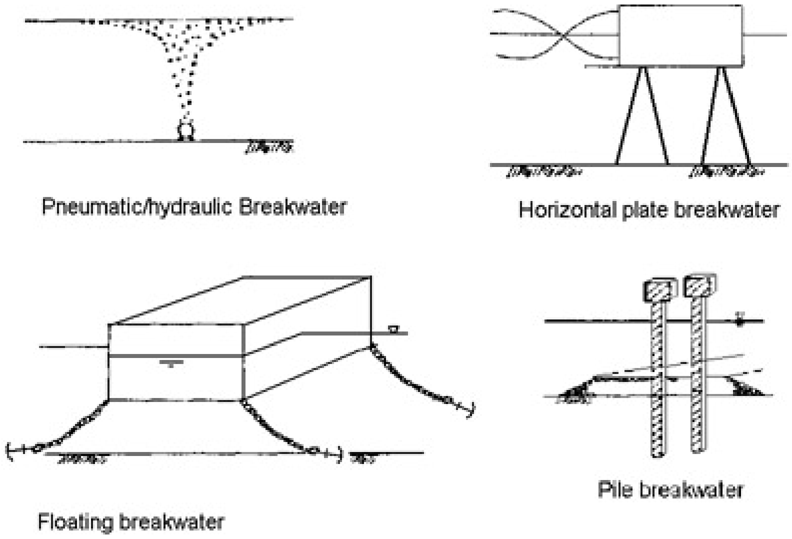


Figure 2.4: Special breakwater types

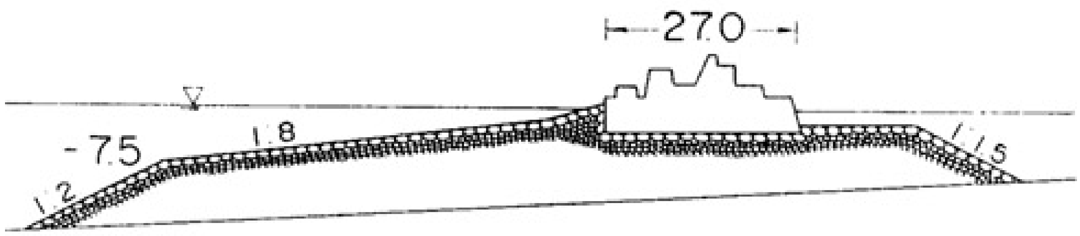


Figure 2.5: Rubble mound breakwater at Citavecchia

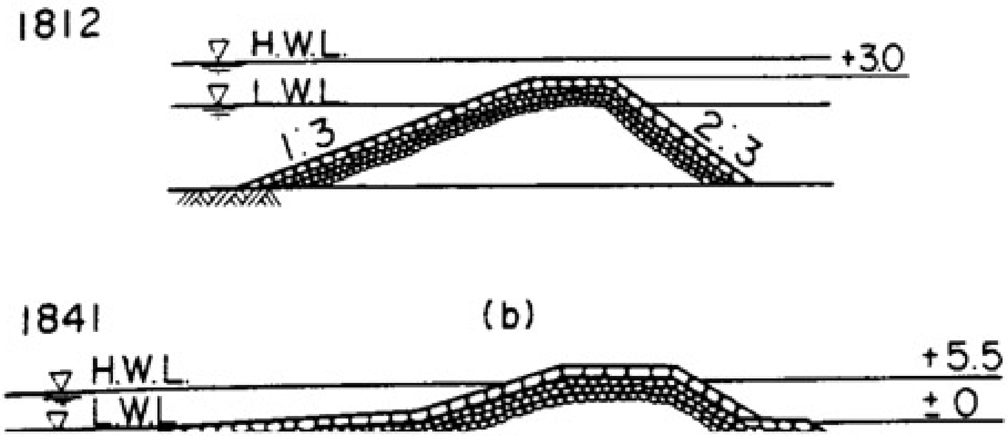


Figure 2.6: Breakwater at Plymouth

In view of the difficulties encountered in Cherbourg and Plymouth, it was decided, in 1847, that a monolithic breakwater should be built at Dover. The construction posed a lot of problems, but the result was quite satisfactory as this breakwater has survived without major damage (Figure 2.8)

The rapidly increasing sea-borne trade in the 19th century led to a large number of breakwaters being built in Europe and in the emerging colonies to protect an expanding fleet of vessels. British engineers, in particular, put the lessons learned from the Dover breakwater construction to use. To avoid the problems of construction in deep water, rubble mound berms were used for the foundation of monolithic superstructures, and thus the first real composite breakwaters came into existence. Here also, however, the process of trial and error took its toll. Many breakwaters had to be redesigned because the berms were originally erected too high and where subject to instability due to wave action.

In France, engineers tried to solve these stability problems by designing flatter slopes above SWL, and by applying extremely heavy (cubic and parallelepiped) concrete blocks as the armour layer. They also started to use smaller-sized stone in the core of the structure. The breakwater at Marseilles (1845) was seen as a success among French engineers just as the Dover type of breakwater was a success for the British. However, it was recognized that the Marseilles type of solution required very heavy armour units and also a lot of material in the cross section, especially in deeper water (Figure 2.9).

These developments made the composite breakwater the most widely used breakwater type in the early 20th century. In Italy, where many breakwaters were constructed in relatively deep water along the Mediterranean coast, the logical solution appeared to be a composite structure consisting of a berm to about half the water depth with a vertical faced wall on top of it. The wall was built of extremely large (Cyclopean) blocks, sometimes interlocking to create the monolithic effect (Figure 2.10). However, these breakwaters were not a success. The shallow berm caused waves to break and slam against the vertical wall causing high impact forces which led to the eventual failure of the breakwater itself.

Concerns over these failures led to the creation of an international association for hydraulic research (IAHR) by PIANC [2] port engineers. The failures of the vertical-wall breakwaters around the Mediterranean

[2] Permanent International Association of Navigation Congresses.

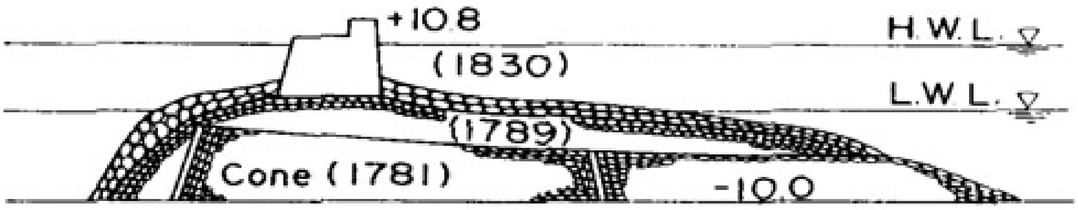


Figure 2.7: Breakwater at Cherbourg

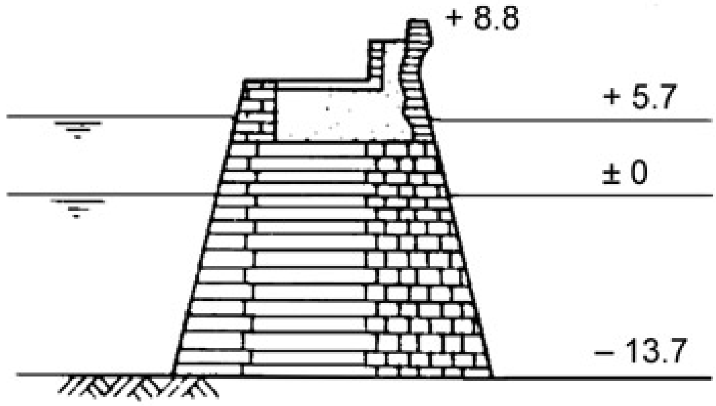


Figure 2.8: Monolithic breakwater at Dover

in the first half of the 20th century marked the end of the popularity of this type of breakwater in western Europe.

The French continued their efforts to optimize their rubble mound concept. To reduce the required weight of the armour blocks, they developed the concept of interlocking them. Thus, in 1949, P. DANEL [1953] of the Laboratoire Dauphinois d'Hydraulique (later Sogreah, now Artelia) designed the Tetrapod armour unit, which was the start of a long series of similar blocks. For a time, the Dolos designed in South Africa, seemed to provide the ultimate solution, until the limited mechanical strength of this block triggered a new series of mishaps. The development of special shaped blocks went on, however, resulting in two other French blocks, which are still quite successful: the Antifer cube and the Accropode. In the US, a stronger version of the Dolos unit was developed, the Core-Loc. In the Netherlands, Delta Marine Consultants created the Xbloc.

In the meantime, the Japanese continued to build and develop monolithic breakwaters. In no other country have so many monolithic and composite breakwaters been built, with varying success. The principal contribution, however, was made by a French engineer, G.E. JARLAN [1961], who introduced the perforated front wall to reduce reflection and wave impact forces.

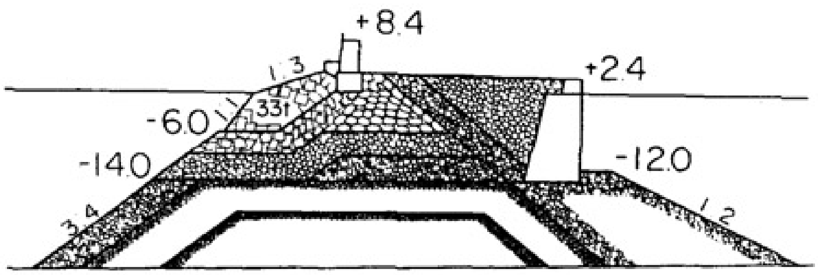


Figure 2.9: Breakwater at Marseilles

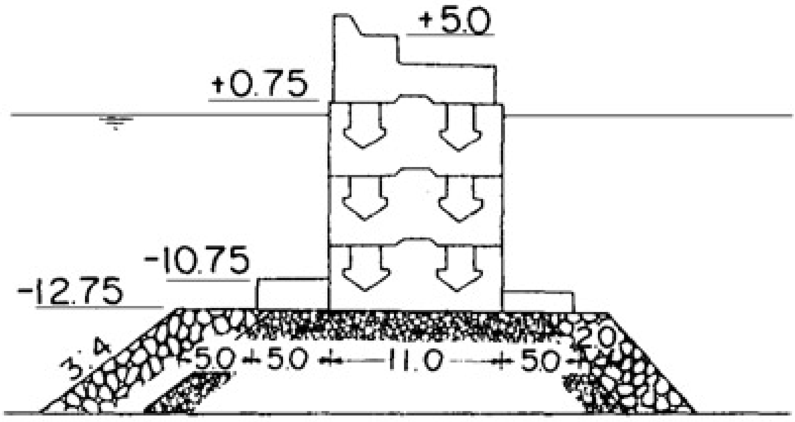


Figure 2.10: Typical breakwater along the Mediterranean Coast

3 THE DESIGN PROCESS

In the context of the subject "breakwaters" some aspects of the design process have been omitted from this book. It is assumed that certain decisions have already been taken at a different level, be it only on a preliminary basis. For the breakwater, these decisions concern the question whether a new port should indeed be built and, if so, at which location, and for what kind of traffic. This does not mean that no strategic choices have to be made. However, the strategic choices no longer refer to the questions of whether and where the structure should be built but rather to how it should be built.

3.1. GENERAL

In the design process both the functional as well as the structural design has to be looked into. This implies that one has to design a construction which fulfils the functional requirements but also ensure that the construction will not fail, collapse, or be seriously damaged with a predefined probability. The objective of the design process is to find a concept that meets the requirement(s) and that can be realised; not only in terms of technical feasibility, but also in terms of cost-benefit ratio and social and legal acceptance. This implies that the solution of the design process must combine the following elements:

- Functionality
- Technology (what is feasible)
- Environment (what is allowed or accepted)
- Cost and benefit
- Paper work (drawing board)
- Matter (actual construction)

3.2. ABSTRACTION LEVEL

In any design process various levels of abstraction can be discerned. In most cases it is sufficient to distinguish three levels:

- Macro level: the system
- Meso level: a component of the system
- Micro level: an element of one of the components

A few examples are presented in Table 3.1. The indication of three levels does not mean that a very complex problem should always be divided into three levels. It is very useful to discern one level that is higher than that on which the actual work takes place and one level that is lower. This enables the designer to refer certain questions to a higher level in the hierarchy and it enables him to leave certain non-essential items to a later stage or to a lower level in the organisation.

When considering the planning of a port, one may distinguish various levels of abstraction including:

- Design of a world or regional concept for the transport of certain commodities
- Design of regional or national economic plans
- Design of a national or provincial zoning policy

	Macro level	Meso level	Micro level
General terms	System	Component	Element
Example 1[a]	Harbour in the global and regional transport chain	Harbour layout	Breakwater
Example 1[b]	Harbour layout	Breakwater	Crest block

Table 3.1: Examples of different scale levels

Phases	Abstraction Level		
	System	Component	Element
Initial	Purpose		
Feasibility	Functionality	Purpose	
Preliminary Design	Shape	Functionality	Purpose
Final Design	Specifications	Shape	Functionality
Detailed Design		Specifications	Shape

Table 3.2: Schematization of the design process

- Design of an overall port plan with intermodal facilities
- Design of the breakwater for such a port plan
- Design of a quarry to provide stone for the breakwater
- Design of the workshop for maintenance of the equipment of the quarry

3.3. PHASES

During the design process, one can also recognise certain design phases that in some countries are related to the general conditions of contract between employer and consultant. Therefore the phases may vary from country to country. The contractual contents of each phase are subject to modifications in the same way. A logical set of phases are:

Initiative Formulation of the ultimate goals of the design object as part of the system.

Feasibility Review of the system with respect to technical, economic, social and environmental consequences and feasibility. Requirements are formulated on the component level.

Preliminary design Giving shape to the system on broad lines, including determination of the exact functionality of the components and definition of requirements at the element level.

Final design Composition of a set of drawings and specifications for the system in which the final shape of the components is fixed and the functionality of the elements is determined.

Detailed design Composition of a set of drawings and specifications in which the final shape of the elements is fixed.

This concept can easily be schematised in a matrix in which each row represents one of the phases and shows which activities will take place at the various levels of abstraction. The columns show how the levels of abstraction in the project become more concrete throughout the phases. The matrix also shows that working on the elements does not start before one reaches the preliminary design phase and certain decisions have been taken about the purpose and function at the system level and about the purpose at the component level.

Following this line of thought helps to ensure that the proper approach is chosen at each stage so that neither too much nor too little detail is sought.

3.4. CYCLIC DESIGN

Each activity in the design process, which is represented by a cell in Table 3.2, is a cyclic process in its own right, consisting of a number of steps.

Analysis Assembling of available data and arranging for the provision of missing data; Drawing up a set of criteria that the design must fulfil (List of Requirements) and crosschecking all with respect to cost and functionality.

Synthesis Generation of conceptual ideas and alternatives that broadly meet the requirements.

Simulation Detailing of concepts and alternatives (by calculation, simulation, or modelling) up to a level that makes them mutually comparable. Again, a crosscheck with respect to cost and functionality is required.

Evaluation Assessment of the concepts and alternatives and comparison on the basis of cost and benefit.

Decision Selection of the best option. If more than one option is acceptable, repeat the process in further detail until a final decision can be taken. This may involve some toggling between the abstraction levels in a particular phase of the design process.

3.5. Consequences of systematic design

It is obvious that a systematic design procedure is essential. It makes no sense to draw a cross-section of a breakwater when neither the depth of the water in which it is to be built nor the acceptable wave action in the lee of the structure is known. One has to start by considering the purpose of the system, i.e. its national or regional socio-economic role in the global transport system. From there, one goes down a step to the port, still as part of the system:

- which cargo flows are foreseen
- which type of vessels will carry the cargo
- what are the requirements for access from the seaward side and from the landward side
- what will be a proper size of the port
- what will be a suitable location

When these questions have been answered, can one start to think in more detail about specifics such as the breakwaters, starting with a rough layout and an indication of the required functions. Only in the final stage of the design process, can the actual design of the cross-section be made, including decisions about crest level, slope, and choice of materials and construction method.

Considering these remarks, one can conclude that a study book on the design and construction of breakwaters deals with the final stages of the design process for the structure itself. Notwithstanding, for a proper understanding of what one is doing, throughout the process the link has to be maintained with the higher abstraction levels. If one fails to do this, the risk emerges that one teaches students to apply prescriptive recipes, instead of designing creative solutions. For this reason, relatively much attention will be given to the link between the purpose and functionality of the system. At the same time, it will be clear that certain details of the design need not be worked out in the early stages. It makes no sense to plan a working harbour in detail before the closure method has been chosen.

3.6. Probabilities

No construction can be designed in such a way that it will never fail. However, the probability of failure has to be very small. The probability of failure of a structure is partly a financial problem (the extra cost of lowering the probability of failure has to be lower than the capitalised cost of failure), and partly depends on non-monetary values, such as loss of life, ecological damage, etc. In case probability of failure is mainly a financial problem, the optimum probability of failure can be computed; this will be explained later. In case numerous non-monetary values are at stake (e.g. a dike protecting an urbanised area or a natural reserve), an objective optimisation is not possible, and usually a political choice is made regarding the allowable probability of failure.

After the feasibility study and preliminary design, the details of the design have to be filled in. As discussed before, this will be done during the stage of the detailed design and sometimes already during the stage of the final design. Basically, this means that each structural part should not fail or collapse within a degree of probability, as follows from the boundaries as set in the feasibility study.

3.6.1. BASICS OF A PROBABILISTIC ANALYSIS AND THE USE OF SAFETY COEFFICIENTS

A structure fails when the load is larger than the strength. In other words, if:

$$Z = R - S < 0 \tag{3.1}$$

where R is the strength and S is the load. [1] Usually R consists of a number of parameters (e.g. material properties) and S consists of a number of load values. In a very simple design, this problem can be solved easily. For example, if one needs to design the cable in a crane, the design force in the cable F is equal to the design mass, multiplied with the acceleration of gravity. The strength of the cable depends on the intrinsic strength (σ) of the cable material, multiplied with the cross sectional area A of the cable:

$$\begin{array}{ll} \text{strength:} & R = A \cdot \sigma \\ \text{load:} & S = M \cdot g \\ & Z = R - S = A\sigma - Mg \end{array}$$

For critical conditions (brink of failure) $Z = 0$. The critical cross sectional area (which, in fact, is the design parameter) is

$$A_{\text{crit}} = \frac{Mg}{\sigma} \tag{3.2}$$

M is the mass of the nominal load to be lifted (design load). This is a clear input parameter, it is defined by the client; σ is prescribed in the specifications and g is the gravitational acceleration. Because there are always uncertainties, in the traditional design process a safety coefficient γ is added:

$$A_{\text{crit}} = \gamma \frac{Mg}{\sigma} \tag{3.3}$$

The magnitude of γ is usually given in professional codes and standards; if not, it is usually based on experience (in case of breakwater design, PIANC has issued values of γ to be used in the design; see chapter 7.5). The safety coefficient γ covers the following uncertainties:

- the actual mass being different from the nominal mass;
- deviations in the value of g, the acceleration of gravity;
- the actual strength of the material σ being different from the specified strength;
- the actual cross section of the cable A being different from the specified cross section.

In more complicated cases, and specifically when there are no codes or when experience is lacking, a probabilistic approach should be implemented, which will be explained later (see Appendix 1).

3.6.2. ADDITIONAL PROBLEM IN COASTAL ENGINEERING

Unfortunately in the design of coastal structures there is a complicating factor. For example the stability of armour units depends on the wave height (H_s), the mass of rock or concrete, the slope of the structure, and many other parameters. In a stability calculation, the wave height is the load parameter, while the other parameters (mass of rock or concrete, slope, shape of the armour, etc.) are strength parameters. Often, the strength parameters are Gaussian distributed with a relatively small standard deviation. So, at the strength side of the equation, the problem is very comparable to the cable example mentioned above.

But for the load parameter (H_s) an "average" value cannot be determined. It has to be a significant wave that does not occur too often. And related to the wave height there is also the wave period (which is usually also present in the more advanced design equations). It means that the definition of our "design wave" or "design storm" is a key problem in our design.

The choice of the probability of the "design storm" is usually the most important parameter decision in the design process. In choosing this probability two cases have to be distinguished:

1. It is a pure economic problem.
2. Also human lives and other non-monetary values are taken into account, such as pro-tection of a museum or a religious site.

[1] S as a symbol for load and not for strength does not seems logical, but it is according to international agreement. R and S are acronyms related to the French words Résistance and Sollicitation ("asking"). We will adhere to this agreement, despite the confusion at first glance.

In the first case, one can calculate the optimal design conditions based on economic restrictions. In the second case, these values cannot be calculated but are subject to political decision making. Typically, for breakwaters, it is purely an economic problem. In case of failure there will be damage: the cost of repairing the direct damage plus the loss of income during non-operation of the breakwater (consequential damage). The details of the economic optimization will be explained in Appendix 6 of this book.

Often such an economic optimization is not made. This is usually due to the fact that decisions on the investments for a breakwater project are not based on proper life-cycle analysis, but on the budget available or on the (short-term) rate or return on the initial investments. Therefore in practice often a political decision is made on the return period of the design storm, based on ad-hoc considerations.

3.6.3. DETERMINATION OF A DESIGN STORM

Usually the design storm is related to the economic lifetime of the structure. For breakwaters, an economic lifetime in the order of 50 years is very common. As a result, decision makers often suggest using the once in 50 years storm as a design storm. The first task for the design engineer is to explain to the decision maker that this does not mean that the design storm will occur after exactly 50 years, but that every year there is a probability of 1/50 (i.e. 2%) that the design storm will occur, which could be next year. The second task for the design engineer is to explain that the probability of serious damage during the lifetime of the construction is given by the Poisson distribution:

$$p = 1 - \exp\left(-f\,T_L\right) \tag{3.4}$$

in which:

p probability of occurrence of an event one or more times in period t_L
T_L considered period (e.g. the lifetime of the breakwater) in years
f average frequency of the event per year

So the assumed lifetime of 50 years and a storm frequency of 1/50 per year, gives

$$p = 1 - \exp\left(-\frac{1}{50}\cdot 50\right) = 1 - \exp(-1) = 0.632$$

This means that there is probability of 63% that the construction will fail during its lifetime. It is clear that this is unacceptable. More acceptable values would be between, say, 5% and 20%. The actual choice depends largely on the purpose of the structure and on the risk involved. In this book, some examples have been worked out based on the relatively high value of 20%. [2] This must not be interpreted as a recommendation, but just as an example!

It means that the storm frequency becomes:

$$\begin{aligned} f &= -\frac{1}{t_L}\ln(1-p) \\ &= -\frac{1}{50}\ln(1-0.2) \\ &= 0.0044 = 1/225 \end{aligned}$$

In case one accepts a probability of failure of 20% during a lifetime of 50 years, one should apply a $1/225 = 4.4\cdot10^3$) per year storm. So realize that in spite of the fact that we did allow (a rather high) 20% probability of failure during lifetime, still we use a design storm with a probability of $4.4\cdot10^3$ per year in our calculations. In the above text, it has been assumed implicitly that the probability of storms has some statistical distribution, but that all other parameters (notably the strength parameters) are fixed, deterministic values. Of course, this is not true. The combined effect of all these uncertainties will be discussed in Section 7.3. It will be shown that the effect of the uncertainty in strength parameters is much less than the uncertainty in the storm occurrence, but not negligible. Because determination of the parameters of the design storm is extremely important for the design, this will be discussed separately in Section 5.3.

[2] The value of 20% is selected because this value is also used in the examples in various PIANC publications; from economic analysis often will follow that values of p in the order of 5% are more economic.

4

Considerations at system level

In this chapter the actual design of breakwaters is linked to considerations and decisions that in fact belong to a different abstraction level than does the design itself. From these links, it is often possible to derive considerations with respect to the functionality of the structure under consideration. Attention is paid to the side-effects of the construction works, which may lead to a reconsideration of decisions taken earlier. For students, this chapter is an indispensable tool to establish the quantified functional requirements for the design of a breakwater or closure dam. It is therefore essential to study this chapter in detail before any design exercise is attempted.

4.1. General

In Chapter 3, it was indicated that a design problem should be considered at various levels of abstraction, starting with the system. In this chapter we attempt to discuss some of the aspects at system level, where the system is either a port or a scheme to close a river or estuary. The breakwater or the closure dam is then an element of that system. By discussing the system, we attempt to approach our design problem from a slightly more abstract position. This refers to both the functions and requirements, and to the side effects of the project.

4.2. Functions of breakwaters and examples

Breakwaters can fulfil a variety of functions; the most important of which are:

- Protection against waves. This can be subdivided into protection of ports and shipping and shore protection.
- Guiding of currents.
- Protection against shoaling.
- Provision of dock or quay facilities .

4.2.1. Protection against waves

Vessels at berth

The function of protection against wave action must be split into sub-categories. The best-known protection function relates to navigation and over the years breakwaters have been used in port construction. However, the status of the vessels (sailing with or without tugs,moored, being loaded/unloaded) or installations that are to be protected makes a big difference to what is required. In other words, one must have an idea how vulnerable the area to be protected is before deciding what degree of protection must be provided.

In general, a vessel is most vulnerable when it is moored alongside a rigid structure such as a quay, a jetty, or alongside another vessel. The acceptable wave height is related to the size of the vessel, on one hand, and the height, period and direction of the waves, on the other hand. THORESEN [2003] gives suggestions for ships at berth in head seas. These values are slightly modified in Table 4.1 according to the experience of the authors. The acceptability of the conditions refers to both damage to the vessel and damage to the structure.

Loading and unloading operations may impose extra restrictions. It will be clear that loading and unloading liquid bulk cargo via a flexible hose allows larger ship movements than placing containers in a slot. Velsink

Type of vessel	Maximum H_s in m At berth (head sea)
Pleasure craft	0.15 - 0.25
Fishing vessels	0.40
Dredges and dredge barges	0.80 - 1.00
General cargo (< 30,000 dwt)	1.00 - 1.25
Dry bulk cargo (< 30,000 dwt)	1.00 - 1.25
Dry bulk cargo (up to 100,000 dwt)	1.50
Oil tankers (< 30,000 dwt)	1.00 - 1.25
Oil tankers (100,000 to 200,000 dwt)	1.50 - 2.50
Oil tankers (200,000 to 300,000 dwt)	2.50 - 3.00
Passenger vessels	0.70

Table 4.1: Maximum wave heights for ships at berth

Type of vessel	Limiting wave height H_s in m	
	0° (head or stern)	45° – 90° (beam)
General cargo	1.0	0.8
Container, Ro/Ro ship	0.5	
Dry bulk (30,000-100,000); loading	1.5	1.0
Dry bulk (30,000-100,000); unloading	1.0	0.8 – 1.0
Tankers 30,000 dwt	1.5	
Tankers 30,000 – 200,000 dwt	1.5 – 2.5	1.0 – 1.2
Tankers >200,000 dwt	2.5 – 3.0	1.0 – 1.5

Table 4.2: Maximum wave heights for loading and unloading operations

and Thoresen approach this question from a different angle. Thoresen gives values for acceptable ship movements; VELSINK [1987] gives limiting wave heights for different directions. The approach of Velsink relates more directly to the functional requirements of the breakwater. Therefore, his data are given in Table 4.2. A comprehensive review of the problem of ship movements is given in PIANC/MARCOM 24 [1995].

How often the exceeding of these limits is accepted is not indicated in the above figures. In other words, they do not indicate for what percentage of time loading and unloading operations may be interrupted, or how often specific berths must be left by vessels needing to find a safer place to ride out a storm. This question must be answered on the basis of a thorough economic analysis, including the risk of negative publicity for the port. Such studies are beyond the scope of this book, but nevertheless the answer to the question must be known when the design of the actual breakwater is started. The point stressed here is that these considerations will lead to the definition of Serviceability Limit State (SLS) that are usually different from the Ultimate Limit State (ULS), which concerns the survival of the structure under extreme conditions.

Figure 4.1 shows the layout of a harbour where the breakwater typically protects the harbour basin, including berths for loading and unloading.

Sailing vessels

So far, we have considered the protection required by vessels at berth. Free sailing vessels are fortunately much less vulnerable.

National regulatory bodies, like the Netherlands Shipping Inspectorate, strictly control the operation and the design of ocean going vessels. The work of these national organizations is coordinated by the International Maritime Organization, IMO. In addition to the Government-related regulatory bodies, there are also private regulatory bodies that check the design of vessels, often on behalf of the insurers. Such private bodies include Bureau Veritas, Det Norske Veritas, and Lloyds. These bodies issue certificates of seaworthiness, with or without certain restrictions.

Ocean-going vessels with an unrestricted certificate are designed to cope with the highest waves. In severe conditions they may adapt their course and speed to the prevailing wind and wave direction, but in principle, modern vessels with an unrestricted certificate can survive the most severe conditions at sea. The situation changes when a free choice of course and speed becomes impossible, for instance because of the proximity of land, the need to sail in a specific (dredged) fairway, or the wish to come to a halt at a mooring or anchorage. The more confined the conditions, the stricter will be the limits with respect to wind, waves and currents.

What applies to vessels designed to sail the high seas without restriction does not apply to all categories of vessels. Some vessels have a certificate that limits their operation to certain areas (coastal waters, sheltered

Figure 4.1: Harbour of Marseilles (France)

waters, and inland waters) or to certain periods in relation to certain areas (North Atlantic summer). Such restrictions refer not only to the structural aspects of the vessel, but also to skill and number of crew.

What does all this mean for the operation of a port, and for the functional requirements of its breakwater? Can a vessel enter the port under any circumstances? Obviously not, but we have already concluded that a sailing vessel is less vulnerable than a moored vessel. The functional requirements for a breakwater that protects only an entrance channel are thus much lower than those for a breakwater that protects a harbour basin. Still, the actual situation will change from place to place. If ships need the assistance of a tug during the stopping operation and the subsequent turning or mooring, the waves must be attenuated to a level that makes tugboat operation feasible. In general, one can assume that a significant wave height of 2 to 2.5 m is acceptable for tugs and their crews working on deck. If only tugs with an inland waters certificate are available, their operation may be restricted to significant wave heights of 1 to 1.5 m. If the limits imposed by the certificate are exceeded, often the insurers will not cover the cost of damage.

Figure 4.2 shows an example of a breakwater, which does not protect any berths.

Here again, decisions must be made as to how frequently interruption of the navigation due to closure of the port for weather conditions can be accepted. One must realize that pilotage also becomes a limiting factor under heavy sea and swell conditions. In general, delays and interruptions are accepted of one or two days per year.

PORT FACILITIES

A third condition that needs attention is the harbour basin itself, with the facilities that may suffer damage if the wave heights in the basin become too high. Quays and jetties and the equipment that is installed on them may be damaged, even in the absence of vessels. Here again, it must be decided whether any such damage is acceptable, and if so what chance of its occurrence is acceptable. It is evident that if the harbour installations are damaged, one is concerned not only about the direct cost of repair but also about the consequential damage due to non-availability of the cargo transfer systems. In this respect one may try and imagine what happens if the only power plant or refinery in a region must be closed because no fuel can be supplied.

SHORE PROTECTION

From coastal engineering theory, we know that waves cause both longshore transport and cross-shore transport. Both phenomena can cause unwanted erosion, especially on sandy shores.

As far as cross-shore transport is concerned, the erosion is often connected with changes in the equilibrium profile. A more gentle profile (after the erosion of dunes) is associated with higher incoming waves, whereas a milder wave climate tends to restore the beach by landward sediment transport. Similarly, when erosion is due to a gradient in the longshore transport, the effect will be less when the wave heights are lower.

In general terms one can therefore conclude that the reduction of wave heights in the breaker zone will mitigate beach erosion. Such reduction of wave heights can be achieved by constructing offshore breakwaters

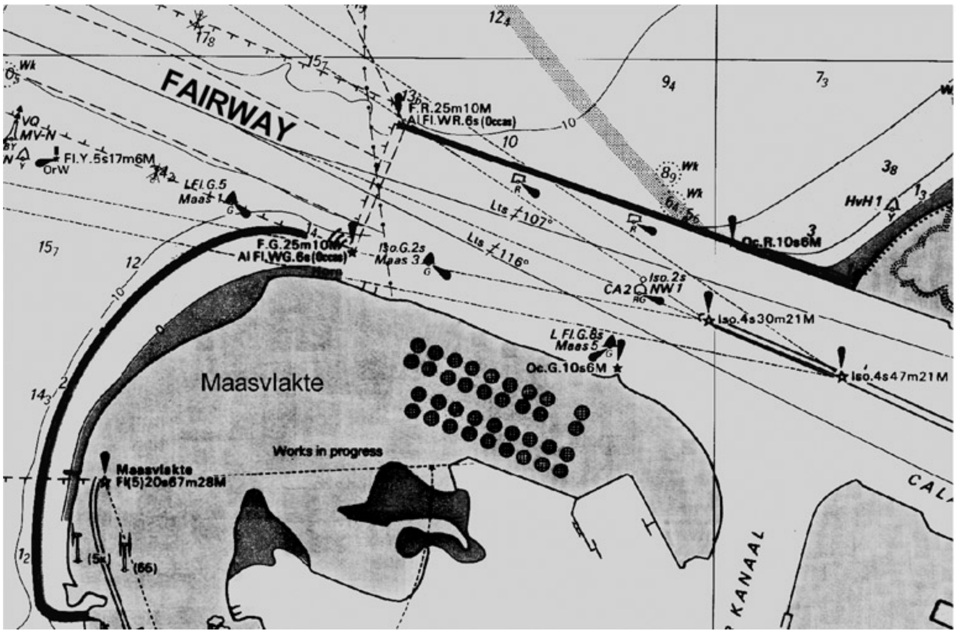


Figure 4.2: Breakwater at the Europoort entrance

parallel to the shore (Figure 4.3). However, from the literature it is known that one must be careful when using this solution. Due to wave set-up, the water level on the lee side of the breakwater rises, which causes a concentrated return current, (comparable with a rip current) between the breakwater sections (BOWDER, DEAN AND CHEN [1996]).

GUIDING OF CURRENTS

When approaching a harbour entrance, vessels are slowing down by reducing power. This is done because at high speed they require a rather long stopping distance and the vessels produce a high wave and a strong return current. A slower speed means that the vessel is more affected by a cross current (or a crosswind), since the actual direction of propagation is the vectorial sum of the vessels own speed and the current velocity. Thus, to sail a straight course into the port along the axis of the approach channel the vessel must move more or less 'crab-wise'.

Closer to the shore, at the same time one must expect stronger tidal currents parallel to the shore. If the port entrance protrudes into the sea, there will possibly be a concentration of flow lines near the head of the breakwater.

The combination of the slower speed of the vessel with the potentially stronger cross currents at the harbour entrances poses manoeuvrability problems. In the lee of the breakwater tugs can assist the vessel, but it takes some time (about 15 minutes) before the tugs have made a connection with the vessel, and in the meantime the vessel continues to sail without external assistance. Assuming a speed of 4 knots, the vessel travels a distance of about 1 nautical mile (1850 m), before the tugs can control the course of the vessel. Only then can the remaining stopping procedure be completed. The vessel gives full power astern and it will stop within 1 to 1.5 times its own length.

This means that cross currents are critical over a considerable distance that extends from well outside the harbour entrance to the point where tugs assume control. It is not only the velocity of the cross current that is important but also the gradient in the cross current, since this forces the ship out of its course.

The entrance to the Port of Rotterdam is a good example of an entrance where the layout of the breakwater is designed to cope with the current pattern (Figure 4.4). In this case, the function of the breakwater is twofold: it guides the current and it damps the waves to a level at which the tugs can work.

4.2.2. PROTECTION AGAINST SHOALING

Many ports are located at a river mouth or in an estuary. Coastal engineers are aware that the entrance channel has an equilibrium profile that is mainly determined by the tidal prism. (D'ANGREMOND AND PLUIM VAN DER VELDEN [2006]). If the natural depth in the entrance channel is insufficient for nautical purposes,

Figure 4.3: A system of detached breakwaters at Fiumicino, Italy

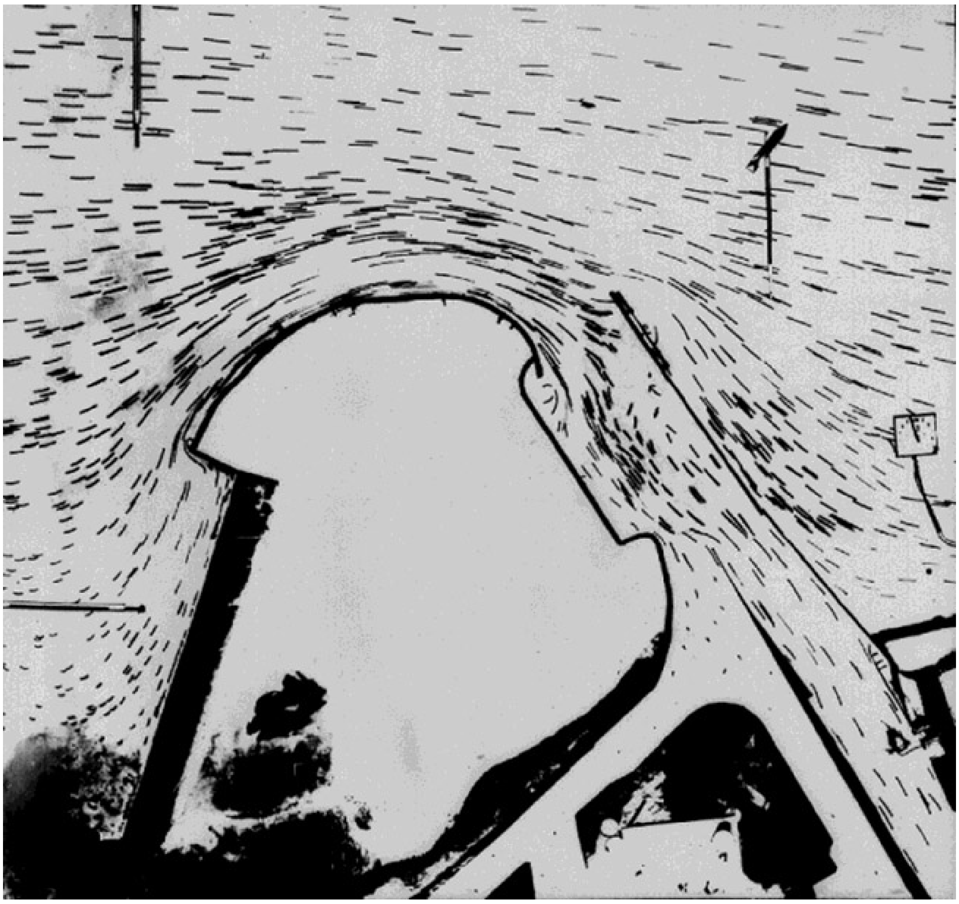


Figure 4.4: Flow pattern at the Europoort entrance

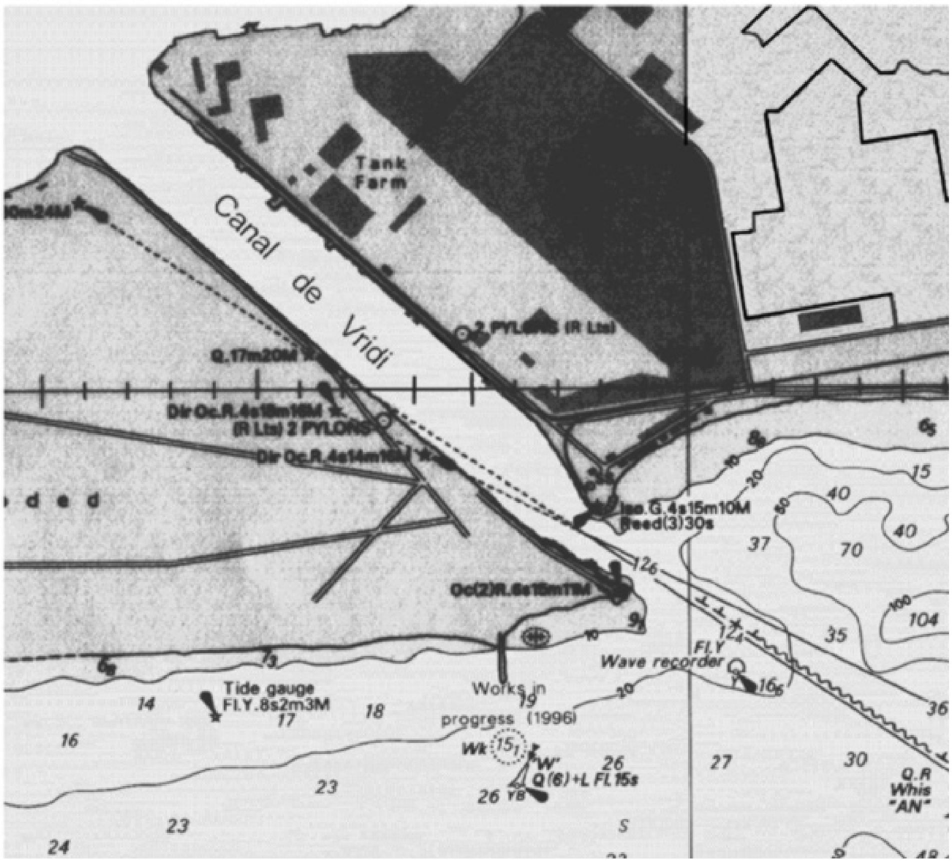


Figure 4.5: Entrance to the port of Abidjan

one may decide to deepen the channel by dredging. Though this may be a very good solution, disturbance of the equilibrium means that dredging has to be continued throughout the life of the port. In a number of cases it has therefore been decided not to dredge, but rather to restrict the width of the natural channel and to force the channel to erode its bed. This may also be the functional purpose of a breakwater that is designed to guide currents. An example of the use of such a solution is the port of Abidjan (Figure 4.5 and Figure 4.6).

It is stressed here, that improvement of the efficiency of dredging and the lower cost of dredging operations have caused a shift away from building breakwaters towards accepting the annual cost of dredging.

Another challenge for those designing entrance channels into a port is the existence of the longshore current along sandy shores. Under the influence of oblique waves, a longshore current develops in the breaker zone. Due to the high turbulence level in the breaker zone, a large quantity of sand is brought into suspension and carried away by the longshore current (longshore drift).

The sand will be deposited at places where the velocity is less, i.e. where the water depth is greater because of the presence of the shipping channel. Thus a dredged or even a natural channel may be blocked after a storm of short duration and high waves or after a long period of moderate waves from one direction. To avoid this, a breakwater can be constructed. For proper functioning, the head of the breakwater must extend beyond the breaker zone, in which case, sand will be deposited on the "upstream" side of the breakwater, whereas erosion will take place at the downstream side. In coastal engineering this is the classical example of erosion problems due to interruption of the longshore transport. A good example is given in Figure 4.7, which shows the actual situation in IJmuiden (The Netherlands).

Even if the breakwater is present, sedimentation of the port's entrance channel may occur. This happens when so much sediment has been deposited on the upstream side of the breakwater that the accumulated material reaches the end of the breakwater and passes around it's head. Dredging is difficult in such cases because of the proximity of the breakwater. An example of a breakwater that is too short is the breakwater of Paradip (India), shown in Figure 4.8.

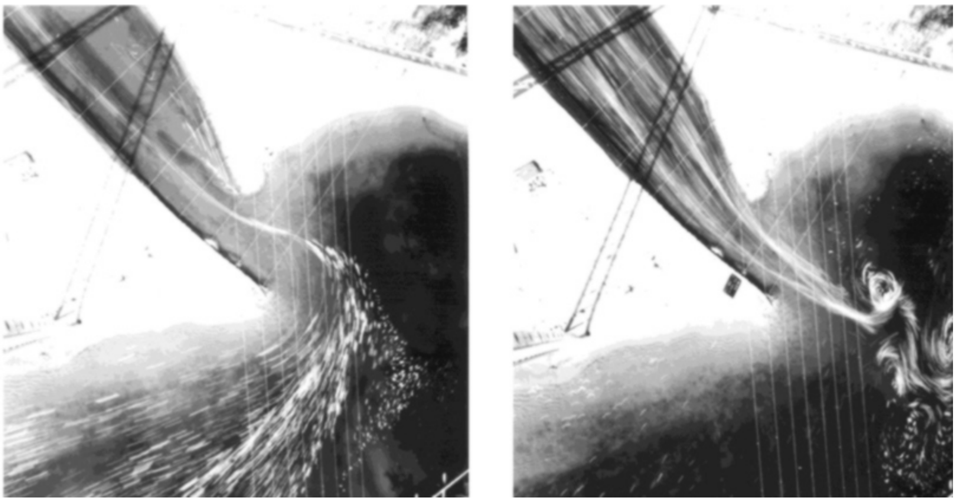


Figure 4.6: Flow pattern at the port of Abidjan

4.2.3. PROVISION OF DOCK OR QUAY FACILITIES
When the breakwater is directly protecting a harbour basin (and therefore already quite high), it is especially attractive to use the crest of the breakwater for transport of cargo and passengers to and from moored vessels. Special facilities must be provided in this case to enable the vessels to berth alongside the breakwater. These facilities may consist of a vertical wall on the inside, or a piled or non-piled jetty connected to the breakwater. In this case, it must be ascertained that the conditions on or directly behind the crest of the breakwater are safe. Again a distinction can be made between operational conditions (Service Limit State or SLS) and extreme conditions like survival of the installations (Ultimate Limit State or ULS). Further details of acceptable conditions relating to run-up and overtopping are given in Chapter 10.

4.3. SIDE EFFECTS OF BREAKWATERS

4.3.1. FAILURE MODES
From the above it is clear that failure to fulfil the functional requirements (at system level) may be due to inadequacies:

- Layout of the breakwater (for example, location, length, orientation, width of the harbour entrance): Such deficiencies may lead to undesirable disturbance in the harbour basin, unsafe nautical conditions, or undesirable accretion or erosion.
- Shape of the cross-section (crest level, permeability for sand and waves): This will lead to similar problems and also to unsafe conditions at the crest of the structure.
- Structural design of the cross-section (stability under severe design conditions, ULS, or due to other unforeseen conditions that are listed in most textbooks on probabilistic design (see Chapter 15)): These deficiencies may lead to unforeseen problems in operation of the port, especially when the breakwater also acts as quay wall.

The present book will mainly discuss failure modes of the last two categories. It is stressed here that the choice of the crest level in relation to the functional requirements is one of the most important design decisions.

4.3.2. NAUTICAL CHARACTERISTICS
Since breakwaters usually have a function connected with navigation, it is of the utmost importance to ensure that the layout of the breakwater(s) and channel creates safe nautical conditions. A first impression may be obtained by following the PIANC/IAPH guidelines (PIANC/MARCOM 30 [1997]).

In practice, a design prepared on the basis of guidelines must always be checked with the aid of navigational models. In this respect there is a choice between physical scale models, real time computer simulation and fast time computer simulation. A discussion of the merits of these methods is beyond the scope of this book.

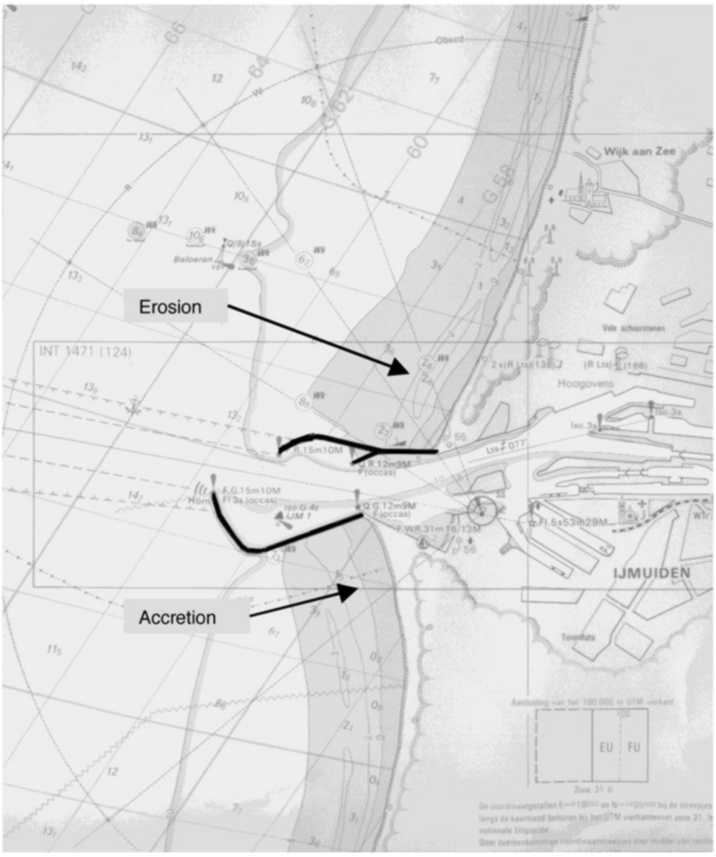


Figure 4.7: Port and breakwaters at IJmuiden

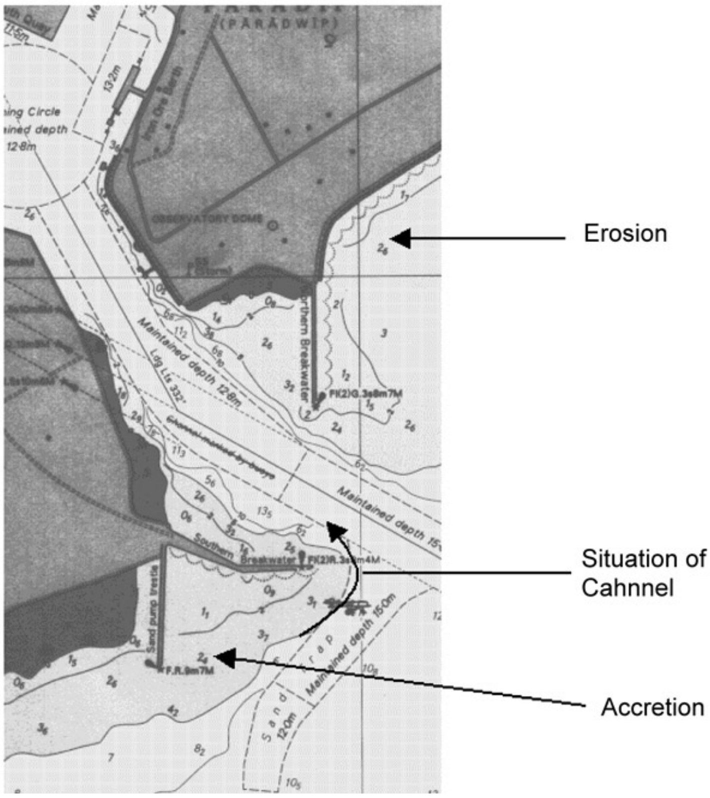


Figure 4.8: Siltation at the entrance to port of Paradip

In this respect, mention must be made of another side-effect of a breakwater that may influence the nautical environment: reflection of waves. Reflection of short waves may cause a choppy sea in the neighbourhood of the breakwater, which is a nuisance to smaller (often local and inland) vessels.

4.3.3. MORPHOLOGY

Although one of the purposes of a breakwater may be to interrupt the longshore sediment transport in order to prevent the siltation of a port entrance, a coastal engineer cannot ignore the consequences of this phenomenon in a larger space and time frame. Accretion and erosion of the coastal zone on either side of the breakwater will most likely pose a serious threat to the community in the region and possibly to the ecosystem as well. It goes without saying that such consequences have to be assessed and quantified, and that remedial measures have to be designed, planned, and executed. In this respect, one may think of:

- an adequate sand-bypassing system;
- replenishing the eroding beach with sand dredged during maintenance operations;
- use of material dredged during port construction as a buffer against future erosion.

5

USE OF THEORY

This chapter gives an overview of the theoretical knowledge needed for the design of breakwaters. The text is intended mainly to refresh the reader's knowledge of results and formulae. It also attempts to present a direct link between the more theoretical considerations and practical applications. Derivations have largely been omitted. The results of very specific model investigations that present empirical relations (such as the stability of rubble mounds) are not treated here, but rather in other dedicated chapters. If the content of this chapter is not familiar, the reader is referred to textbooks on these subjects.

5.1. GENERAL

It is impossible to discuss the design of breakwaters without referring to certain subjects from the theory of Fluid Mechanics and/or Geotechnology. It is assumed that readers of this book have basic knowledge of these fields. The basics of tides should be familiar to the reader. It is assumed that the origin of the tide, tidal constituents, spring and neap tide, and diurnal and semi-diurnal tides are all known to the reader. It may also be useful to remind that the travelling speed of a tidal wave can be approximated by the celerity formula [1]:

$$c = \sqrt{gh} \tag{5.1}$$

which for the oceans is about 200 m/s. Travelling up the Atlantic to the North Sea, for instance, takes 24 hrs. On the continental shelf the depth of the water diminishes to 200 m. In the southern North Sea the depth is about 25 m, so the wave speed is reduced to 15 m/s. Shallows, funnel-shapes and upland discharges have impacts on the penetrating wave. Finally, the wave enters estuaries and river-mouths.

In deep water the velocity of the water due to the tide is small, but when approaching the coast, and especially in tidal inlets and estuaries, these velocities are considerable. For the closure of tidal inlets this is a very important aspect. The relation between the water level variation and the flow velocities is an important characteristic of the tide. In relatively short basins (length shorter than 0.05 times the tidal wave length), the two variables will be 90 degrees out of phase. At the moment of high tide, the basin is full and the inflow stops. This situation is reversed at low tide. At the moment of mean level, the ebb and flood flows are at their maximum. In long basins there is a propagating wave. In that case the slack water/still water after ebb or flood may lag behind for some hours. If so, the maximum flood flow occurs during higher water levels on average than the ebb flow. The mass of water entering the estuary during the flood period, the flood volume, has to flow out during the ebb period with lower levels. Ebb velocities are therefore generally the largest and follow the deep gullies[2].

Also the reader should be familiar with the basics of waves. It is assumed that linear wave theory is known, as well as the behaviour of regular waves near the coast (effects like shoaling, refraction, diffraction, breaking, reflection). In chapter 5.3 some attention will be given to wave spectra and the behaviour of irregular waves near the coast. Also some discussion will be presented on wave long term wave statistics. In the end of this chapter some geotechnical aspects are discussed.

[1] In spite of the large depth in the ocean, the formula for the shallow water conditions is used because the wave length of approximately 8000 km is much larger than the water depth

[2] For a semi-diurnal tide (with a period of 12.5 hrs) and a water depth in the estuary of approximately 10 m, the length of the tidal wave is cT, where $c = \sqrt{gh}$. The wavelength is 450 km, so tidal basins shorter than 20 km can be considered as “short” basins.

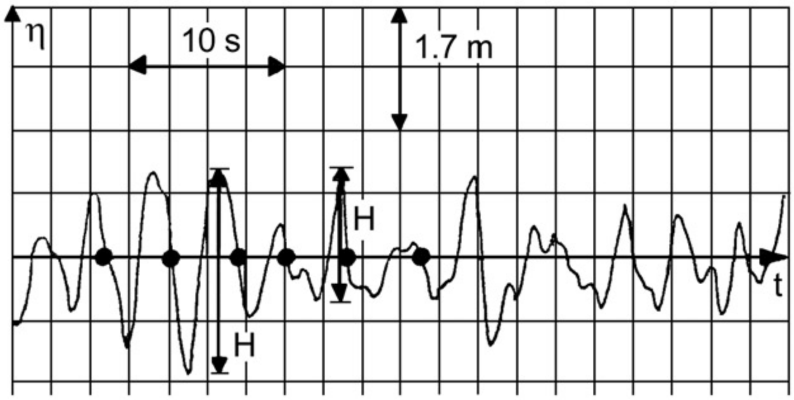


Figure 5.1: Irregular wave

5.2. WAVES

5.2.1. REGULAR WAVES

Knowledge of regular, small amplitude waves is essential for the understanding of loads on breakwaters. However, because this subject is treated in detail in many other courses, it is not repeated in this book. A summary of these topics is given in Schiereck [2001].

5.2.2. IRREGULAR WAVES IN DEEP WATER

Waves in nature are not small in amplitude and do not show regular character with respect to height H and period T. Therefore describing the behaviour of waves with the linear theory of regular waves has limitations.

Irregularity takes place on at least two distinctly different time scales that are characterized by short-term and the long-term variations respectively. The easiest way to distinguish these two phenomena is to assume for the time being that during a particular storm the wave pattern is stationary. In other words, we neglect the gradual growth and decay of the wave field, and we consider the storm more or less as a block function. Even then, the wave motion is irregular as is demonstrated by the wave record shown in Figure 5.1.

5.2.3. SHORT-TERM STATISTICS IN DEEP WATER

Individual waves can be differentiated according to international standards by considering the water surface elevation between two subsequent upward or downward crossings of the Still Water Level (SWL) [3]. The time span between these crossings is the wave period (T), and the range between the highest and the lowest water level is the wave height (H). In this way, a height and a period can be defined for each individual wave i ($0 < i < n$) from the wave train. Since all heights and periods of individual waves are different, it is logical to apply statistical methods to characterize the set of data. The easiest way is to determine the statistical properties of the wave heights only.

It appears that in deep water, the probability of exceeding the wave heights follows a Rayleigh distribution:

$$P\left(\underline{H} > H\right) = \exp\left[-2\left(\frac{H}{H_s}^2\right)\right] \tag{5.2}$$

Wave periods are generally treated in a slightly different way. It is possible to consider the irregular surface level $\eta(t)$ to be the sum of a large number of periodic waves:

$$\eta(t) = \sum_i a_i \cos\left(2\pi f_i t + \phi_i\right) \tag{5.3}$$

in which:

a_i = amplitude of component i
f_i = $1/T_i$ = frequency of component i
ϕ_i = phase angle of component i

One can make a histogram of the wave height for all the selected frequencies. This is shown in Figure 5.3.

[3]Still Water Level is the water level in the absence of the waves, it is taken equal to the mean water level during the recording period

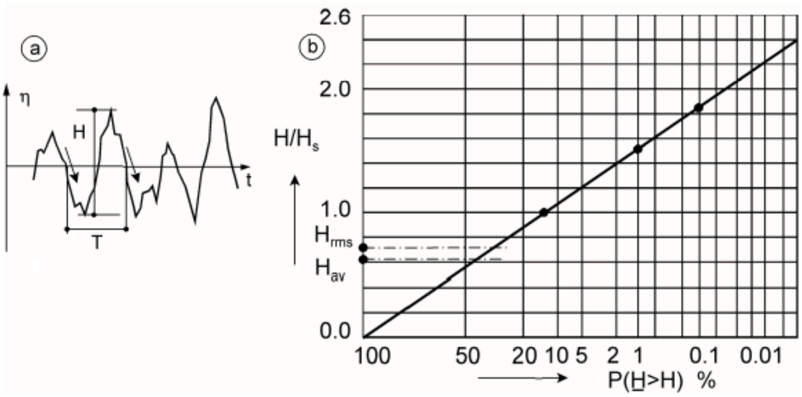


Figure 5.2: Rayleigh graph paper

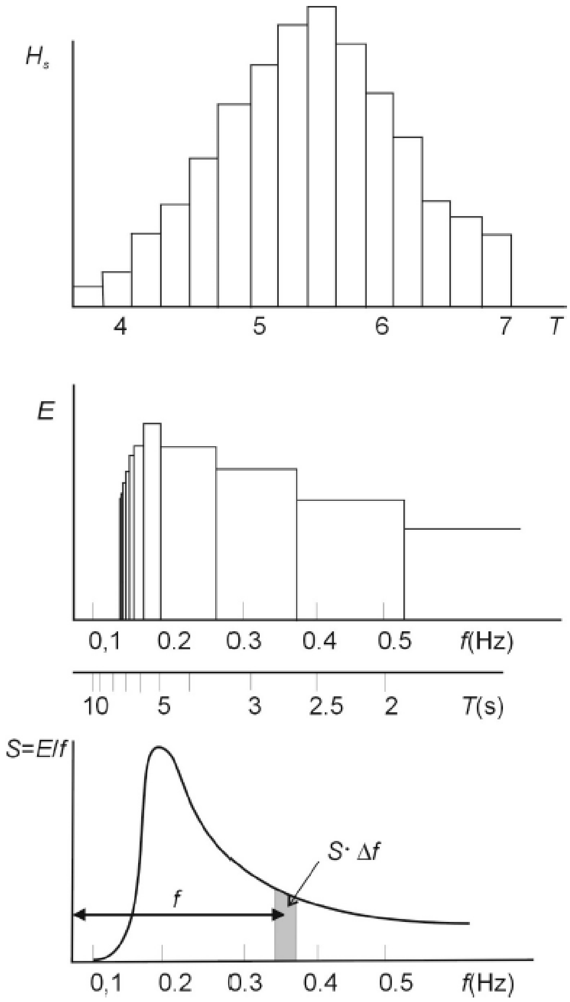


Figure 5.3: Construction of a wave energy spectrum

On the horizontal axis is the period, on the vertical axis the average Hi for that period. The next step is changing the horizontal axis. Now we plot the frequency instead of the period ($f = 1/T$). The result is that the vertical columns become quite narrow for low frequencies (long periods) and rather wide for high frequencies (short periods). Also the vertical axis is changed. Now the energy is plotted instead of the wave height. The energy is a function of the square of the wave height. In the last step we change from discrete classes to a continuous function. This implies that on the vertical axis we don't plot the energy, but the energy per Hz. The unit on the vertical axis is therefore “Energy”/Hz. [4]

In a more mathematical way, one can express this spectral energy density $S(\omega)$ as:

$$S(\omega) = \frac{1}{2} \sum_{\Delta\omega} a_i^2 / \Delta\omega \tag{5.4}$$

Integrating this energy density spectrum (integrating S, or E/f) over the whole interval, leads by definition to the total wave energy in the wave field:

$$\int_0^\infty S df = E_{tot} = m_0 \tag{5.5}$$

or:

$$m_0 = \overline{\eta^2} = \frac{1}{2} \sum_{i=1}^{n} a_i^2 \tag{5.6}$$

This integral is the total surface under the curve. Sometimes this is also called the zeroth moment of the spectrum (this explains the 0 in m_0).

In general one can define the moment of a spectrum with:

$$m_n = \int_0^\infty f^n S(f) df \tag{5.7}$$

The first order moment is always defined as (arm × surface area), second order moment is (arm^2 × surface area). See also the lower drawing in Figure 5-11. One can even define a (rather abstract) first order negative moment (arm^{-1} × surface).

The determination of $S(f)$ in practice is based on a more mathematical concept via the auto-correlation function $R(\tau)$ and its Fourier transform. In this process, some mathematical hiccups may occur. It is therefore recommended to check whether a direct analysis of the wave height distribution yields the same significant wave height as the spectral approach. In other words, check whether

$$H_s = 4\sqrt{m_0} = H_{13.3\%} \tag{5.8}$$

When the wave energy spectrum has been established in this way, in most cases it is possible to distinguish a frequency $f = 1/T$ or period T where the maximum energy is concentrated. This value is called the peak period T_p. Of course, one can also count the total number of waves (n) during the recording period and thus define an average period T_m. One can also define the period directly from the spectrum:

$$T_{m02} = \sqrt{\frac{m_0}{m_2}} \tag{5.9}$$

or:

$$T_{m-1,0} = \frac{m_{-1}}{m_0} \tag{5.10}$$

Usually T_{m02} is in the same order as T_p and T_m is in the same order as $T_{1/3}$. For deep water conditions, the peak period is 10% larger that the $T_{m-1,0}$. However, for shallow water conditions, this can be completely different. It will be explained later that for wave-structure interaction, the value of $T_{m-1,0}$ is better able to describe this interaction. The example of a typical (deep water) wave spectrum is given in Figure 5.4.

It is stressed that the spectral analysis and the Rayleigh distribution are only valid to analyse a stationary process. One must therefore be careful to choose a measuring period that is not so long that one is almost

[4]The wave energy is expressed as energy per unit of surface. The unit of energy is Nm. The unit of E should therefore be Nm/m^2. However, it is common in wave mechanics to divide by ρg, which implies that the unit for E becomes $Nm/m^2 \cdot m^3/kg \cdot s^2/m = m^2$. The unit of S is therefore m^2/Hz or $m^2 s$.

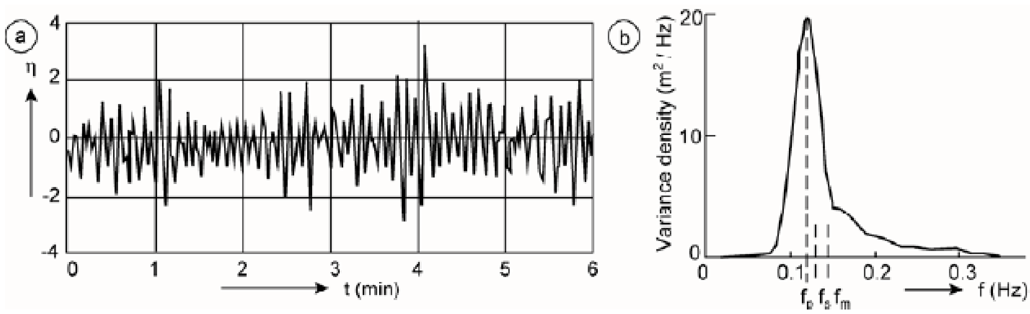


Figure 5.4: Typical wave spectrum, measured in the North Sea

sure that the wave climate will change during the observation period. On the other hand, one must choose a long enough observation period to ensure that the sample leads to statistically reliable results. It has become common practice to measure waves during a period of 20 to 30 minutes at an interval of 3 or 6 hours.

Summarizing, one can state that the short-term distribution of wave heights, i.e. the wave heights in a stationary sea state shows some very characteristic relations:

Name	Notation	$H/\sqrt{m_0}$	H/H_s
Standard deviation free surface	$\sigma_\eta = \sqrt{m_0}$	1	0.250
RMS height	H_{rms}	$2\sqrt{2}$	0.706
Mean Height	$\overline{H} = H_1$	$2\sqrt{\ln 2}$	0.588
Significant Height	$H_s = H_{1/3}$	4.005	1
Average of 1/10 highest waves	$H_{1/10}$	5.091	1.271
Average of 1/100 highest waves	$H_{1/100}$	6.672	1.666
Wave height exceeded by 2%	$H_{2\%}$	5.670	1.4

Table 5.1: Characteristic wave heights for Rayleigh distributed waves in deep water

In a similar way, periods can be related:

Name	Notation	Relation to spectral moment	T/T_p
Peak period	T_p	$1/f_p$	1
Mean period	T_m	$\sqrt{(m_0/m_2)}$	0.75 to 0.85
Significant period	T_s		0.9 to 0.95

Table 5.2: Characteristic wave periods

These relations are valid only for deep water, i.e. in the absence of breaking waves.

5.2.4. LONG-TERM STATISTICS

It has been shown that it makes no sense to determine the significant wave height and a spectrum if the wave train is not part of a stationary process. Therefore, waves are measured during a relatively short period (15 to 30 min.) at regular intervals of 3 to 6 hours. Each record is considered to be representative for the whole interval. In this way, a new time series is developed, consisting of the significant wave height (and period) per interval. This time series may cover a period from several months to several years.

The results of series of wave observations covering a longer period will therefore again become a set of random data that represent the long-term wave climate of the location. In this set, one may distinguish different patterns. It is possible that there is a real stationary condition throughout a year (trade winds), or that there may be distinct seasons (summer/winter, monsoon periods), while superimposed on these there may be incidents like the passage of a storm, a hurricane or a cyclone (Figure 5.5).

Depending on the purpose of the analysis, one must decide what to do with a long series of wave observations. In a number of cases, when one is interested in workability or in the accessibility of a port, one can

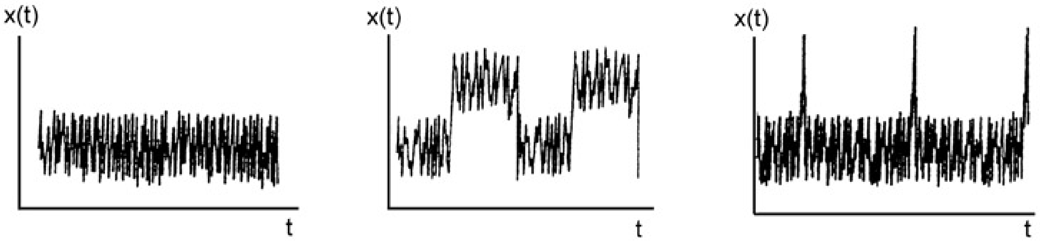


Figure 5.5: Typical types of wave statistics patterns

simply analyze all data and determine the so-called Serviceability Limit State or SLS. For such considerations, the incidents need not be important, provided that a timely warning is received and one can take some measures. The case is different when one is interested in the maximum loads exerted on a structure during its lifetime. Exceeding of the design load may cause serious or even irreparable damage to the structure. In that case, extreme value theories must be applied to determine the Ultimate Limit State (ULS). For such analyses, one uses only the highest observations from the set, as observed once per month or once per year. For some types of structure (rubble mounds) it is sufficient to characterize the load conditions by the intensity of the storm, i.e. by H_s. This simplification is justified because the damage of a rubble mound breakwater progresses slowly. When a caisson-type breakwater or another structure with a brittle failure behaviour is considered, failure may be caused by just one extremely high wave. This means that one has to establish an extreme value distribution for individual waves. This can be done by combining the long term and the short term wave statistics.

Whatever the case, it will be difficult to collect sufficient data from actual wave observations, simply because the period of wave observations is too short to establish a reliable prediction of extreme events. Thus it is necessary to use long-term wind records or visual observations of wave heights made on board ships to try to establish a long-term distribution of wave conditions. Actual measurements can then be used to calibrate the model that is used to determine the wave conditions indirectly.

Observation periods and storm duration

As mentioned above, for determination of the ULS, one has to apply extreme value theories. One method to do this is to take the highest observation value from a set of given duration (e.g. a month or a year). The disadvantage of taking the highest value in a month is that in such an analysis an unwanted seasonal effect may be introduced. Taking the highest value in a year has the disadvantage that the amount of data available becomes very restricted.

Therefore, an analysis is often made of all storms in the record. A storm is defined as a period with a more-or-less constant and relatively high wave height. In the PoT-analysis (Peak over Threshold) a storm is explicitly defined as a period of time during which the wave height is higher than a given threshold value; the magnitude is defined by the highest wave observation within the storm. This method is worked out as an example in Appendix 1.

Even when in this way a number of storms are identified, an extrapolation must be made to arrive at an assessment of the (rather rare) design storm. For this extrapolation, one may apply the Exponential, the Gumbel or the Weibull distribution. For details, one is referred again to Appendix 1.

The final result depends on the threshold value adopted and on the statistical distribution used in the extrapolation. In general, the most reliable results are obtained with relatively high threshold values. Even then, the result depends on the statistical method used. Table 5.3 shows the results of the calculation from Appendix 1, with a threshold value of 4.0m, which results in an average of 5.3 storms per year.

Statistical Method	$H_{ss\ 1/225}$
Exponential	8.31 m
Gumbel	7.81 m
Weibull	7.77 m

Table 5.3: Example of predicted design wave heights

The difference between the methods is in the order of half a meter. N_s is the number of storms per year

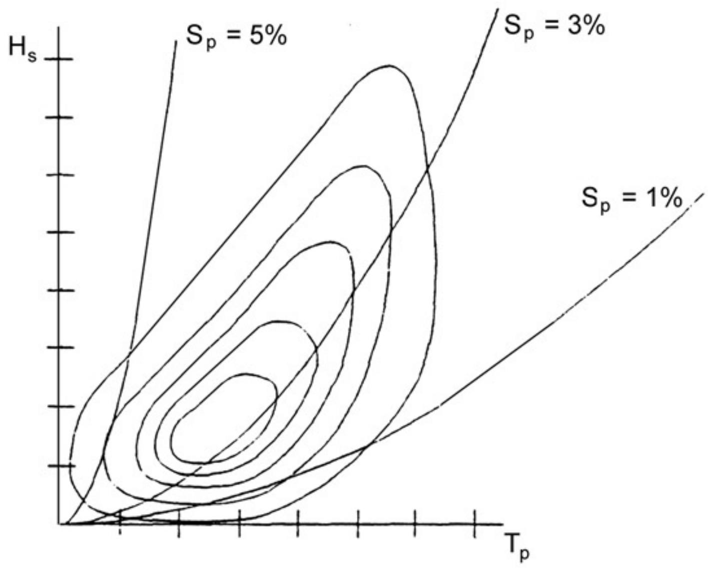


Figure 5.6: H/T diagram

in the database. Lowering the threshold level will increase the number of storms in the analysis, but at the same time make the analysis less reliable. The example in Appendix I is based on a good quality and extensive database of wave observations. Even in this case the difference between various methods is large. This shows that it is ridiculous to try to determine design waves with accuracies of centimetres.

RELATION BETWEEN WAVE HEIGHT AND WAVE PERIOD

When analysing series of wave data it is always good to make a number of checks. One of those checks is to study the relation between H_s and T_p, as calculated for each wave record. Plotting the data in a H/T diagram may lead to a result such as that shown in Figure 5.6.

One must realize that substituting T_p in the deep-water wavelength formula leads to a deep-water wavelength L_{0p}. The fictitious wave steepness can then be expressed as $s_{0p} = H_s/1.56T_p^2$. Values of sop are rarely higher than 5 or 5.5%. These high values are representative for waves that are generated by one typical nearby wind field. Low values of sop indicate swell from remote wind fields. The low values are associated with low wave heights and very long periods. Values below 1% are rarely reported, maybe partly because many measuring instruments do not measure this type of wave accurately.

5.2.5. TRANSFORMATION OF IRREGULAR WAVES IN SHALLOW WATER

In the small amplitude approach, it is possible to distinguish individual waves with a single height and a single period, and to study changes in their direction and height on the basis of the ratio between water-depth and wavelength .

For irregular wave fields, the analysis of changes in the wave pattern is much more complicated when the waves enter into shallower water. Because the wave field consists of many periods, no single set of wave orthogonals can be calculated.

Modern wave models, such as SWAN (see `www.swan.tudelft.nl`), are able to calculate wave growth and wave changes in near shore conditions. These models give the local wave spectrum (directional spectrum) at any desired location in the computational area and are therefore classified as spectral models. The method is based on considerations of energy flux By adding criteria for wave breaking, the models replace separate calculations for shoaling, refraction and breaking. For complicated bathymetries and detailed studies, a two-dimensional computation is needed. For preliminary studies and simple cases a one-dimensional computation is often sufficient; for example, with SwanOne (see `www.kennisbank-waterbouw.nl` and go to software).

Because of the variability in wave heights, one must expect that some waves will break due to their extreme height, whereas other (lower) waves remain unaffected. This means that the Rayleigh distribution cannot be valid in shallow water.

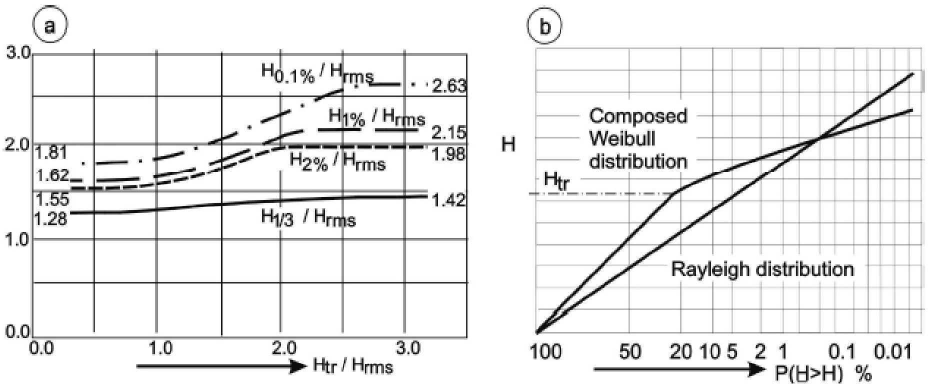


Figure 5.7: Wave height distribution in shallow water (Battjes-Groenendijk,2000)

For the wave height distribution, GROENENDIJK [1998] made a so-called Composed Weibull-distribution. His work is based on the BATTJES AND JANSSEN method [1978] and is also summarised in BATTJES AND GROENENDIJK [2000]. Below the transitional value of the wave height (H_{tr}), the Rayleigh distribution remains valid. $H_{tr} = (0.35 + 5.8 \tan\alpha)\, h$, in which α is the local bed slope and h the local water depth. Above this value the exponent in the distribution function has a different value (≈ 3.6):

$$P(\underline{H} \leq H) = \begin{cases} F_1(H) = 1 - \exp\left[-\left(\frac{H}{H_1}\right)^2\right] & \text{if } H \leq H_{tr} \\ F_2(H) = 1 - \exp\left[-\left(\frac{H}{H_2}\right)^{3/6}\right] & \text{if } H > H_{tr} \end{cases} \tag{5.11}$$

H_{tr} at a certain water depth is found from the spectral area, m_0 (known from computations like SWAN), the foreshore slope angle and the waterdepth. H_{rms} is also a function of m_0 and the water depth. Figure 5.7 shows various wave heights as a function of H_{tr}, all made dimensionless with H_{rms}.

Figure 5.7 shows that the ratios between various wave heights and the H_{rms} change when the waves enter shallow water. For very shallow water, $H_{1\%} \approx 1.6 H_{rms}$ instead of $2.15 H_{rms}$ as would follow from Equation (4.13). These corrections for the Rayleigh distribution can be used in equations for wave run-up and overtopping, but especially for the determination of armour stability (see chapter 7).

As a rule-of-thumb one may state that the maximum value of H_s is approximately $0.55h$. This means that the limited water depth very effectively protects structures in shallow water. It is emphasized, however, that steeper slopes of the foreshore will certainly reduce this protection. It is also emphasized that the construction of a breakwater may lead to erosion, and consequently greater water depths and more severe wave attack. Finally, attention is drawn to the fact that in many locations the occurrence of a storm or hurricane causes not only high waves, but also a storm set-up. Since these two phenomena are not independent stochastic events, their joint occurrence must be taken into account. Owing to the higher water levels resulting from the storm surge, water depths will also be greater and higher waves can therefore penetrate to the location of the structure.

A short remark must also be made on changes that occur to the shape of the wave spectrum after breaking. Where the spectrum in deep water will show one or two distinct peaks, these peaks disappear largely in shallow water. This may have serious consequences for the behaviour of coastal structures. This is one of the reasons why in shallow water conditions the period $T_{m-1,0}$ is preferred to T_{m02}.

In case of two wave fields one may calculate the (directional) wave spectrum at the toe of the structure with a full 2D Swan calculation. However, in case the interaction between the wave fields is limited, one may make two SwanOne calculations resulting in two wave spectra at the toe of the structure. One may add the values of m_0, m_{-1} and m_2 of the two spectra an in this way determine the load on the structure.

The example above shows also a danger of neglecting the influence of swell waves in the overall wave climate. Often data are available is a format like is shown in Figure 4-14. Following the method of Appendix 1 leads to a design storm with a given H_s and T_p, for example the 3 m, 8s condition as presented in the above example. However, there could be simultaneously a significant swell. Not including this swell will lead to a considerably lower (and incorrect) design wave at the toe of the structure.

Example 5.1:

At a certain location we have at deep water a swell (H_s=1m, T_p=20sec, approaching the coast from -30 degrees) and sea (H_s=3m, T_p=8sec, approaching the coast from 30 degrees). The toe of the breakwater is at a depth of 5 m. This results in the following values:

at -5 m	H_s	$T_{m-1,0}$	m_0	m_{-1}	m_2
sea waves	1.84	6.45	0.21155	7.733 E-3	1.3649
swell waves	1.16	16.39	0.08358	0.472 E-3	1.3699
total			0.29513	8.205 E-3	2.7348

One can now calculate the local combined wave height $H_s = 4\sqrt{(m_0)} = 4\sqrt{(0.295)}$ =2.17 m and a period $T_{m-1,0} = m_{-1}/m_0$ = 8.2E-3/0.295 = 9.27s. Using Battjes-Groenendijk gives a $H_{2\%}$ = 3.04m at the toe of the structure. It is obvious that this gives a different result than simply adding the two local wave heights and period.

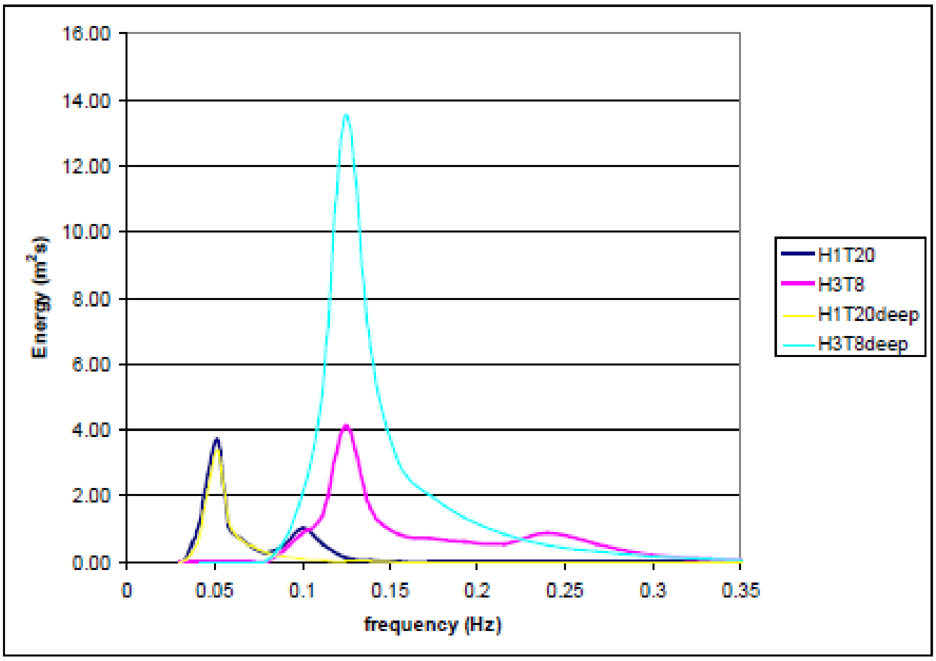


Figure 5.8: The deep water and the shallow water spectrum from example 5.1

5.2.6. SUMMARY DETERMINATION SHALLOW WATER WAVE CONDITIONS

So, in summary, one should determine the shallow water boundary conditions as follows:

- Determine the deep water wave conditions (e.g. with one of the methods described in Section 0). This means a wave height, a wave period and a spectrum shape is determined. Typically, one determines a significant wave height and a mean period and a Jonswap spectrum shape is assumed.
- Calculate the shallow water wave condition with a spectral model (e.g. with SWAN, either in 2D or in 1D option). This gives the local H_{m0}, T_{m02} and $T_{m-1,0}$.
- Use the Battjes-Groenendijk method to determine the $H_{2\%}$.

5.3. GEOTECHNICS

5.3.1. GEOTECHNICAL DATA

The construction of breakwaters takes place along shores and river mouths with a great variety of subsoil conditions. Bedrock underlies these layers at various depths. Sometimes local deposits, as boulder clay or cobbles, may be encountered. In several deltaic regions, layers of peat may also be present. These soil layers form the foundation bed for the structure and need to withstand groundwater flow, wave loads and differential water pressures. During the construction of breakwaters, they are the sub-base for vehicles and equipment driving across the site. When submerged, they will be exposed to eroding currents and waves. Therefore, a thorough knowledge of the soil types present, including their characteristics and the stratification, is a prerequisite for the design and execution of breakwater construction and closure operation.

Geotechnical data can be obtained from sample-borings analysed in laboratories and penetrometer-tests executed in the field. The number of borings is usually limited in relation to the area considered and to obtain a reasonable picture of the sub-bottom, an overall examination, together with historical and geological information, is required. Particularly in deltaic areas and tidal inlets, former gullies filled up with different types of soil may give sudden changes in the sub-bottom profiles of soil layers. Very localized deviations from the general picture as well as obstacles (fossil trees, large boulders) seldom appear in the results of a field survey. The most important parameters of the sub-bottom are stratification, soil type and phreatic levels. Laboratory tests on samples of every layer of clayey soil will give the values for cohesion, angle of internal friction, Atterberg limits and water content. Granular soil types are characterized by the grading of the grain sizes, the sharpness (roundness) of the grains and the pore volume (relative density).

Cohesion and friction-angle are the most important parameters for soft soils. The field tests are done by using Standard Penetration Tests (SPT), giving the number of blows needed to hammer a pin down into the ground a predetermined depth or by using the Dutch Cone, which gives the force required to push a cone down into the soil. These values are sometimes translated via relation-tables into values for cohesion and friction-angle. Triaxial testing of soil samples is far more accurate. Triaxial tests can be drained or undrained and consolidated or unconsolidated, which leads to different values for the same soil. Which of these is the most appropriate depends on the purpose.

For granular material (sand), the relative density and the permeability are the most important parameters. The grain structure itself is strong enough to withstand considerable surcharges. Problems may arise if some grains want to rearrange to form slightly more dense packing and the pore water cannot escape. The latter is the case if the permeability is low. This applies to sand with a high content of fines. Generally, the 10% finest part of the sieve curve determines the permeability.

Apart from the subsoil, the materials used for the dam construction form part of the total soil mass to be considered. Sand-fill, rock masses and clay cores are a surcharge on the one hand and on the other hand are subject to instability. Stability criteria have to be determined not only for the final design but also for the various construction stages. Surcharging compressible soils with low permeability will result in an increase in the water-stress which fades away slowly and thus the rate of loading may be a determining factor. Therefore, the planning of the construction is important and to ensure stability during construction, considerable waiting periods may be required between subsequent construction phases. If the waiting times are too long, it may become necessary to apply costly remedial techniques to avoid mishaps.

5.3.2. GEOTECHNICAL STABILITY

The most important property of the soil is its bearing capacity, which frequently is a determining condition for the design and a limiting factor for the operations. The soil mechanics problems, related to bearing capacity, are:

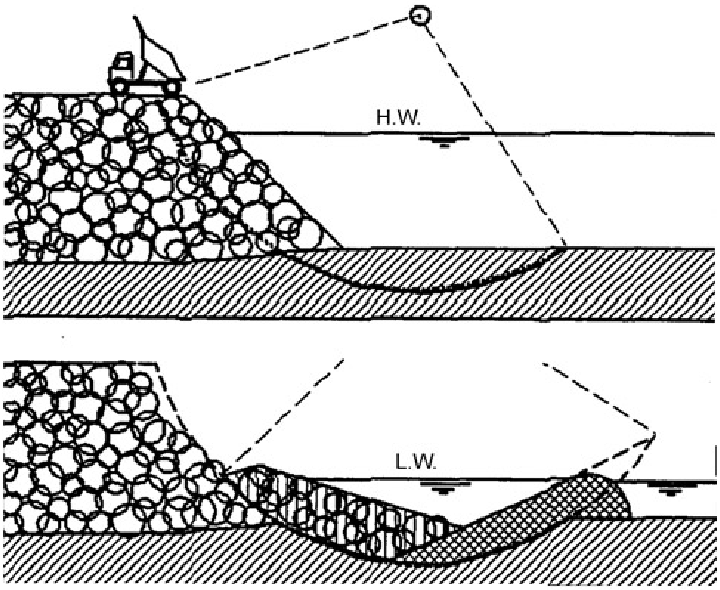


Figure 5.9: Dam profile after slide

- Sliding
- Squeeze
- Liquefaction

SLIDING:

In dam and breakwater constructions, sliding is the instability of an earthen or quarry stone embankment along the side slope or at the construction front. The driving force of the slide is the surcharge on the subsoil by the embankment's own-weight. The steepness of the slope is an important parameter. In any dam construction two aspects have to be considered in particular. One is the water level variation at the dam site, the other is the erosion of the soil in front of the dam.

Water level variations influence the amount of surcharge. A body made of quarry run will have a specific weight above water of about 17 kN/m^3, however, when submerged its effective weight will be only 10 kN/m^3. [5].

As a part of the dam may be submerged during high water, the weight gradually increases during falling water levels due to loss of buoyancy. Such dams, constructed by dump truck tipping, have very steep natural slopes (gradients up to 1V:3H). Although extended without problems during the high water period, the dam may suddenly fail during low water.

A dam made of sand and constructed by hydraulic filling results in far more gentle slopes. Below the waterline, this will be in the order of 1V:5H to 1V:15H (depending on the grain size of the sand) and even flatter above water. The fill is fully saturated material, which has a weight of about 20 kN/m^3 and 10 kN/m^3 when submerged. In spite of the greater unit weight, the flatter slopes tend to provide extra resistance against sliding.

The possible erosive action alongside and in front of the embankment resulting from the high flow velocities of the currents around the dam head are more difficult to predict. Erosion pits may develop rather quickly and take away part of the soil that is assumed to provide the counterweight which is necessary to keep the dam stable. These erosion pits seldom appear on the design drawings. Moreover, it is difficult to predict their shape, size and depth. However, a designer has to include these conditions in their considerations that govern a safe operational procedure. Preventive measures, such as placing a protective layer of quarry run ahead of the progressing works, will avoid the erosion. For a permanent structure like a breakwater, more attention must be paid to the design of the protective layer, probably by including a granular filter or geotextile.

In practice, sliding should be prevented or used intentionally. Preventive measures must cover all circumstances during the construction of the initial closure dam profile. During a later stage of construction,

[5] 1 m^3 of rock with a porosity of 65% weighs 17 kN, 1 m^3 of submerged rock weighs $0.65 \cdot 1600 \cdot 9.8 = 10$ kN

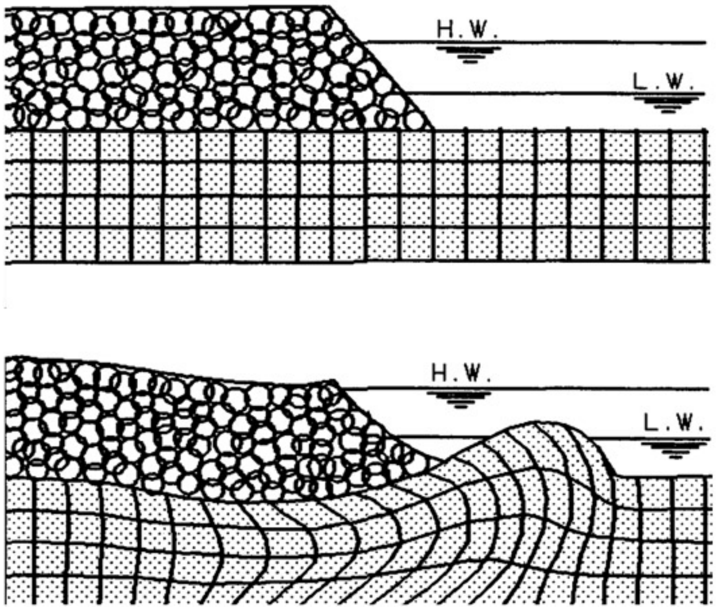


Figure 5.10: Squeeze

the dam profile will be enlarged by further heightening, widening and finishing of the slopes. In many cases, with the knowledge that the soil will gain strength after some time, the construction of the final profile may be scheduled in such a way that stability is not critical. The calculations for the initial and final profiles require different cohesion values.

Sliding is used intentionally if a soft layer of limited thickness is situated on top of a good bearing subsoil. There are two design choices; either to remove the soft layer by dredging (and backfill the created trench with good sand) or to construct on top of the soft soil and press the embankment down into it by using its own weight, which involves sliding. The first choice is the safest and should always be used for critical parts and operations of the closure dam construction. Nevertheless, the second method is used sometimes, although there is a risk that the soft soil will be only partly pushed away and that in later stages deformation will continue. For such a procedure the safety factor should be much lower than 1, as the more difficult to achieve than ascertained stability.

Squeeze

It is clear that in a situation in which very weak subsoil is surcharged by very stable dam material, instability may be restricted to the subsoil. Instead of sliding along a plane, deformation of the soil layer occurs that is comparable to ice-cream squeezing out between two wafers. Over-stressing will then start at one location and locally lead to deformation without changing volume. The stress transfers to the surrounding area in various directions so the deformation expands. The dam body on top sinks into the subsoil and the same volume of weakened soil escapes on the edge of the dam. This type of instability is called squeeze. As this failure starts by over-stressing in one point and then progressively expands via deformation, the mathematical approach is different from that used for the slide along a plane in basically undeformed bodies. For the calculation of instability by squeeze, a mathematical model (e.g. Plaxis) has to be used.

The Figures 5.9 and 5.10 show that sliding and squeeze are completely different. This is more difficult to establish by observation in the field. The profile after instability is seldom as ideal as the one sketched and in both cases a heap of subsoil material emerges in front (or at the side) of the breakwater.

Taking samples at the toe of the slope will show whether sliding or squeeze has occurred. In practice, the difference between sliding and squeeze has few implications. A choice has to be made in the design between soil improvement and deliberately induced subsidence with associated waiting times.

Liquefaction

Liquefaction may occur in loosely packed sands. This is aggravated by low permeability of the soil. The strength of a sand mass is determined by the transfer of the forces from grain to grain. In saturated sand, the

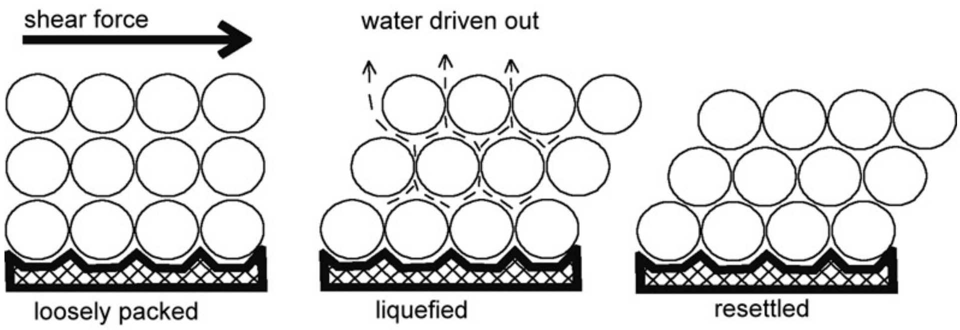


Figure 5.11: Liquefied sand

voids between the grains are filled with water.

The water pressure is hydrostatic and does not bear or support the grains. If, for some reason, a shear force leads to movement of the grains, they will try to obtain a denser packing, which is possible only when some of the water is driven out of the pores. At that moment, the water is taking part of the load from the grains. If the permeability of the sand is low, for instance, because of a high content of fines; the water cannot escape and the grains lose their stabilizing contact forces. A fluid mass of water and sand grains, with a density of 1800 kN/m^3 results and it behaves like a heavy liquid (quicksand).

From the above, it is clear that liquefaction may occur in loosely packed sand of poor permeability only after an initial event has triggered a disturbance of the grain structure. In nature, such deposits of sand occur in areas where the deposition environment led to loose packing of the grains; there having been no turbulence, no waves and not too much current flow. Artificially made bodies of sand, placed under water, for instance by hydraulic filling, always have a loosely packed grain structure. Additionally, the quantity of fines determines sensitivity to liquefaction. The initiating event may vary in nature. If the grains are in a stress situation, for instance because of a developing scour hole, a little vibration or shock may start the flow. This applies also to wave induced varying loads transferred into the soil by a rigid structure like a pier or a caisson. Unlike slip-circle and squeeze, liquefaction will result in a flow slide with very gentle slopes at the toe.

In practice, many stability failures have occurred in closure operations and during breakwater construction. Usually they resulted from variations in soil characteristics, changes in the works program, limited soil information and unforeseen conditions. Apparently, soil conditions on the sites where these works were executed, gave little margin for error. Of course, it is important to know what should or should not be done in such cases. After a failure, the soil is disturbed, the soil structure is distorted, excess pore pressure is still present and the new situation has a very vulnerable equilibrium state. Shifted material will be in the wrong place, for instance at the toe of the slope, and the top layer of the dam will be too low. However, every corrective action to take away the wrong toe material or extend the dam by recharging the top will lead to continuation of the failure. The only possible measure to improve the situation is to let the soil mass consolidate and let the stresses in the water diminish. This takes time and can only be slightly accelerated. If it can be installed, artificial drainage by vertical drains may be helpful. Otherwise, it is better to bypass the area and thereby provide time for the water pressure to reduce to normal. Afterwards, preferably working from the toe side uphill and very gently layer by layer, the profile of the dam can be restored. It must again be emphasized that it is of the utmost importance to avoid failure of the foundation by adequate design and by adequate analysis of all construction phases.

SETTLEMENT

The problem of settlement is mainly an issue for the determination of the final design profile rather than for the various construction phases. If the long period of settlement after completion of the works is unacceptable for operational reasons, one may decide to enhance consolidation by applying artificial drains combined with a temporary surcharge.

In practice, the expected settlement is often added to the design height of the structure. Consequently, the surcharge onto the subsoil increases initially with the extra height of dam material and may put the profile at risk of failure. The over-dimension is not necessary in the first instance and in a critical situation, may be postponed until the subsoil has gained strength by some degree of consolidation. Then, after some time, the top layer of the dam has to be completely reconstructed. Alternatively, the settlement-sensitive soil layer can be dredged away and be replaced with better soil (sand).

6

Data collection

Chapter 6 summarizes data that must be collected before a large hydraulic project can be designed or constructed. It gives readily available sources of data and it discusses some methods that can be used for the collection of the relevant data. This chapter is meant to illustrate to students the preliminary work that has to be done in the early stages of project preparation.

6.1. General

In Chapter 5, we recalled some of the theory that we need for designing structures like breakwaters and closure dams. In practice, however, it is not only important to master the theory, it is also equally important to assess the physical presence of certain phenomena that are known from theory. For each subject, this chapter pays attention to the availability and, if necessary, the collection of relevant data on the prevailing hydraulic, geotechnical and other conditions.

For design purposes, we are sometimes interested in extreme events, specifically, when we try to establish the loading conditions for the Ultimate Limit State. In many cases, it is impossible to use direct observations since our records are too short to make a sensible assessment of extreme events. In such cases, we have to rely upon an indirect approach, in which we use data that have been recorded over a sufficiently long period. In this respect, specific reference should be made to meteorological data (wind direction, wind speed, barometric pressure, temperature, rainfall, visibility, humidity) that have been collected (even in remote areas) over a much longer time-span than those of wave heights, tidal currents and river discharges. Meteorological stations, airports, hospitals and even missionary outposts may collect a surprising wealth of data. Calculation and calibration of such data can then be transformed into the required data.

6.2. Meteorological data

Although meteorological phenomena generally do not play a direct role in the design of hydraulic structures, they are indirectly important. Barometric pressure and wind data act as generators of surges and waves. In a similar way, precipitation plays a major role in the generation of river discharges. Wind plays a direct role when one considers forces on ships and structures or the effects of spray from breaking waves.

Other factors may be important as well, though their roles are less obvious. In this context, consider visibility, which is important because any marine operation is seriously hampered by fog. Temperature and humidity are important to equipment (cooling, corrosion), but also for the hardening of concrete, and the formation of cracks during the hardening process. Freezing conditions and the presence of ice must also be taken into account where applicable.

Meteorological data are generally available from national meteorological offices. Where more localized data are required, measuring instruments are easily available in the market.

6.3. Hydrographic data

6.3.1. Bathymetry

Before starting a design job, it is essential to have proper maps of the seabed or riverbed. In most cases, hydrographic charts are useful but their scale is such that the information is not detailed enough for engineering purposes. The same applies to standard river navigation charts. Another problem is that the datum of those charts is often related to tidal levels (MLWS or similar) and may not be the same for the entire chart. This can pose serious problems when quantities of dredge or fill materials have to be determined on the basis of such charts. The advantage, however, is that for most regions in the world, such reliable charts have been available for at least 200 years. The hydrographic departments in many countries preserve these old charts. They can provide very valuable information about long-term morphological developments in the region.

At this moment online hydrographic charts are free available (e.g. `http://navionics.com`). For more recent morphological changes also Google Earth can proved fast information, using their "history" option.

In practice, this means that most projects require that site specific maps should be made. Nowadays a hydrographic survey poses few problems. One needs a survey launch equipped with an echo sounder, GPS positioning, a reliable radio tide gauge and a data logger. Often these four elements are built in into one single device. When the sea is rough and the survey launch is subject to considerable movements due to waves, one may add a heave compensator. With the aid of modern software and without much human effort, a plotter input can draw the maps. Because the generated data sets are available in digital form, it is also possible to use them for many kinds of arithmetical exercises, such as calculating volumes to be dredged or filled. The data can also be used to assess erosion or accretion between subsequent surveys.

It is wise to pay attention to the type of echo sounder used. When a high frequency (210 kHz) is used for the measuring beam, the depth indicated is at the top of a soft soil layer. When lower frequencies (30 kHz) are used, the beam penetrates into the soft mud layers to harder layers beneath. By using a dual frequency instrument, one can obtain an estimation of the thickness of such layers of soft mud.

6.3.2. Tides

Vertical tides

The tidal constants for most important harbours in the world are known. They are published annually either by a national hydrographic office or by the British Admiralty. Via the Internet many tidal predictions can be obtained. The British and the French hydrographic offices have online tide predictors for most ports in the world (`http://easytide.ukho.gov.uk` and `http://www.shom.fr`). The disadvantage of these sites is that the prediction period is rather short. An international operating site is `http://tbone.biol.sc.edu/tide/`. For the Netherlands one is referred to `http://www.rijkswaterstaat.nl/water`. For minor ports, one has to rely on national or local authorities, and the reliability of data provided may not always be as good as required. Setting up a local observation point and performing hourly observations of the water level during a period of one month can yield a provisional insight. The application of harmonic analysis techniques easily leads to a reasonable estimate of the most important tidal constants. Only when one is interested in the long periodic components is a longer observation period required.

For dedicated purposes (for instance to obtain boundary conditions for a mathematical model) one can nowadays obtain reliable data by remote sensing techniques.

Horizontal tides

Tidal currents are sometimes indicated on hydrographic maps, the standard of accuracy of which is usually insufficient for design or planning purposes. Some hydrographic departments issue flow atlases with more comprehensive information. Usually it is necessary to make dedicated flow measurements, which are time consuming since they have to be continued for at least 13 hours. It is therefore advisable to analyse flow phenomena by using a mathematical model and to use field measurements mainly to calibrate the model. The above mentioned website `http://tbone.biol.sc.edu/tide` provides also some data on horizontal tides.

6.3.3. Storm surges

Tidal water level variations can be predicted accurately on the basis of astronomical facts. In addition to the tidal variations, there may be meteorological effects that influence the water levels. Since meteorological effects cannot be predicted long in advance, they must be taken into account on a statistical basis. If no direct observations are available, one may use records of wind velocities, barometric pressures and hurricane or cyclone paths to estimate the probability of extreme water levels.

6.3.4. Waves

There are few places in the world where long series of wave observations are available. This is simply because reliable instruments for wave recording did not exist until approximately 1970. The oldest observations of waves were carried out onboard ships that had a voluntary agreement with a meteorological office to carry out certain observations. Although the recordings of waves were visual observations, their accuracy is acceptable since the officers were well trained and could compare the observed sea state with standard pictures provided by the met-office. These observations were collected and sorted according to locations spread over the oceans. A large collection of similar data has been assembled and edited by YOUNG AND HOLLAND [1978] and by HOGBEN, DACUNHA AND OLIVER [1986]). A disadvantage of these data sets is that the oceans have been divided into relatively large areas, so that detailed information close to the shore is still not readily available. More detailed information can be obtained via commercial wave data banks, like Argoss (`http://www.waveclimate.com`). Local public agencies often publish their data on the internet, like `http://www.ndbc.noaa.gov/` and for the Netherlands `http://live.waterbase.nl`.

Direct measurement of wave heights in the preparation phase of a project will never provide the required long-term data. However, it is still useful to have such direct observations, if only to calibrate the calculation methods used to transform indirect observations into local wave data. Modern methods for wave measurement are

- electric (resistance or capacitor type) wave gauges, mounted on a platform or a pile
- acceleration type gauges, mounted in a floating buoy
- pressure gauges, mounted on the seabed
- inverted echosounder, measuring the distance from sea bed to water surface
- remote sensing techniques (from satellite)

One problem encountered is that one wants to measure wave heights in relatively deep water, so that shoaling or breaking does not yet affect the measured heights. This makes all pile or platform mounted gauges relatively expensive, unless use can be made of an existing facility. The use of pressure gauges is not recommended because the actual wave pressure at seafloor is a function not only of the wave height, but also of the ratio between wavelength and water depth. One of the most popular deep-water wave recording instruments is the Waverider Buoy, from Datawell, Haarlem, Netherlands. This device measures the vertical acceleration of the water surface, and transforms this by double integration into a vertical motion. This makes the observation of very long swell ($T > 20$ s) difficult and not fully reliable. Pitch and roll are also measured, which makes measurement of wave direction possible. These buoys are always delivered with complementary software to analysis the collected data. Standard output of these buoys is a directional wave spectrum each selected interval (e.g. every half hour). Because accelerometers are nowadays a standard part of any smartphone, one can also use a smartphone in a casing as a wave measurement device, like the Wavedroid (`www.wavedroid.net`).

Because it is a spot measurement, the buoy gives the wave condition for this point only. With remote sensing techniques (altimeter and other platforms) one may get directional wave spectra, but this spectrum is an average of the footprint of the radar or lidar. Both systems have advantages and disadvantages. An example of the transformation of measured wave data or of wave data from a general database into exceedance curves which can be used to determine design wave heights, is given in Appendix 1.

6.4. GEOTECHNICAL DATA

The geotechnical data required for a breakwater certainly includes all data required to assess the bearing capacity of the subsoil, both during construction and after. Stability of the works must be ascertained during all phases. Furthermore, one wants to predict settlement as a function of time in order to ensure that the required crest level is fulfilled at all times. In many cases, the works will be accompanied by substantial dredging. Therefore, soil properties for this purpose must also be known. If erosion or scour is expected to occur, it is necessary to establish the resistance of the existing seabed to this threat. Table 6.1, after the Rock Manual (CIRIA/CUR/CETMEF, 2007]), gives a good impression of the geotechnical failure mechanisms and their relation with basic geotechnical data.

This means that for a major project a general geological analysis must always be carried out to determine the geophysical and hydro-geological conditions. The most important aspects that require attention are:

- Geological stratification and history
- Groundwater regime
- Risk of seismic activities

Basic data can be obtained from the national geological services and, in more general terms, from scientific libraries and universities. Most of the available information will refer to land and rarely to estuaries and

Figure 6.1: Wavedroid

sea. Such basic geological data will provide general insight into what can be expected in the area of interest. Usually this is insufficient for engineering purposes, so soil investigations are always necessary. These investigations may include the following methods:

- Penetration tests (CPT or SPT) to establish in-situ soil properties
- Borings to take samples from various depths for further analysis in the laboratory
- Geophysical observations

These specific investigations are expensive and difficult because they must be done at sea, under the direct influence of tides, waves and currents. This makes any penetration test or boring a time consuming and risky affair. Therefore there is a tendency to limit the number of such local tests. This imposes the risk that discontinuities that occur in between the measuring locations will not be recognized. This is the reason why local observations should be combined with a geophysical survey. The geophysical survey uses electro-resistivity and electro-magnetic and seismic techniques to obtain a continuous image of the soil conditions in the tracks sailed by a survey vessel. The disadvantage is that there is usually no direct link between measured data and the geotechnical soil properties. Combination of geophysical survey and point measurements eliminates the disadvantages of both. The geophysical data ensure that no discontinuities are overlooked, while the borehole and penetration tests provide the link with the actual engineering properties of the soil.

Table 6.2 gives a complete view of the available in-situ test methods and their applicability

6.5. Construction materials, equipment, labour

6.5.1. Construction materials

The most important construction materials for breakwaters are quarry stone and concrete.

Quarry stone

Quarry stone is natural rock obtained from quarries. There are three or four basic types of quarries:

- Commercial quarries for the production of ornamental stone (e.g. marble tiles, kitchen blades, etc.); so-called dimension stone quarries.
- Commercial quarries for the production of fine aggregates for concrete, road construction, etc.
- Commercial quarries for the production of large sized rock for hydraulic engineering
- Dedicated quarries for the same purpose

In first instance it seems that dimension stone quarries are not relevant to our purpose. However, a large part of the production of these quarries does not fulfil the aesthetic requirements. For the dimension quarry this is “waste material”, but these stones are excellent for armour (usually the stones are quite large and

Macro instability			Macro-failure	Macro instability	Geotechnical information	
Slip failure	Lique-faction	Dynamic failure	Settle-ments	Filter erosion	Name	Symbol
A	A	A	A	A	Soil profile	-
A	A	A	A	A	Classification/grain size	D
A	A	A	B	A	Piezometric pressure	p
B	B	B	A	A	Permeability	k
A	B	B	A	B	Dry/wet density	$\rho_d, \rho_{sat}, \rho_{sub}, \gamma = \rho g$
-	A	B	-	-	Relative density, porosity	n , n_{cr}
A	B	B	-	C	Drained shear strength	c, ϕ
A	-	-	-	C	Undrained shear strength	s_u
B	-	-	A	-	Compressibility	C_c, C_s
A	-	-	A	-	Consolidation coefficient	c_v
B	B	A	A	-	Moduli of elasticity	G, E
B	A	A	A	-	*In situ* stress	σ
-	A	B	A	-	Stress history	OCR
B	A	A	B	-	Stress/strain curve	G, E

Table 6.1: Required soil data for the evaluation of the geotechnical limit states; A-B-C is ranking of applicability (after Rock Manual, 2007)

blocky). Some companies are specialised in trading dimension stone for use as armourstone. The quarries for the production of aggregates projects, although the fine material that they produce can be used as filter material.

In some parts of the world, where a regular demand for larger sized stone exists, a few quarries have specialized in this field. Such quarries exist for instance in Belgium, Germany, Norway, Sweden and Scotland. The relevant properties of the stone are widely known and listed in catalogues.

The situation is different if a large project is to be executed in an area where no such quarries exist. In that case, a rock formation has to be found that can be used to open a quarry that is specifically dedicated to the project. The following data should always be obtained:

- Specific weight and density of the material
- Durability in air and in water (fresh and saline)
- Resistance to abrasion
- Strength (tensile and compressive)
- Maximum size that can be obtained and distribution (yield) curve

In general, these data are so important that it is worthwhile to employ geological specialists to find a suitable location for a quarry. It is even recommended that one or more test blasts should be carried out before a final decision is taken to open a quarry. Apart from the technical data on the rock, it is necessary to be sure that the quarrying operation is acceptable from social, environmental and legal points of view. Since quarry stone and quarries are quite essential for any major hydraulic engineering project, more details are provided in Appendix 2.

In Europe EU-directives require that products are labelled with standardised overview of the different properties. Figure 6.1 shows a sample label. Of course the main property is the grading, but also other properties like resistance to breakage should be mentioned. For each parameter a value or a code is given. The code (for example A or B) means that the product complies with certain criteria without exactly defining the criteria. For example, the length-to-thickness ratio is called LT. The label may give LT_A, LT_{NR} or LT_{xx} in which xx is a number, for example 8. When the label LT_{NR} is given, this means that the manufactures gives no guarantee at all for the length-to thickness ratio. For some applications (for example for breakwater core material) this is not a problem, but normally this is not allowed. For most applications the length-to-thickness ratio should be smaller than a certain value. This value depends on the size of the stone. For small stones (called "course gradings" in EN 13383) less than 20% of the stones should have a length-to-thickness ratio of more than 3. However, for a "Heavy grading", less than 5% should have a length-to-thickness ratio of more than 3. When a certain stone delivery fulfils this requirement, the label mentions LT_A. For the armour layer the designer should always prescribe that stones should have LT_A. Some producers have stones which

	Geophysicical methods				Penetration methods						Borings		
	Seismic	Electr. resist.	Electro-mag-netic	nuclear	Cone penetr. test (CPT)	Piezo conc. test (CPTU)	Stand. penetr. test (SPT)	Field vane test (VST)	Press. meter test (PMT)	Dilato meter test (DMT)	Dist. samples	Undist. samples + Lab. tests	Moni-ring wells
Soil profile	C/B	C/B	C/B	-	A	A	A	B	B	A	A	A	-
Classification	-	-	-	-	B	B	B	B	B	B	A	A	-
Water content	-	-	-	-	-	-	-	-	-	-	A	A	-
Pore water pressure	-	-	-	-	-	A	-	-	B/C	-	-	-	A
Permeability	-	-	-	-	-	B	-	-	B	-	C	A	B/C
Dry/wet density	-	-	-	A	C	C	C	-	-	-	C	A	-
Density index	-	-	-	-	B	B	B	-	C	C	-	A	-
Friction angle	-	-	-	-	B/C	B/C	B/C	C	C	C	-	A	-
Undr. shear strength	-	-	-	-	B	B	C	A	B	B	-	A	-
Compressibility	-	-	-	-	B/C	B/C	-	-	-	C	-	A	-
Rate of consolidation	-	-	-	-	-	A	-	-	A	-	-	A	C
Creep settlement	-	-	-	-	-	-	-	-	-	-	-	A	-
Elastiticy modulus	A	-	-	-	B	B	B	A/B	B	B	-	A	-
In situ stress	-	-	-	-	C	C	-	C	B	B	-	A	-
Stress history	-	-	-	-	C	C	C	B	B	B	-	A	-
Stress/strain curve	-	-	-	-	-	C	-	B	-	C	-	A	-
Liquefaction	-	-	-	-	A/B	A/B	A/B	B	B	-	-	A	-
Ground conditions													
Hard rock	A	-	A	A	-	-	-	-	A	-	A	A	C
Soft rock-till, etc.	A	-	A	A	C	C	C	-	A	C	A	A	A
Gravel	A	B	A	A	B/C	B/C	B	-	B	-	A	C	A
Sand	A	A	A	A	A	A	A	-	B	A	A	C	A
Silt	A	A	A	A	A	A	A/B	B	B	A	A	A	A
Clay	A	A	A	A	A	A	C	A	A	A	A	A	A
Peatl-organics	C	A	A	A	A	A	C	B	B	A	A	A	A

Table 6.2: In situ test methods and their perceived applicability (after Rock Manual 2007)

only just exceed the values for LT_A. For example, when a supplier in a heavy grading has 8% stones with a length-to-thickness ratio of more than 3, it cannot be marked at LT_A. However, for a filter layer one may accept 8%. In this case, the supplier marks the stones with LT_8.

In EN 13383 three classes of gradings are mentioned: Course Grading, Light Grading and Heavy Grading. Notice that Course Grading is smaller than Light Grading.

Course grading:	45/125 mm – 63/180 mm – 90/250 mm – 45/180 /mm – 90/180 mm
Light Grading:	5/40 kg – 10/60 kg – 40/200 kg – 60/300 kg – 5/300 kg
Heavy Grading:	0.3/1 ton – 1/3 ton – 3/6 ton – 6/10 ton – 10/15 ton

In EN 13383, the requirements for the sieve curve are exactly described. For all gradings there is a category A (additional requirement for the median weight) and a class B. For the armour layer one should use class A; for other layers class B is sufficient. The category 90/180 mm is especially for use in gabions.

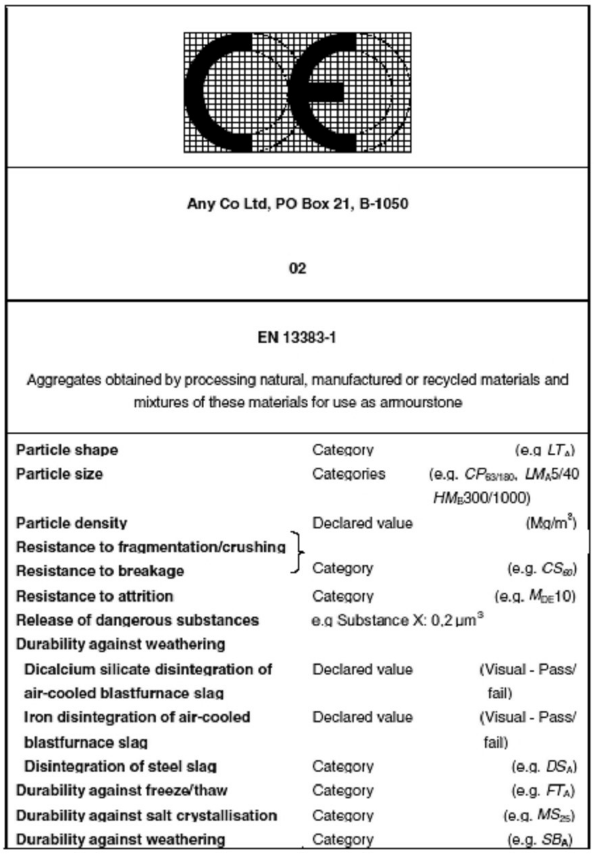
Figure 6.3 shows that, in fact, when a class B 0.3-1 ton is delivered ($HMB_{300-1000}$), the real distribution curve can be anywhere in the gray area indicated with I. When a class A is delivered, the sieve curve goes through the small horizontal line in the middle of the gray area.

EN 13383 also requires that the density of the stone is on the label; the average density has to be above 2.3 ton/m^3. Other parameters on the label are the resistance to wear and resistance to breakage. For more details on all parameters, one is referred to the Rock Manual [2007].

Concrete

When a large project is to be executed in a remote area, it is also essential to be sure of the quality and availability of other construction materials. For closure dams and breakwaters, it is almost impossible to avoid the use of concrete. It is therefore recommended that data on the availability and quality of cement, aggregates, water and reinforcing steel should be collected. It is also essential to study the climatological conditions to see if special measures are required for curing the fresh concrete.

In this respect, it is also important to be aware of any local codes and standards, and whether the obligatory sections thereof will interfere with procedures planned by the designer or contractor.



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02

EN 13383-1

Aggregates obtained by processing natural, manufactured or recycled materials and mixtures of these materials for use as armourstone

Particle shape	Category	(e.g LT_A)
Particle size	Categories	(e.g. $CP_{63/180}$, LM_A5/40 HM_B300/1000)
Particle density	Declared value	(Mg/m³)
Resistance to fragmentation/crushing **Resistance to breakage**	Category	(e.g. CS_{60})
Resistance to attrition	Category	(e.g. M_{DE}10)
Release of dangerous substances	e.g Substance X: 0,2 µm³	
Durability against weathering		
Dicalcium silicate disintegration of air-cooled blastfurnace slag	Declared value	(Visual - Pass/ fail)
Iron disintegration of air-cooled blastfurnace slag	Declared value	(Visual - Pass/ fail)
Disintegration of steel slag	Category	(e.g. DS_A)
Durability against freeze/thaw	Category	(e.g. FT_A)
Durability against salt crystallisation	Category	(e.g. MS_{25})
Durability against weathering	Category	(e.g. SB_A)

Figure 6.2: Requirements for CE marking

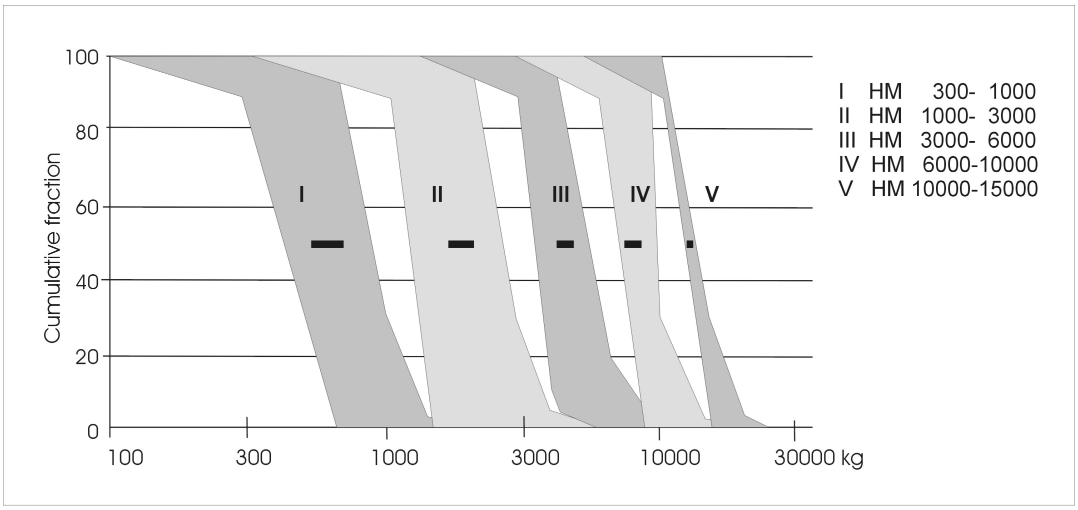


Figure 6.3: Weight distribution for Heavy Grading (EN13383)

6.5.2. EQUIPMENT

There is a high degree of interdependence between the design of a breakwater or closure dam and the construction method. In the same way, the equipment to be used depends largely on the construction method and vice versa.

Similarly, questions relating to the maintenance and repair of the final structure must be discussed during the design stage. Do we rely upon regular inspection and maintenance, or do we opt for a more or less maintenance-free structure? For construction stages (and a closure dam is usually considered a construction stage), one need not worry too much about maintenance and repair. Even if a minor part of the works should be lost, the contractor is still there with his equipment to take care of repairs. Nevertheless it makes sense to analyse the construction risk, since this may constitute a considerable part of the construction cost.

The above queries lead to the main question of whether locally available equipment will be used or must the required heavy equipment be obtained from elsewhere. If local equipment is preferred, it is necessary to obtain a detailed assessment of the quality, capacity and cost of such equipment. If it is decided to mobilize the equipment, the questions that arise are how to get the equipment to the location, and whether there are any restrictions on temporary import. Local conditions like temperature (cooling of engines), dust (capacity of air filters), quality of fuel and lubricants and availability of spare parts also play a role.

6.5.3. LABOUR

When planning a large project, it is also essential to know whether there is a skilled local labour force and whether the employment of skilled and partly skilled expatriate labour is permitted. In many cases, it is necessary to provide special facilities for the accommodation of personnel. Such facilities must be available from the start of the actual construction. Poor working and living conditions will have a strong negative influence on the quality of the work.

7

Stability of randomly placed rock mounds

In this Chapter, attention will be paid to the stability of individual stones on a sloping surface under wave attack, i.e. stability of the armour layer. Due to the complex water movement of waves breaking on a slope, it is not yet possible to derive a satisfactory theoretical expression for the forces on and the stability of such stones. This means that this chapter contains a multitude of empirical formulae, all based on the results of small-scale experiments. Although it would not be wise to learn all these formulae by heart, it is necessary to understand them and their significance to the designer of a breakwater. Although the stability of individual stones on a slope under wave attack is certainly not the only criterion for the proper functioning of a rubble mound breakwater, it deserves great attention. This is because many breakwaters have failed due to a defective design in this respect.

7.1. Stability formula for rock

7.1.1. General

As indicated in Chapter 2, for many years breakwater design was a question of trial and error. It was shortly before World War II that, in an attempt to understand the influence of rock density, Iribarren developed a theoretical model for the stability of stone on a slope under wave attack. Iribarren continued his efforts throughout the years until his final publication on the subject at the PIANC Conference of 1965 in Stockholm.

In the meantime, in the USA, the US Army Corps of Engineers had developed a keen interest in the stability of breakwaters, and long series of experiments were carried out by Hudson at the Waterways Experiment Station in Vicksburg.

Where Iribarren focussed on a theoretical approach, assisted by some experiments, Hudson concentrated on collecting a large data set from hydraulic model experiments to derive conclusions from an analysis of those data. In both cases, experiments were carried out using the then standard techniques, i.e. by subjecting the models to regular, monochromatic waves.

The experiments comprised the construction of an infinitely high slope, covered with stones of a particular weight, shape and density. The slope was then exposed to a wave train with waves of a particular height and period, starting with low waves and increasing their height in steps, until loss of stability of the stones was observed. It must be kept in mind that loss of stability is not a clearly defined phenomenon. Some subjectivity is involved, in particular because the loss of the first stones cannot always be entirely attributed to wave action since it may at least in part, be due to the random position of the stone after construction. In the following sections, the work of Iribarren and Hudson will be explained in more detail.

Also one should realise that the wave steepness was kept rather constant by the experimenters (they used wave steepnesses in the order of 3% to 4%, but without analysing the effect of the wave steepness in detail. Also the model breakwaters were very permeable.

7.1.2. Iribarren

Ramón Iribarren Cavanilles [1938, 1950, 1953, 1954, and 1965] considered the equilibrium of forces acting on a block placed on a slope. Since the considerations of Iribarren referred to forces, the weight of the block W

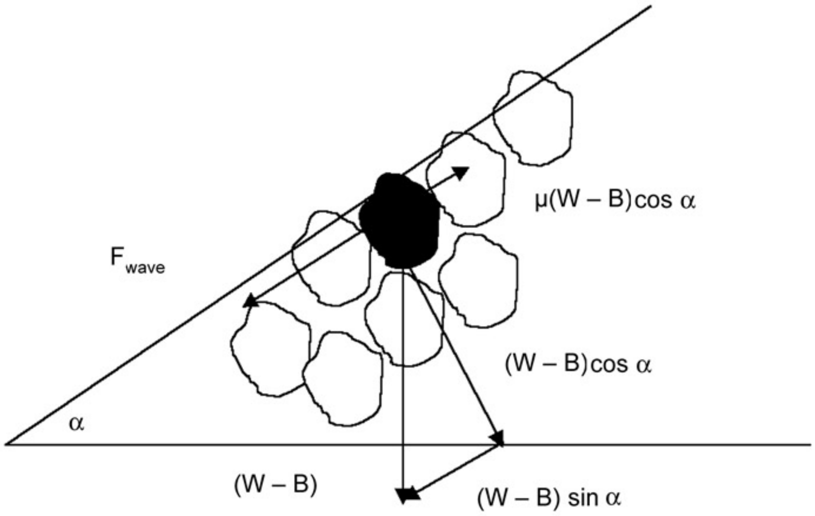


Figure 7.1: Equilibrium after Iribarren (downrush)

is introduced as a force, and thus expressed in Newton. The forces acting on a unit positioned on a slope at an angle α are (see Figure 7.1):

- Weight of the unit (vertical downward)
- Buoyancy of the unit (vertical upward)
- Wave force (parallel to the slope, either upward or downward)
- Frictional resistance (parallel to the slope, either upward or downward, but contrary to the direction of the wave force)

Iribarren resolved these forces into vectors normal and parallel to the slope. Loss of stability occurs if the friction is insufficient to neutralize the other forces parallel to the slope. The parameters are:

W	= weight of block	[N]
B	= buoyancy of block	[N]
$W-B$	= submerged weight of block	[N]
V	= volume of block	[m^3]
α	= angle of slope	[-]
μ	= friction coefficient	[-]
ρ_s	= density of block (rock)	[kg/m^3]
ρ_w	= density of (sea) water	[kg/m^3]
Δ	= $(\rho_s - \rho_w)/\rho_w$	[-]
H	= wave height	[m]
d_n	= characteristic size of stone = $V^{1/3}$	[m]
F_{wave}	= wave force	[N]
g	= acceleration of gravity	[m/s^2]

Iribarren assumed a set of simple relations between F_{wave}, D_n, H, ρ and g as follows:

$$F_{wave} = \rho_w g d_n^2 H \tag{7.1}$$

Note: For an element in permanent flow the force $F = C_i u^2 d_n^2$ and also the maximum velocity at the bed in a wave is proportionally to the wave height; compare also the formula of Izbash, developed in the same period (see SCHIERECK [2000], p. 51).

$$W - B = (\rho_s - \rho_w) g d_n^3 \tag{7.2}$$

and

$$W = \rho_s g d_n^3 \tag{7.3}$$

It must be mentioned here that these relations may be criticized, since they are too simple. It must be expected that the shape of the block and the period of the wave play a role. Furthermore, the relation between the wave force and the wave height and stone size indicate the dominance of drag forces, whereas acceleration forces are neglected.

In case of downrush for stability, the following condition is required:

$$F_{wave} < N\{\mu(W-B)\cos\alpha - (W-B)\sin\alpha\}$$
$$\rho_w g d_n^2 H < N\{(W-B)(\mu cos\alpha - \sin\alpha\}$$
$$\rho_w g d_n^2 H < N\{(\rho_s-\rho_w) g d_n^3 (\mu cos\alpha - \sin\alpha\}$$

$$\frac{H}{\Delta d_n} < N(\mu\cos\alpha - \sin\alpha) \tag{7.4}$$

In case of uprush, a similar equation can be derived

$$\frac{H}{\Delta d_n} < N(\mu\cos\alpha + \sin\alpha) \tag{7.5}$$

When Equations 7.4 and 7.5 are expressed in terms of required block weight, as Iribarren originally did, this leads to:

$$W \geq \frac{N\rho_s g H^3}{\Delta^3(\mu\cos\alpha - \sin\alpha)^3} \quad \text{(for downrush)} \tag{7.6}$$

$$W \geq \frac{N\rho_s g H^3}{\Delta^3(\mu\cos\alpha + \sin\alpha)^3} \quad \text{(for uprush)} \tag{7.7}$$

N is a coefficient that depends, amongst other factors, on the shape of the block, and its value must be derived from model experiments. The friction factor μ can be measured by tilting a container filled with blocks and determining the angle of internal friction. In Iribarren [1965], recommendations are given for values of N and μ. The most important values are given in Table 7.1. The values of N refer to zero damage.

type of block	downward stability $(\mu\cos\alpha - \sin\alpha)^3$		upward stability $(\mu\cos\alpha + \sin\alpha)^3$		transition slope between upward and downward stability
	μ	N	μ	N	$\cot\alpha$
rough angular quarry stone	2.38	0.430	2.38	0.849	3.64
cubes	2.84	0.430	2.84	0.918	2.80
tetrapods	3.47	0.656	3.47	1.743	1.77

Table 7.1: Coefficients for Iribarren formula

It must be kept in mind that the coefficient N represents many different influences. At first, it is a function of the damage level defined as "loss of stability". It also includes the effect of the shape of the blocks but not the internal friction, because this is accounted for in the separate friction coefficient. Finally, it covers all other influences not accounted for in the formula.

The friction coefficient μ seems to be on the high side, but is clearly related to the test procedure that Iribarren used. He found a large difference in friction, depending on the number of units in the slope. For details, one is referred to his original publication.

Iribarren concluded on a theoretical basis that the stability of rock is (amongst others) a function of the slope, described as $(\mu\cos\alpha \pm \sin\alpha)^3$. Later investigators mainly performed a curve fitting on the relevant parameters and found a good correlation with $\tan\alpha$. Numerically this gives the same accuracy in the area of interest (relatively steep slopes), however it is fundamentally less correct.

7.1.3. HUDSON AND VAN DER MEER

Because of the political situation in Spain around the 2nd World War, the method of Iribarren did not get attention in a large part of the world. In the period after the 2nd World War, in the US, research started on this matter, resulting in the formula by HUDSON [1953, 1959, and 1961] based on tests with regular waves performed at the Waterways Experiment Station in Vicksburg, USA. This formula has been adapted several times. The formula is mainly based on a dimension analysis and curve fitting of data. All unknown factors are included in the 'dustbin' factor k_D. The formula has been adapted several times, also to make it possible to include random waves (Shore Protection Manual 1984 and the present Coastal Engineering Manual 2002). The most recent version of this formula is:

$$W_{50} = \frac{\rho_s g H_{1/10}^3}{K_D \Delta^3 \cot\alpha} \tag{7.8}$$

For rock, K_D is between 3 and 6 depending on the storm type, acceptable damage, etc. For rock, this formula is not often used any more, but this equation can be found in many equations for concrete armour units. This application will be discussed in Section 0.

Around 1985 in the Netherlands, an extensive research program started, resulting in the thesis of VAN DER MEER [1988]. On the basis of his experiments Van der Meer found that a distinction should be made between plunging and surging breakers. This resulted in the following set of equations:

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \quad \text{for plunging waves} \tag{7.9}$$

$$\frac{H_s}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{cot\alpha}\, \xi_m^P \quad \text{for surging waves} \tag{7.10}$$

For the plunging waves a value of $c_{pl} = 6.2$ gives the average expected value of $H_s/\Delta d_{n50}$, for design a value of $c_{pl} = 5.5$ is recommended (this is the 95% exceedance value, the standard deviation of $c_{pl} = 0.4$, so the 95% value is $6.2 - 1.64 \cdot 0.4 = 5.5$. The recommended values for c_s are 1.0 and 0.87 (= $1.0 - 1.64 \cdot 0.08$). For constructions in deep water these formulas are still the most common formulas; details will be discussed in the following section.

7.1.4. MODERN STABILITY FORMULAE

In this chapter, modern stability formulae are discussed. First a rather general formula will be presented, later some simplifications will be presented which lead to different formulae frequently used in literature. Therefore, this chapter does not follow the history of the development of these formulae.

The stability of rock can be expressed with the dimensionless parameter $H/\Delta d_n$. The higher this relation is, the larger the waves that can be accommodated by the same stone size. This number has to be a function of the structure side slope, as discussed before.

An other important parameter is the Iribarren Number, $\xi = \tan\alpha/\sqrt{s}$ where s is a wave steepness [1]. Also the permeability of the structure is important (because large internal reflection inside the breakwater will result in a heavier load on the rocks.) Furthermore, one should use a design period which gives more emphasis to the longer periods in the spectrum that the shorter waves. Such a value is $T_{m-1,0}$ (see eq. 5.40). Also, the stability depends more on the higher waves in the short term distribution; so therefore the value $H_{2\%}$ is a better value than H_s.

Following the dimension analysis of VAN DER MEER [1988], leads to the following formulae:

$$\frac{H_{2\%}}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(s_{m-1,0}\right)^{0.25} \sqrt{cot\alpha} \quad \text{for plunging waves} \tag{7.11}$$

$$\frac{H_{2\%}}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(s_{m-1,0}\right)^{-0.25} \left(\xi_{m-1,0}\right)^{P-0.5} \quad \text{for surging waves} \tag{7.12}$$

The transition between plunging waves and surging waves can be calculated using the critical value for the surf similarity parameter:

[1] In the Iribarren Number a ratio between wave height and wave period is needed. However, because this ratio needs to be dimensionless, the period is multiplied with $g/2\pi$. Thus the "deep water" wavelength is used. In shallow water the value of s is the local wave height divided by the deep water wavelength, consequently it is not the real steepness of the wave at that location. Therefore it might be better to call s the "fictitious steepness".

$$\xi_{cr} = \left[\frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}} \tag{7.13}$$

For $\xi_{s-1,0} < \xi_{cr}$ waves are plunging and Equation 7.11 applies; For $\xi_{s-1,0} \geq \xi_{cr}$ waves are surging and Equation 7.12 applies. In these formulae the following parameters are present:

$H_{2\%}$	= wave height exceeded by 2% of the waves in the (short term) wave distribution
$T_{m-1,0}$	= period of the waves, calculated from the first negative moment of the spectrum (see eq. 5.19)
$s_{m-1,0}$	= fictitious wave steepness: $2\pi H_{2\%}/\left(gT^2_{m-1,0}\right)$
$\xi_{s-1,0}$	= surf similarity parameter: $\tan\alpha/\sqrt{s_{m-1,0}}$
d_{n50}	= nominal median block diameter, or equivalent cube size, $d_n = (M/\rho_s)^{1/3}$.
Δ	= relative mass density $(\rho_s - \rho_w)/\rho_w$ where ρ_s is mass density of stone and ρ_w is mass density of water
N	= number of waves
S	= damage level $A/(d_{n50})^2$, where A = erosion area in a cross-section
α	= angle of the seaward slope of a structure
P	= notional permeability coefficient

Depending on slope and permeability, the transition lies between $\xi_{s-1,0} = 2.5$ and 4. From experiments followed that $c_{pl} = 8.4$ and that $c_s = 1.35$; this implies a ratio $c_{pl}/c_s = 6.2$. Use of the values of 8.4 and 1.35 leads to the expected value when damage occurs. These numbers have a standard deviation of 0.7 and 0.15, so that for a safe design, a 5% exceedance value is recommended. So for design one should use the values 7.25 and 1.05.

In the equations, the fictitious wave steepness is used. This is a parameter to include the effect ratio between wave height and wave period. The equation is similar to the equation of the steepness, but it is not real steepness; it is the local wave height, divided by the deep water wave length (calculated with the local period). Therefore, it is called fictitious.

For analysing the equations, an example is used. The standard case is:

$H_s = 2$ m	$H_{2\%} = 2.8$ m	$T_{m-1,0} = 6$ s
$\cot\alpha = 3$	$N = 3000$	$\Delta = 1.65$
$P = 0.5$	$d_{n50} = 0.6$ m (300-1000 kg)	$S = 2$

(Note: This is the same case as used in SCHIERECK [2000]).

WAVE STEEPNESS

When the Stability Number increases, the stability of the construction also increases. In other words, when the Stability Number is higher, the same stone is stable in case of a wave larger than $H_{2\%}$. From the equations, it follows that the Stability Number has a minimum at the location of the critical Iribarren number (ξ_{cr}). This is indicated in Figure 7.2. From the critical number, the value of the Stability Number increases as a function of the steepness. Because of this, the power of the fictitious steepness in the equations is negative for plunging waves, and positive for surging waves. It is important to notice that the fictitious wave steepness is in fact a more relevant parameter than the period as such. When plots are made of the Stability Number as function of the period, it seems that the period is relevant. However, in those cases the wave height is usually kept constant. In nature, during a storm, there is a high correlation between period and wave height; the fictitious wave steepness remains constant. However, in those cases when the design condition is not a storm wave, but for example a swell wave or a diffracted wave, wave steepness can be less.

PERMEABILITY

In case of a low permeability of the sub layer, the waves are reflected against the sub layer and subsequently increase the lift forces on the armour layer. The permeability has therefore an influence on the stability of the armour layers. This is expressed via the 'notional permeability' as a factor P (as defined by VAN DER MEER, [1988]), for which he indicates values based on a global impression of the stone size in subsequent layers (Figure 7.3). It is emphasized that, in fact, P is not a permeability parameter, although it is referred to as being the permeability parameter. It merely indicates the composition of the breakwater in terms of the mutual relations of the grain sizes in subsequent layers.

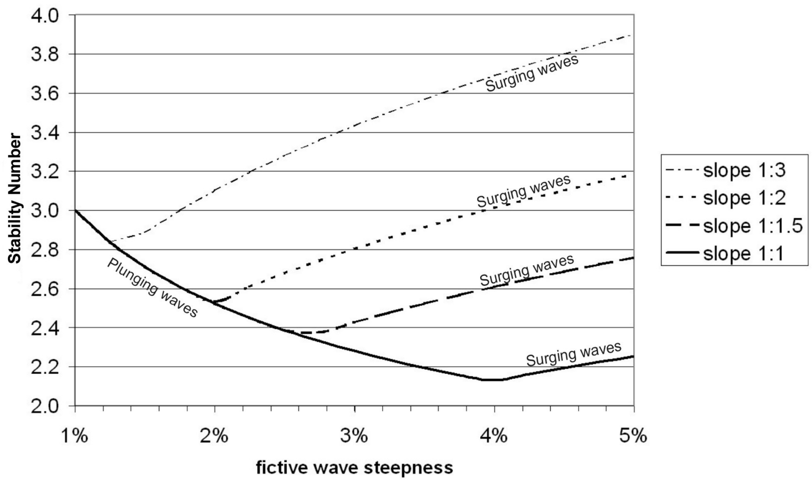


Figure 7.2: Example of the relation between Stability Number and fictive wave steepness

VAN GENT ET AL. [2004] have published a formula where the notional permeability P is replaced by an expression including the diameter of the armour and the diameter of the core. However, because this formula has other parameters than Equations (7.11) and (7.12), one cannot easily transfer the expression of Van Gent into Equations (7.11) and (7.12).

The advantage of the permeability expression of Van Gent is that it yields a value that can be derived from factual material properties, while the value P in the Van der Meer approach only can be estimated. The disadvantage of the Van Gent expression is that it is limited to standard breakwaters and a limited range of wave steepnesses.

DAMAGE LEVEL

The level of damage is expressed by the parameter:

$$S = \frac{A}{d_{n50}^2} \tag{7.14}$$

in which:

A	= the erosion area in a cross section	[m^2]
d_{n50}	$= \left(\frac{W_{50}}{g\rho_s}\right)^{1/3} = \left(\frac{M_{50}}{\rho_s}\right)^{1/3}$	[m]
W_{50}	= mean weight of armourstones	[N]
ρ_s	= density of armourstone	[kg/m^3]

For a definition sketch, refer to Figure 7.4. The area A is often measured by using a rod with a hemisphere of a specific size attached to it.

The erosion in the area A is partly caused by settlement of the rock profile and partly by removal of stones that have lost stability. Since the erosion area is divided by the area of the armourstone, the damage S represents the number of stones removed from the cross-section, at least when permeability/porosity and shape are not taken into account. In practice, the actual number of stones removed from a d_{n50} wide strip is between 0.7 and 1.0 times S.

If the armour layer consists of two layers of armour units, one can define limits for acceptable damage and failure. These limits are more liberal for gentler slopes, since in that case, the damage is distributed over a larger area. Critical values for S are given in Table 7.2.

For rather uniform units the damage value S is often replaced by the damage numbers N_{od}. This is the number of displaced armour units within a strip of breakwater with a width d_{n50}. The relation between S and N_{od} is given by:

$$N_{od} = G(1 - n_v)\,S \tag{7.15}$$

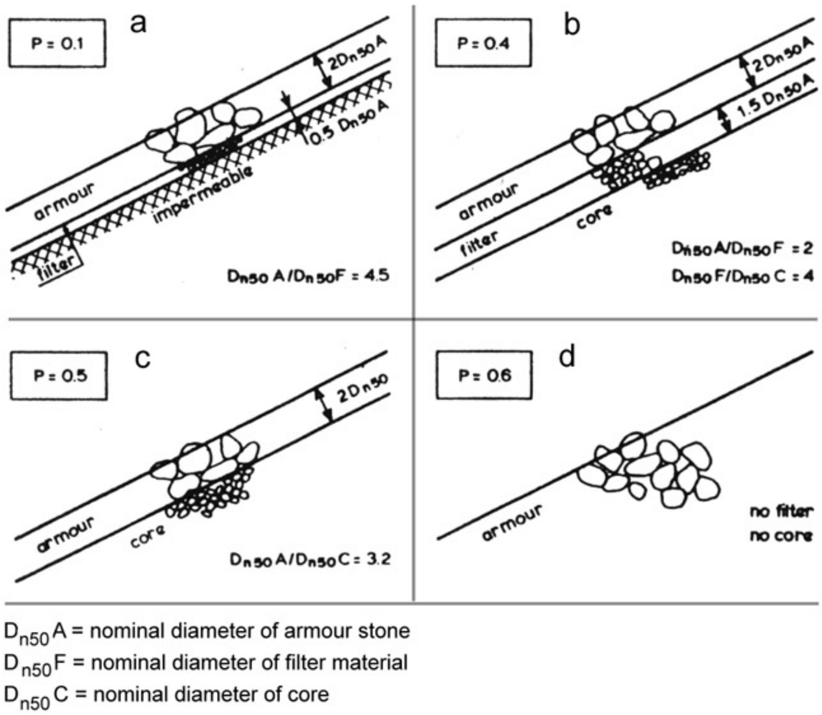


Figure 7.3: Notional permeability coefficients for various structures

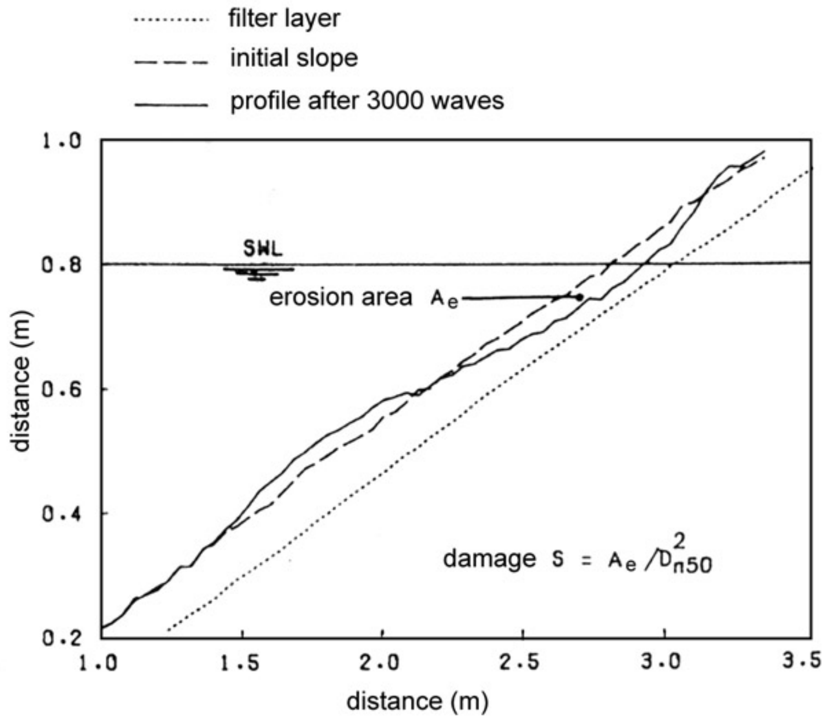


Figure 7.4: Damage(S) based on erosion area (A)

Slope	Initial Damage (needs no repair)	Intermediate Damage (needs repair)	Failure (core exposed)
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4	3	8 – 12	17
1:6	3	8 – 12	17

Table 7.2: Classification of damage levels S for quarry stone

in which G is the Gradation factor (-) depending on the armour layer gradation; for uniform units, like concrete elements, $G = 1$. The porosity of the layer is given as n_v. For concrete armour units $n_v = 0.45$–0.55. This will further be elaborated when discussing concrete armour units.

Deep and shallow water

In Equation (7.11) and (7.12) the wave height $H_{2\%}$ and the wave period $T_{m-1,0}$ is used. Experiments have shown that these values describe the processes in the best way. However, usually these values are not given as boundary condition and have to be determined using a spectral wave model (e.g. Swan) to determine $T_{m-1,0}$ from the T_p on deep water and the Battjes-Groenendijk approach to determine $H_{2\%}$ from the H_s.

For deep water these relations are rather constant. Therefore, for deep water the equations (7.11) and (7.12) can be reduced to:

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \quad \text{for plunging waves} \tag{7.16}$$

$$\frac{H_s}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{cot\alpha}\, \xi_m^P \quad \text{for surging waves} \tag{7.17}$$

In these equations one should use $c_{pl} = 6.2$ and $c_s = 1.0$ as predictors for the average damage. For design values, a 5% value of these coefficients is recommended (5.5 and 0.87). The transition between plunging and surging waves remains the same (eq. 7.8) [2]. Note that now $\tan\alpha$ moved from the "plunging" equation to the "surging" equation. This is because the deep water equations are written in terms of ξ, and not in terms of s. In this equation H_s is the significant wave height (in deep water) and ξ_m is based on the mean period ($\xi_m = \tan\alpha / \sqrt{(2\pi H_s / g T_m^2)}$).

Equations (7.16) and (7.17) were initially presented by VAN DER MEER [1988] in his Ph.D-thesis.

Further simplifications

The Van der Meer equation (for plunging waves) can be re-written in the following form:

$$\frac{H_{2\%}}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(s_{m-1,0}\right)^{0.25} \sqrt{cot\alpha} \tag{7.18}$$

For slopes between 1V:1H and 1V:2H, $(\cot\alpha)^{0.5} \pm 1.08(\cot\alpha)^{1/3}$. Furthermore, for deep water and storms still under development, the wave steepness is in the order of 4%, while the maximum number of waves is 7500. For a normal riprap breakwater the notional permeability is $P = 0.6$. Entering these values in Equation 7.18 leads to:

$$\frac{H_{2\%}}{\Delta d_{n50}} = 1.25\,(\cot\alpha)^{1/3} \tag{7.19}$$

This can be rewritten as:

$$\frac{H_{2\%}}{\Delta d_{n50}} = \frac{(K_D \cot\alpha)^{1/3}}{1.27} \tag{7.20}$$

where $K_D = 4$. Because $H_{1/10} = 1.27 H_s$ one can write this as:

[2] When using the standard relations for converting $H_{2\%}$ to H_s and $T_{m-1,0}$ to T_p in deep water this leads to a slightly different number. This might be due to the effect of the foreshore. Research on this matter is still ongoing.

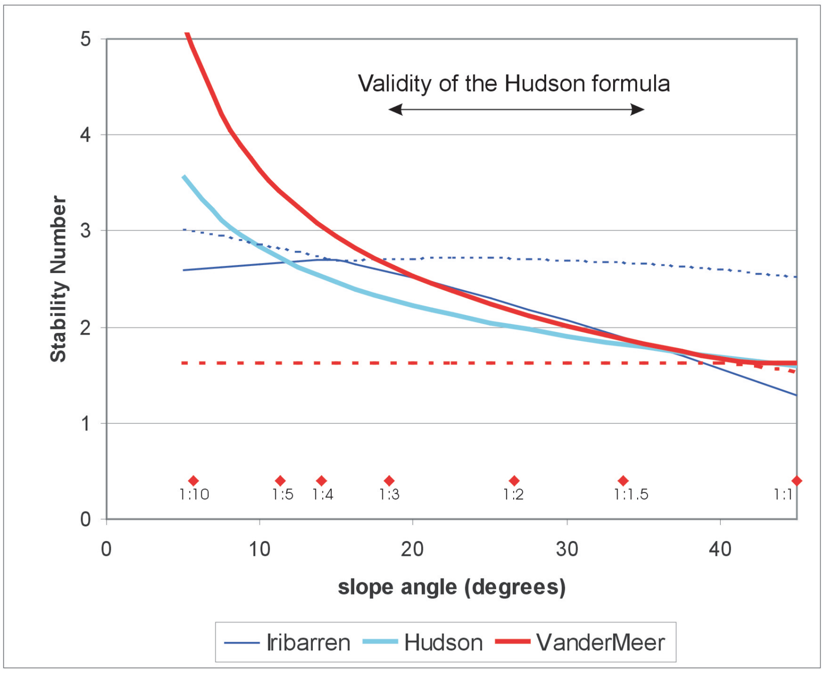


Figure 7.5: Variation of the Stability Number as function of the slope for the example (the dashed lines indicate the non-valid branches of the formula)

$$\frac{H_{2\%}}{\Delta d_{n50}} = (K_D \cot\alpha)^{1/3} \tag{7.21}$$

or:

$$W_{50} = \frac{\rho_s g H_{1/10}^3}{K_D \Delta^3 \cot\alpha} \tag{7.22}$$

which is the Hudson formula as it is presented in the US literature (Shore Protection Manual 1984 and the present Coastal Engineering Manual 2002). This formula is also used for concrete armour units, because usually the slope, the permeability, the design wave steepness, etc. are constant. In that case the only remaining extra parameter is the effect of the shape of the unit. For each concrete unit therefore a specific K_D-value is recommended. This matter will be discussed later in more detail.

7.1.5. COMPARISON OF THE IRIBARREN, VAN DER MEER AND HUDSON FORMULAE

When comparing the formulae of Iribarren, Van der Meer and Hudson, the difference appears to be greater than it actually is. The influences of wave height, rock density and relative density are equal. The coefficients are different but can easily be compared. The main difference occurs in the influence of the slope. A comparison of the equations within the validity area of the Hudson formula ($1.5 < \cot\alpha < 4$) reveals that the correct choice of coefficients leads to a similar outcome for all formulae. It is evident that for very steep slopes (close to the angle of natural repose) Hudson and Van der Meer cannot give a reliable result. It is also likely that for very gentle slopes, waves will tend to transport material up the slope; a factor that was not considered by Hudson at all.

Using the same example as given before, one can plot the Stability Number as a function of the slope angle for the three formulae (see Figure 7.5).

The dashed lines indicate parts where the formulae are not valid (for Van der Meer this depends on plunging/surging, for Iribarren this depends on uprush or downrush). The figure illustrates that for normal slopes (between 1V:1.5H and 1V:3H) there is not much difference between the formulae. For very gently slopes, the approach of Iribarren is much better; but this part is not very interesting for breakwater design.

7.2. CONCRETE ARMOUR UNITS

7.2.1. HISTORICAL OVERVIEW

The availability of large rock is limited. Natural blocks with a weight of more than 10 tons are very rare, but in some cases such heavy blocks are required. In such cases, the use of artificial blocks made of concrete becomes an interesting alternative. Also, one may use special shapes which interlock. Three approaches are distinguished:

- The use of units that obtain their resistance mainly by their weight;
- The use of units that obtain their resistance mainly by interlocking between elements;
- The use of units that obtain their resistance mainly by friction between elements

The last group consists of pattern placed concrete blocks and columns, mainly used in block revetments. These blocks will not be discussed here. One is referred to SCHIERECK, (2001) for more information about this type of armouring. Blocks mainly functioning due to their weight consist of randomly placed cubes which can either be smooth or grooved (also called Antifer cubes.) These elements are always placed in double layers.

Classical interlocking blocks are the Tetrapod and the Dolos. Because of the interlock, the units themselves can be made lighter and more slender. This means less mass per running meter, and is therefore more economic. The most economic block shapes are very slender blocks, like Dolos. However, due to higher fragility, their use is limited to smaller sizes. This reduces their applicability. It must be realized that special block shapes are costly because of the higher cost of the moulds, the labour intensive use of the moulds, and the difficulties of handling and stacking the blocks.

The use of various kinds of specially shaped concrete armour units has become popular. This started with the development of Tetrapods by Sogreah, followed by the Akmon, Dolos, and many others. The merit of the Tetrapod was that it demonstrated that by interlocking, a K_D value could be obtained that was about twice as high as the values for quarry stone, thus leading to amour unit requirements of half the weight. The disadvantage was the complicated shape, requiring an expensive mould. One of the main reasons to start the development of the Akmon by Delft Hydraulics was to find a way to overcome the Sogreah patent.

Following the development of the Akmon, Merrifield and Zwamborn in South Africa attempted to increase the permeability/porosity of the Akmon while maintaining its basic shape by making the legs more slender. This work lead to (unpatented) Dolos Initially, this was very promising, yielding K_D values of 20 and higher. At the same time, sizes of sailing vessels were increasing, requiring longer breakwaters which extend into deeper water with higher waves. This resulted in the design of breakwaters (Sines, Portugal is the most striking example) with very large unreinforced units. In a very short period thereafter, a large number of breakwaters failed. It was discovered that the mechanical strength of the concrete was inadequate to resist the forces, especially during rocking of the units against each other. This was never investigated in a model. Even if tests had been carried out, the results would have been of no use due to scale effects.

Following these incidents, guidelines like the Shore Protection Manual recommended that a more conservative approach should be adopted and that the rocking of slender units larger than 20 to 25 tons should be prevented. Another recommendation was to avoid the use of slender units altogether and to rely upon simple cubes. Although the required weight of a cube is greater than for the more sophisticated shapes, considerable savings are achieved on the cost of moulds, the cost of casting, and the storage and handling costs. Because a number of Dolos breakwaters needed repair, the US Army Corps of Engineers started the development of a unit similar to the Dolos, but less sensitive to breakage. This resulted in the development of Core-Loc.

In 1981 Sogreah developed a massive block, the Accropode, which can be used in a single layer, provided it is carefully placed in a specific pattern. Sogreah makes this unit available without the payment of royalties, on condition that both the design and method of placing are checked and approved by Sogreah. Since 2001 an alternative to the Accropode, the Xbloc, of Delta Marine Consultants, is available. In 2004 Sogreah had developed an improved version of the Accropode, the Accropode II. For details on concrete armour units, refer to Appendix 3.

7.2.2. STABILITY CALCULATIONS FOR CONCRETE BLOCKS

When testing armour layers of artificial material like concrete, it makes no sense to vary the slope of the breakwater. Since the block weight is not so strictly limited as it is for quarry stone (the quarry has a clear maximum block size), it is much more effective to increase the concrete block weight than to reduce the slope. Because of gravity the interlock increases for steeper slopes, so it is effective to make a slope as steep as possible. It is even so that interlocking blocks are quite unstable on the (nearly) horizontal parts of a breakwater, like the crest. This makes using the Iribarren number ξ in a formula less relevant, since this expresses the influence of both, wavelength (or period) and slope. All formulae for concrete units, except Accropodes, are based on a slope of 1V:1.5H. (Accropodes are placed on a slope 1V:1.33H)

Since the mechanical strength of concrete blocks may play a role in breakwater failure, it is useful to distinguish damage due to displaced units (their number is indicated by N_{od}) and damage due to blocks that might break because they are rocking against each other (their number is indicated by Nor). The total number of moving units (N_{omov}) is equal to the number of displaced blocks, plus the number of rocking blocks i.e. $N_{omov} = N_{od} + N_{or}$. The value of N_{od} is comparable with the value of S; compare equation (7.14). S is about double the value of N_{od}.

Van der Meer [1988] gives the stability for various frequently used blocks (table 7.3). He makes a distinction between displaced blocks and moving blocks. The difference appears to be a reduction of the stability number by 0.5. The scatter of data for cubes and Tetrapods is normally distributed and has a relative standard deviation $\sigma/\mu = 0.1$.

Note that d_n is the nominal diameter of the unit, or the cubic root of the volume. For various blocks this leads to:

Cubes	d_n = equal to the side of the cube
Tetrapods	$d_n = 0.65d$ if d is the height of the unit
Dolos	$d_n = 0.54d$ if d is the height of the unit (waist ratio 0.32)
Accropode	$d_n = 0.7d$ if d is the height of the unit
Xbloc	$d_n = 1/\sqrt{3}d$ if d is the height of the unit

Like the damage levels for quarry stone, damage levels can also be classified for concrete units as in Table 7.3.

Block Type	Relevant N-value	Start of Damage	Initial Damage (needs no repair)	Intermediate Damage (needs repair)	Failure (core exposed)
Cube	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Tetrapod < 25 ton	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Tetrapod >25 ton	N_{omov}	0	0 – 0.5	0.5 – 1.5	> 2
Dolos < 20 ton	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Dolos > 20 ton	N_{omov}	0	0 – 0.5	0.5 – 1.5	> 2
Core-loc	N_{od}	0			>0.5

Table 7.3: Classification of damage levels N_{od} and N_{omov} for concrete multi-layer elements (slope 1V:1.5H)

Note: In table 7.3 values are given for the condition that the unit itself is not damaged. Especially in case of the Dolos this is a condition which is difficult to fulfil.

CUBES

$$\frac{H_s}{\Delta d} = \left(6.7\frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \tag{7.23}$$

$$\frac{H_s}{\Delta d} = \left(6.7\frac{N_{omov}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} - 0.5 \tag{7.24}$$

TETRAPODS

$$\frac{H_s}{\Delta d} = \left(3.75\frac{N_{od}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} \tag{7.25}$$

$$\frac{H_s}{\Delta d} = \left(3.75\frac{N_{omov}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} - 0.5 \tag{7.26}$$

DOLOS

HOLTZHAUSEN AND ZWAMBORN [1992] investigated the stability of Dolos with the following result:

$$N_{od} = 6250\left[\frac{H_s}{\Delta^{0.74} d_n}\right]^{5.26} s_{op}^3 w_r^{20 s_{op}^{0.45}} + E \tag{7.27}$$

in which: w_r = Waist ratio of the Dolos
E = Error term

The waist ratio has been made a variable in the Dolos design to enable the choice of a less slender shape with less chance of breaking. Waist ratios are between 0.33 and 0.4. The error term E represents the reliability of the formula. It is normally distributed and has a mean value equal to zero, and a standard deviation $\sigma(E)$:

$$\sigma(E) = 0.01936\left[\frac{H_s}{\Delta^{0.74} d_n}\right]^{3.32} \tag{7.28}$$

CORE-LOC

Because a number of Dolos breakwaters needed repair, a special unit has been developed by the US Corps of Engineers for Dolos breakwater rehabilitation. This block, the Core-loc [MELBY AND TURK, 1997], can also be used for new breakwaters. For Core-loc stability a Hudson type formula is recommended:

$$\frac{H_s}{\Delta d_n} = 3.7 \quad \text{(Start of damage, } N_{od} = 0\text{)} \tag{7.29}$$

$$\frac{H_s}{\Delta d_n} = 4.1 \quad \text{(Failure, } N_{od} > 0.5\text{)} \tag{7.30}$$

7.2.3. SINGLE LAYER ELEMENTS

In recent years a number of special units have been developed for use on steep slopes in single layer. These elements can be placed on a slope of 1V:1.33H. Examples of these elements are the Accropode, developed by Sogreah, and the Xbloc, developed by Delta Marine Consultants [REEDIJK ET AL, 2003]. These elements can only be applied under special license from the developers.

Placing of the elements has to be done in accordance with detailed specifications provided by the developer. For example placing scheme for Xbloc is given in Appendix 3. It seems that there is no influence of storm duration; and because these units are only applied on a slope of 1V:1.333H (3V:4H), the slope is not part of the stability formula. This formula thus becomes quite simple; $H_s/\Delta d_n$ equals a given number that is developed on the basis of model experiments (Table 7.4).

	Accropode	Xbloc
Start of damage ($N_{od} = 0$)	3.7[a]	3.5[b]
Failure ($N_{od} > 0.5$)	4.1[a]	4.0[b]
Design value	2.5[a,b]	2.8[b]

a) Based on 2D-tests by Van der Meer. b) Based on 2D-tests by developers.

Table 7.4: ($H_s/\Delta d_n$) for single layer elements (slope 1H:1.33V)

The values for "start of damage" and "failure" as given in the table above can be considered as stochastic variables with a standard deviation of 0.2. Because wave heights causing failure are only slightly higher than the wave height associated with "start of damage", a higher safety coefficient has to be applied than for normal rock structures. Van der Meer recommends for Accropode a design value of 2.5. Tests have indicated that failure only occurs at values of around 4. Also for Xblocs, the recommended value for design is much

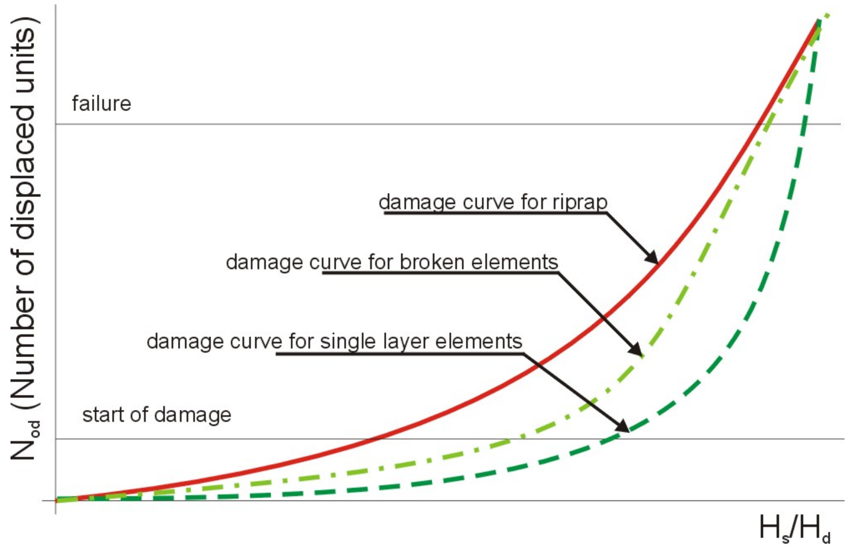


Figure 7.6: Damage progression for different type of armour units (from DE ROVER [2007])

lower than the value for failure. This is necessary because the single layer units have a much more "brittle" behaviour than double layer systems, see also Figure 7.6.

7.3. STABILITY CALCULATION

The computation for the stability of armour units can be made by substituting the significant wave height of the design storm into the stability equations like (7.11) or (7.14). In the computation, one can use a deterministic approach based on partial safety coefficients, or one can use a full probabilistic approach. In case of a design storm one determines the H_s with the method as described in Chapter 5, using a probability of exceedance developed along the lines as described in Chapter 3.

In case of using the deterministic approach with partial safety coefficients one uses the H_s with a probability of exceedance of once per lifetime (e.g. 1/50), and includes in the design equation two partial safety coefficients; one for the strength and one for the load. The values to be used follow from international guidelines, such as PIANC/MarCom 12 [1992]. It is also possible to use a self-developed design wave height (Chapter 3) and to use the safety coefficient for the strength parameters only.

In a probabilistic approach one re-writes the design equation as a Z-function (see Section 3.6), and determines the probability of $Z < 0$, given the distributions of all parameters in the equation. This is usually done in a probabilistic computer program, using either the FORM-technique (first order reliability method) or the Monte Carlo technique. In the latter case, one has to know the distribution of H_s. These methods are worked out in an example in Appendix 1.5.

7.4. SPECIAL SUBJECTS

7.4.1. SHAPE OF QUARRY STONE

Hudson had already indicated, by varying values of K_D, that the angularity of quarry stone has an influence on stability. LATHAM ET AL. [1988] investigated the influence of the shape of individual stones on their stability. They used designations like "fresh", "equant", "semi-round", "very round", and "tabular". For a visual impression of block shapes, one is referred to Figure 4-26. As compared with "standard" quarry stone, the coefficient in the (deep water) Van der Meer formula changes slightly as shown in Table 7.5.

Similar investigations were carried out by BURGER [1995]. In his master's thesis, he indicates a relationship between stability and the l/d ratio of the quarry material.

7.4.2. GRADING OF QUARRY STONE

When quarry stone is purchased from a commercial block-stone quarry, gradation is usually according to national standards. For Western Europe, one is referred to data in the Rock Manual (2007). Examples of gradings are given in section 6.5.1. Converting mass into diameter is done by using the well-known d_n (nominal

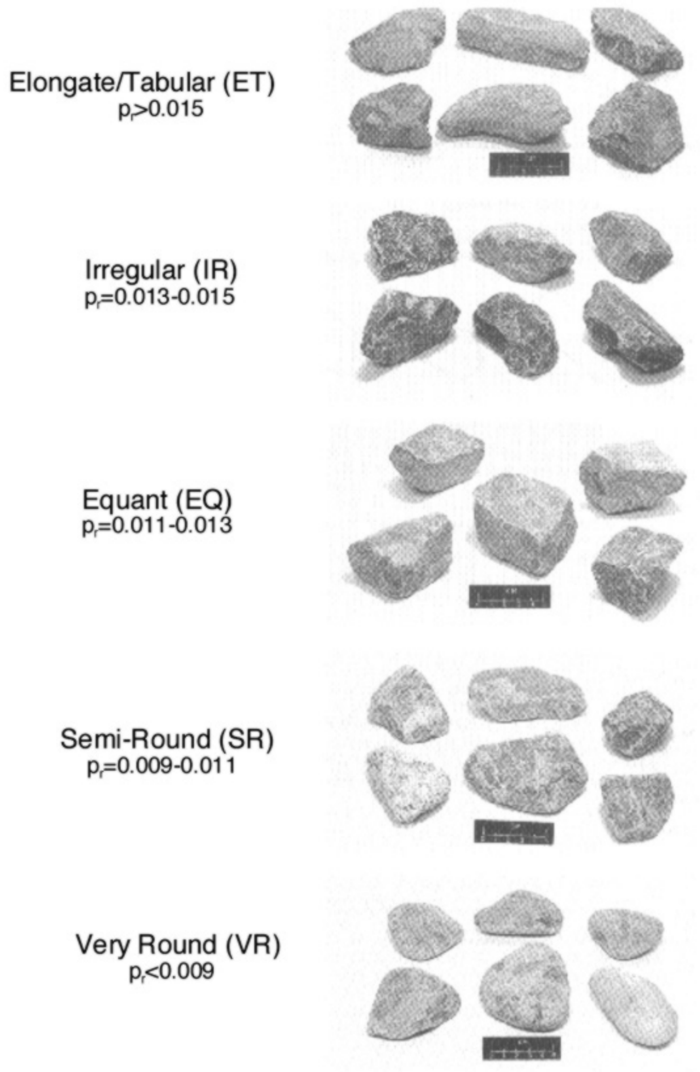


Figure 7.7: Visual comparison of block shapes (from Rock Manual, 2007)

Rock shape	Plunging waves	Surging waves
Elongate/Tabular	6.59	1.28
Irregular	6.38	1.16
Equant	6.24	1.08
Standard v.d. Meer	*6.2*	*1.0*
Semi-round	6.10	1.00
Very round	5.75	0.80

Table 7.5: Effect of stone shape on stability

diameter) method:

$$d_n = \sqrt[3]{\frac{M}{\rho}} \tag{7.31}$$

If the stone is classified according to sieve diameter, one can determine d_s. Although sieving is not a practical method for the larger stones, one can establish a general relation:

$$d_n = 0.8 d_s \tag{7.32}$$

The grading of a stone class is often defined as d_{85}/d_{15}. Common values are:

Type of grading	d_{85}/d_{15}
Narrow	< 1.5
Wide	$1.5 - 2.5$
Very wide (quarry run or riprap)	$2.5 - 5$ and more

Stability is usually investigated for grades classified as wide. The use of very wide grades will result in slightly more damage than narrow and wide grades. However, the very wide grades can easily lead to demixing or segregation, so that it is difficult to effectively control the quality of stone delivered.

7.4.3. STABILITY OF THE TOE

It is certainly not necessary to extend the armour layer over the full water depth down to the seabed. At a water depth of about one wave height below still water level the effect of the wave action is limited and no heavy armour is needed any more. The armour layer should then be supported by a toe. Of course, some optimisation is possible. The higher the toe is placed, the less armour is needed. However, a higher toe means also larger stones in the toe, and consequently a more costly toe.

Gerding (VAN DER MEER, D'ANGREMOND AND GERDING [1995]) performed useful work on this subject for his master's thesis. He investigated the relation between the unit weight of toe elements, toe level, and damage (N_{od}). His findings were confirmed by the thesis work of L. DOCTERS VAN LEEUWEN [1996], who also varied the rock density ρ_r. This resulted in two equations for toes which are not too deep (only for $h_t/H_s < 2$):

$$\frac{H_s}{\Delta d_{n50}} = \left(0.24\frac{h_t}{d_{n50}} + 1.6\right) N_{od}^{0.15} \quad 3 < h_t/d_{n50} < 25 \tag{7.33}$$

$$\frac{H_s}{\Delta d_{n50}} = \left(6.2\frac{h_t}{h} + 2\right) N_{od}^{0.15} \quad 0.4 < h_t/h < 0.9 \tag{7.34}$$

[3]

Test results are given in Figure 4-28. Critical values for N_{od} are given in TABLE 7-6. These values apply for a standard toe, with a height of $2 - 3d$ and a width of $3 - 5d$. A definition sketch is given in Figure 7.8.

N_{od}	Character of damage
0.5	Start of Damage
1.0	Acceptable Damage
4.0	Failure

Table 7.6: Critical values for N_{od}

Unfortunately, test data show quite some scatter around the formulas mentioned above. Research is ongoing to clarify this. A promising approach (BAART [2008]) is to express the damage N_{od} as a function of the parameter $\hat{u}_b/\hat{u}_c$, in which $\hat{u}_b$ is the amplitude of the wave at the toe and $\hat{u}_c$ the critical downrush velocity of the stones.

[3] Originally Gerding did not use a constant (2) in his equation, however, Van de Meer reasoned that for a value of $h_t = 0$ such an equation should lead to a stability number of zero, and suggested to add a constant with a value 2. Additional tests by EBBENS [2008] showed that this constant indeed had a value 2.

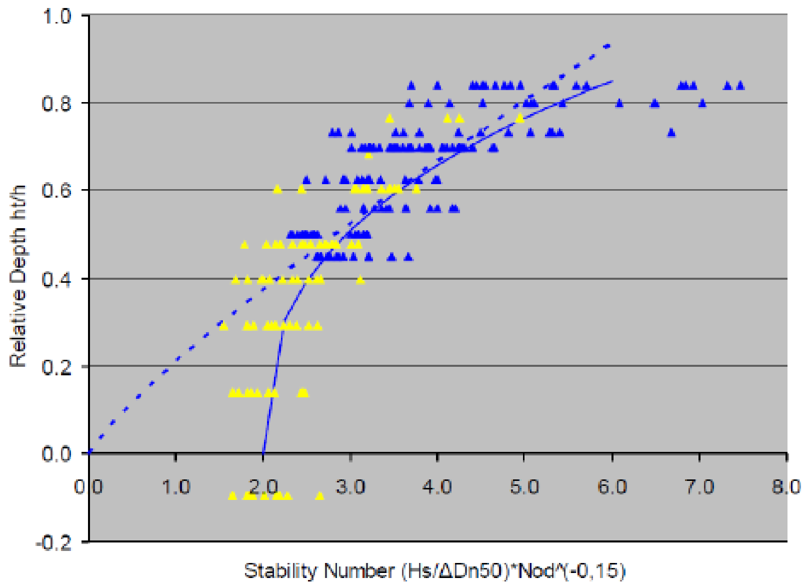


Figure 7.8: Toe stability according to equation 7.31

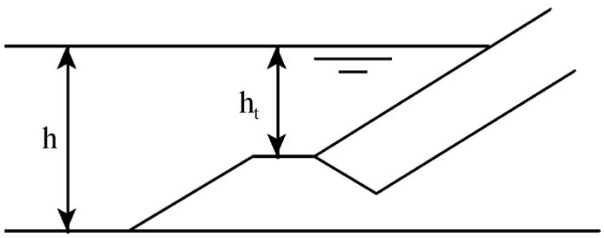


Figure 7.9: Definition sketch toe stability

7.4.4. Breakwater head

The head of a breakwater is relatively vulnerable since, due to the curvature, the armour units are less supported and/or less interlocking. In general, damage occurs on the inner quadrants, which is understandable if one looks at the 3D shape. There is no good theoretical way to determine the required Stability Number for a roundhead. Some guidelines for practical application are given in the Rock Manual (2007), but in general, a physical model test is recommended. Some practical guidance for cube roundheads is given by MACIÑEIRA and BURCHARTH [2008].

Therefore, the head of a breakwater is often reinforced either by:

- using larger size armour units;
- reducing the slope;
- increasing the density of the armour units.

Neither of the structural solutions is ideal: larger and heavier blocks pose construction problems and a reduced angle of slope may cause a hazard to navigation. As an alternative, one may use elements with the same size and placed with the same slope but using a higher density. This might be achieved by extra heavy aggregates, like Magnadense, a special aggregate made from iron ore which allows for the production of concrete with a ρ_s up to 4000 kg/m^3.

7.4.5. Stability of crest and rear armour

As long as the crest of the structure is high enough to prevent considerable overtopping, the armour units on the crest and the rear slope can be much smaller than the armour on the front slope. In this context, it must be mentioned that in the Netherlands the 2% run-up level is used for dikes with an inner slope consisting of grass covered clay.

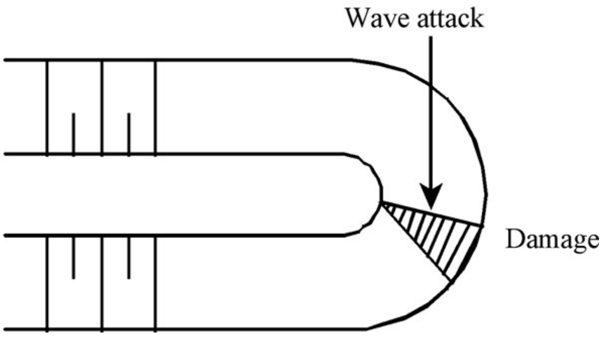


Figure 7.10: Typical damage pattern breakwater head

The size of stone on the inner slope of a high-crested breakwater is often determined by waves generated in the harbour basin by wind or passing ships. Only in the vicinity of the harbour entrance must waves penetrating through the entrance be taken into account.

In many cases, however, the designed crest height will allow for considerable overtopping during design conditions. In some cases, the crest will even be below still water level under such conditions. This means a reduction in the direct attack on the armour on the outer face, and at the same time, a more severe attack on the crest and the inner slope. In this way, there is a relation between the choice of crest level and the material on the crest and inner slope. The reduction of the attack on low crested breakwaters is discussed in Section 0.

In a limited study for a breakwater with a Tetrapod armour layer, DE JONG [1996] concludes that the worst condition for the lee armour exists when the freeboard, R_c, divided by the nominal diameter is between 0 and 1 ($0 < R_c/d_n < 1$).

Another student [BURGER, 1995] analysed data of Van der Meer and Vidal. He reached the conclusion that the inner slope is relatively safe when the dimensionless freeboard R_c/d_{n50} is larger than 4. When the crest is lower than this value, wave attack on the inner slope necessitates the use of relatively heavy blocks. When the crest is submerged equally (freeboard $< -4R_c/d_{n50}$), the wave attack is reduced so much that the damage decreases rapidly for all sections (crest, front slope and rear slope).

For the worst conditions (as defined by DE JONG) the blocks just around the water surface are most unstable. This is caused by the fact that the attack by the overtopping water is considerable, and the weight is reduced by the fact that these blocks are just submerged (VAN DIJK, 2001).

On the basis of a number of tests, VAN GENT AND POZUETA (2005) developed a stability formula for crest and inner slopes. For details, one is referred to the Rock Manual (2005).

7.4.6. STABILITY OF LOW AND SUBMERGED BREAKWATERS

Essentially two cases at any point in time can be discerned: a crest above still water level and a submerged crest. The European project Delos has lead to much understanding of low and submerged structures and gives detailed design guidelines (BURCHARTH ET AL, 2007). In the following section R_c is the crest height above still water level; so for submerged structures R_c is negative.

LOW CREST

VAN DER MEER (1990) derived a reduction factor for the armour size d_n. This reduction factor is:

$$\frac{1}{1.25 - 4.8R_p^*} \quad \text{for } 0 < R_p^* < 0.052 \tag{7.35}$$

With:

$$R_p^* = \frac{R_c}{H_s}\sqrt{\frac{s_{0p}}{2\pi}} \tag{7.36}$$

Application of this formula leads to a reduction in block size of up to 80% of the original value if the crest is at still water level. This is equivalent to a weight reduction of about 50%. The reason is that much wave energy passes over the breakwater and is not dissipated. The above equations can be used as a first approach.

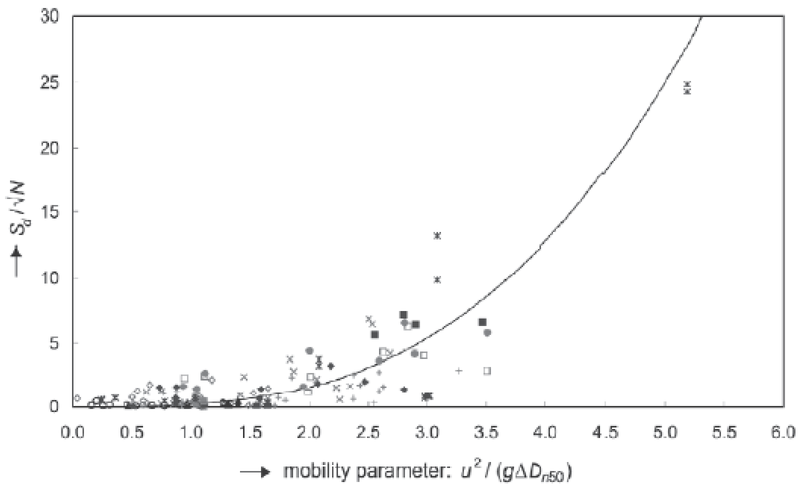


Figure 7.11: Illustration of the spreading around Equation (7.36) for the stability of near-bed structures

SUBMERGED CREST

When the crest is submerged, the wave attack is no longer concentrated on the slope, but rather on the crest itself. So, for submerged crest breakwaters, crest stability is the most relevant item. One should distinguish two types of submerged breakwaters, the stable ones, and the "reef" breakwaters, where the top may change. Both the Rock Manual (2007) and the Delos final report (2007) give overviews of many formula available for this structures. From the Delos project, it follows that the Stability Number of the crest mainly depends on the ratio of the submergence and the rock size:

$$\frac{H_s}{\Delta d_{n50}} = A + B\frac{R_c}{d_{n50}} + C\left[\frac{R_c}{d_{n50}}\right]^2 \tag{7.37}$$

Many researchers have provided values for A, B and C for all kind of conditions. A overview of these values can be found in the Rock Manual (2007). The differences are not very significant, so that as a rule of thumb one can use also the following equation:

$$d_{50} \geq 0.3d \quad \text{for } \frac{H_s}{h} = 0.6.\cot\alpha_s \geq 100 \text{ and } \Delta \approx 1.6 \tag{7.38}$$

in which:

d = height of the crest, measured from the bottom
h = water depth, at the toe of the structure
α_s = slope of the foreshore (in degrees)

7.5. NEAR BED STRUCTURES

Near bed structures are submerged structures where the crest is relatively low, such that wave action does not have a significant influence on the armour stability. Example applications of near bed structures are pipeline protections and intake/outfall structures near power and desalination plants. Hydraulic loads on near-bed structures include waves, currents and combinations of waves and currents. For the stability calculations, one usually follows a Izbash type of approach, in which the shear stress velocity is replaced by characteristic velocity (including both waves and currents) near the bed. Tests by WALLAST AND VAN GENT (2003) and LOMONACO (1994) showed that the effect of the current usually is negligible in relation to the effect of the wave.

Therefore the following stability relation is suggested:

$$\frac{u^2}{g\Delta d_{n50}} = \left(5\frac{S}{sqrtN}\right)^{1/3} \tag{7.39}$$

$$u = u_0 = \frac{\pi H_s}{T_m}\frac{1}{\sinh(-kR_c)} \tag{7.40}$$

in which:

R_c	= Crest height above waterline (in this case R_c is always negative)
S	= Damage level (as defined by equation (7.9))
N	= Number of waves

As can be seen from Figure 7.11 there is a large spreading of the results around this design line.

8

Breakwaters with a berm and berm breakwaters

There is a distinct difference between a breakwater with a berm and a berm breakwater. As the name says, a breakwater with a berm is a breakwater with a clear berm. The function of this berm is to reduce wave load on the main armour and to decrease overtopping over the breakwater. A berm breakwater is also a breakwater with a berm, but is designed in such a way that during storms the berm may deform into a more stable profile. These breakwaters are also called dynamically stable breakwaters. Apart from these two groups there is also the Icelandic breakwater. This is a breakwater with a berm, allowing only little deformation, but consisting of many different rock sizes. Although this seems not very practical for many projects, in some cases this may be a great advantage, because such a breakwater will use the available quarry material in the most optimal way. So remember: breakwaters with a berm are nearly all dynamically stable but not all berm breakwaters are dynamically stable.

8.1. Introduction

In Chapter 7, the stability of armour on a slope was studied on the condition that the units would be stable. In principle, no movements were permitted. We have seen that this requires the use of heavy armour. It is not always possible to obtain such heavy units of quarry stone because of geological limitations of the quarry. Casting the armour in concrete is complicated because very large units are rather sensitive to breaking.

Whether it would be possible to allow slight movements of the armourstone, so that the shape of the outer slope can adapt itself to the prevailing wave conditions, has therefore been studied. It is evident that a steep slope tends to become more gentle to stabilize itself against the battering waves. To maintain the overall function of the breakwater, including the required crest level, extra material must be provided. That is why this type of breakwater is often called a berm breakwater, since the extra material is placed in a berm on the outer side of the structure.

The application of extra material is only feasible when the cost of this material is not too high. This is the case when the quarry is not too far away from the construction site of the breakwater. Concrete units are never used in berm breakwaters, they are too costly and their sensitivity to abrasion is too high.

An additional advantage of this type of breakwaters is the fact that a wider gradation of material can be used. This avoids expensive sorting operations at the quarry. The wider gradation, and the limited maximum stone size, makes it easier for one to design a berm breakwater that encompasses the entire yield curve of the quarry. In the simplest form, the quarry yield is split into a maximum of three categories: filter material, core material and berm material.

Allowing the waves to reshape the outer slope eliminates the need to construct this slope at a specific angle. The contractor can dump the stone by truck and level it with a bulldozer, leaving the slope to assume the angle of internal repose. This again represents an important saving on construction costs. However, one must ascertain that a sufficient volume of material is used per running metre of cross section. Berm breakwaters may be divided into three types [PIANC/MARCOM 40, 2003]:

- Statically stable non-reshaping structures. In this condition only some stones are allowed to move similar to conventional rubble mound breakwaters.

- Statically stable reshaped structures. In this condition the profile is allowed to reshape into a profile, which is stable and where the individual stones are also stable.
- Dynamically stable reshaped structures. In this condition the profile is reshaped into a stable profile, but the individual stones may move up and down the slope.

While a conventional rubble mound breakwater is required to be almost statically stable for the design wave conditions, berm breakwaters have different stability criteria. In the following sections the dynamically stable reshaped berm breakwater is discussed. At the end of this chapter, attention is paid to the statically stable non-reshaping berm breakwater, also called the Icelandic breakwater. In Table 8.1 both types of breakwaters are compared.

The dynamically stable reshaped berm breakwater consists of only two stone classes, stones and quarry run. The advantage of using several stone classes is that the lighter grade can be used in specified places inside the structure. Instead of the wide stone gradation of the dynamically stable breakwater, the statically stable non-reshaping berm breakwater has a narrow stone gradation. This leads to higher permeability and an increased ability of the structure to absorb the wave load.

Dynamically stable reshaped breakwaters	Statically stable non-reshaping breakwaters (Icelandic Breakwaters)
Two stone classes	Several stone classes
Homogeneous berm	Berm of graded layers
Wide stone gradation	Narrow stone gradation
Low permeability	High permeability
Reshaping structures	Non/reshaping structure
Allowed erosion < berm width	Allowed recession $2D_{n50}$
More voluminous	Less voluminous
No interlocking	Interlocking prescribed

Table 8.1: Comparison between dynamically stable reshaped breakwaters and statically stable non/reshaping breakwaters

Traditionally a breakwater design starts with the design wave height, which leads to a set of required rock sizes. When these rocks are to be purchased on the market, this is a good approach. However, when a dedicated quarry has to be opened for the breakwater, one may reverse the design process and design the breakwater according the output yield of the quarry.

The procedure to select the most appropriate type of breakwater can be described as follows [SIGURÐARSON ET AL, 2004]:

1. Is it economical to design a conventional rubble mound breakwater following the Van der Meer method? Check if all quarried material can be used in the project or sold to other projects.
2. Is it more economical to design a statically stable non-reshaping breakwater with the largest stone class similar to Van der Meer criteria with $H/\Delta d_{n50}$ up about 2.0? The demand for large stones is usually less in (2) than in (1) but the total demand of stones is larger. If there is a quarry available to dedicate to the project then 2) is often more economical, usually for design wave heights $H_s > 2-3$ m.
3. If large stones (according to the Van der Meer criteria) are not available, then use wider and more voluminous berm breakwaters (statically stable reshaped type).
4. If 1) to 3) are not possible options, then apply a still wider and more voluminous berm breakwater design of a dynamically stable structure. This could be a suitable structure for a trunk section connecting an island to the shore, but it is not suitable for a head section.

8.2. SEAWARD PROFILES OF DYNAMICALLY STABLE BUNDS

Model tests on slopes that are not statically stable have indicated that a typical profile is formed according to Figure 8.1.

The characteristic dimensions can be expressed in terms of wave parameters [VAN DER MEER, 1984]:

$$l_c = 0.041 H_z T_m \sqrt{\frac{g}{d_{n50}}} \tag{8.1}$$

$$l_s = l_r = 1.8 l_c \tag{8.2}$$

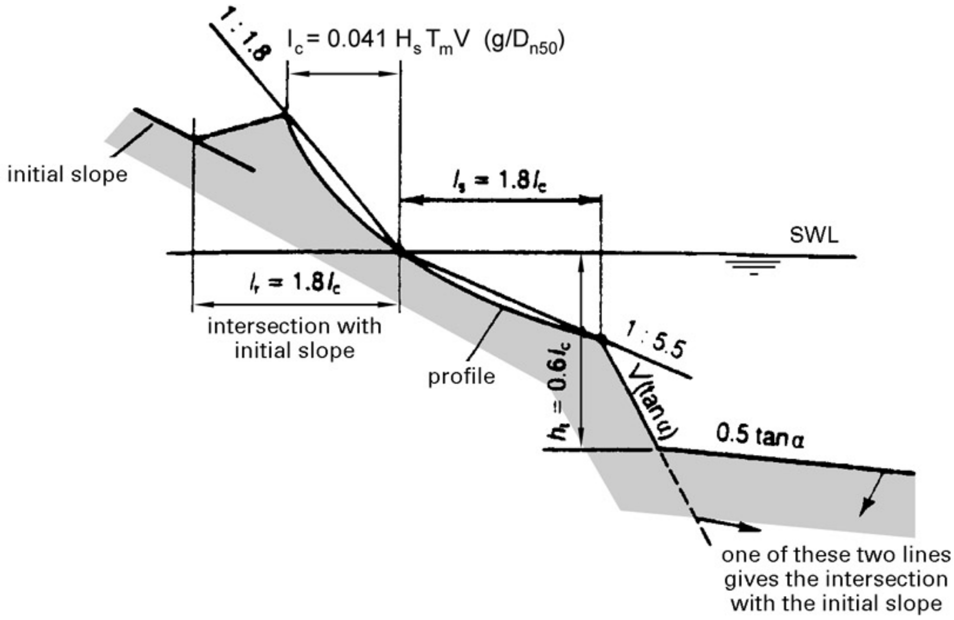


Figure 8.1: Schematised profile for sand gravel beaches

$$h_t = 0.6 l_c \tag{8.3}$$

As can be seen from Figure 8.1, the intersection point of the profile with the still water level determines the position of the newly formed slope. From this point, an upper slope is drawn at 1V:1.8H and a lower slope at 1V:5.5H. The horizontal distance l_c determines the position of the crest on the upper slope; the distance l_s determines a transition point on the lower slope. The actual slope in the zone of wave attack is a curved line through the three points. Below the lower transition point, a very steep slope develops at the angle of natural repose ϕ. If the original slope was already steep, the steep lower slope continues until the bottom is reached. If the original slope was gentle, the steep part continues until a level h_t below SWL. From the newly formed crest, the equilibrium profile connects to the original slope at a distance l_r (the run-up length). The position of the intersection point with SWL is not known in advance, but can be found easily when one realizes that the volume of erosion should be equal to the volume of accretion. An example of slope development based on the formulae is shown in Figure 8.2.

The designer of a berm breakwater can change the width and height of the berm by trial and error in such a way that the core is always protected by at least a double or triple layer of armour material. The trial and error work is made easier by the use of appropriate software like "BREAKWAT" of Deltares.

In principle, two types of initial cross-sections are used: one with a berm at crest level and one with a berm slightly above MSL. In the latter case, the chosen berm level is at such a height that trucks can safely drive over the berm. Practically this means that the recession length Rec should be less than the berm width B. See also Figure 8.3.

Usually the recession length is made dimensionless by dividing by d_{n50}. Apart from the above iteration procedure, several researchers have developed more direct approximation methods, using curve fitting. All these equations have the form:

$$\frac{Rec}{d_{n50}} = f(H_0 T_0) = f\left(\frac{H_s}{\Delta d_{n50}} \frac{T_m}{sqrt d_{n50}/g}\right) \tag{8.4}$$

Note: the notation $H_0 T_0$ may cause some confusion; both H_0 and T_0 are dimensionless parameters, H_0 is equal to the stability number. Because of this potential confusion, the parameters H_0 and T_0 will not be used elsewhere in this book. However, in literature on berm breakwaters, these parameters are quite common. One of the most commonly used curve fitting formulas is the formula developed by Tørum [PIANC-MARCOM 2003]:

$$\frac{Rec}{d_{n50}} = A(H_0 T_0)^3 + B(H_0 T_0)^3 C(H_0 T_0) - f(f_g) - f\left(\frac{h}{d_{n50}}\right) \tag{8.5}$$

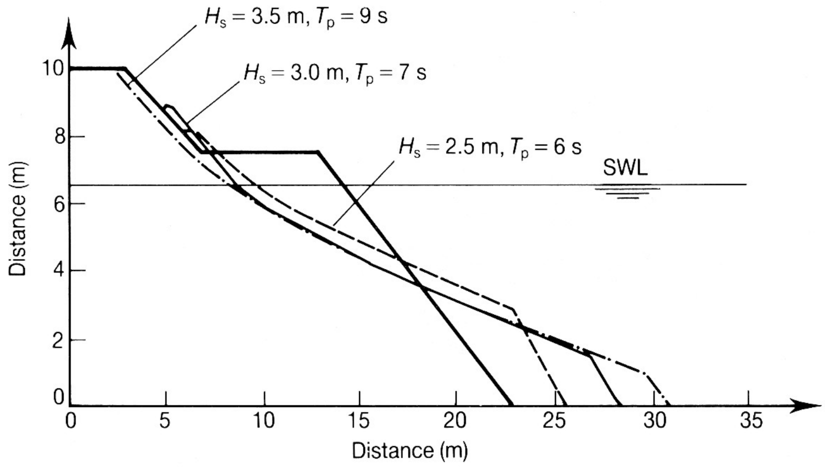


Figure 8.2: Influence of wave climate on a berm breakwater profile

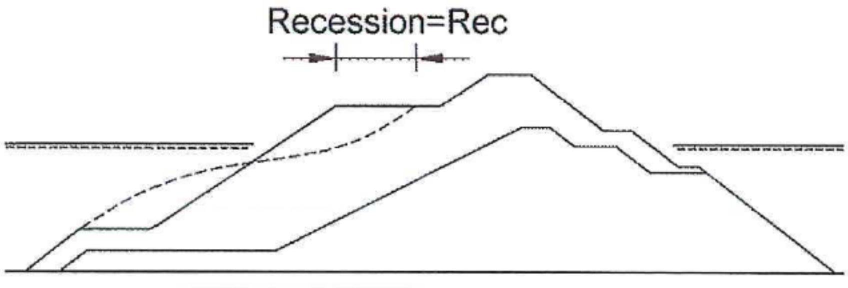


Figure 8.3: Recession on a reshaping breakwater (VAN DER MEER AND SIGURDARSON 2016

with:

$$f(f_g) = -9.9 f_g^2 + 23.9 f_g - 10.5$$

$$f\left(\frac{h}{d_{n50}}\right) = -0.16\left(\frac{h}{d_{n50}} + 4.0\right) \quad \left(12.5 < \frac{f}{d_{n50}} < 25\right)$$

$$f_g = \frac{d_{n85}}{d_{n15}} \quad (1.3 < f_g < 1.8)$$

$$A = 2.7 \cdot 10^{-6} \quad B = 9.0 \cdot 10^{-6} \quad C = 0.11$$

A similar formula with a somewhat better fit is given by Lykke Anderson [2007], but this formula contains even more fit parameters.

8.3. LONGSHORE TRANSPORT OF STONE

When a statically stable breakwater loses armour units from the cross-section, it is not relevant in which direction the units are moving, since they are already accounted for as damage. When armour units of a dynamically stable berm breakwater are moving out of their places, it is assumed that they find another, more stable position within the same cross-section. This assumption is not correct when the wave approaches the structure at an angle. In that case, the armour unit may be transported for some distance along the breakwater. Another unit, originating from a profile a little further upstream may fill its place. This process cannot continue indefinitely, since there will be a section that is eroded continuously so it is no longer possible to maintain an equilibrium profile.

This is why one should not accept considerable longshore transport along a berm breakwater. BURCHARTH AND FRIGAARD [1988] did some research on this and they concluded that longshore transport remains within reasonable limits if the armour size used for berm breakwaters is not too small. They recommend the following limits:

Trunks exposed to steep waves:

$$\frac{H_s}{\Delta d_{n50}} \leq 4.5 \tag{8.6}$$

Trunks exposed to oblique waves:

$$\frac{H_s}{\Delta d_{n50}} \leq 3.5 \tag{8.7}$$

Breakwater Heads:

$$\frac{H_s}{\Delta d_{n50}} \leq 3 \tag{8.8}$$

8.4. THE ICELANDIC BREAKWATER

A modification of the stable breakwater with a berm is the "Icelandic Breakwater." This type is aptly named as it was developed by Sigurdður Sigurðarson of the Icelandic Maritime Administration. It is composed of several size-graded layers, rather than just two, and the largest stone class is placed on the surface of the berm to reinforce the structure. The breakwater is designed to retain its integrity and only minor deformation of the berm is allowed under design conditions. Reshaping into an S-profile is not allowed but it is recognised that some deformation will occur with time as the result of repeated wave action. Well known examples of Icelandic breakwaters are the Sirevåg breakwater and the Melkøya breakwater; both projects are in Norway [SIGURÐARSON, 2004, 2005].

Preliminary design of the Icelandic breakwater is based on initial size distribution estimates from potential quarries, and the final design is tailored to fit the estimated yield curve obtained from a thorough investigation of the selected quarry. Quarry selection is a process which aims to provide rocks best suited to the wave conditions of the construction site and at the same time to minimise transport costs and environmental disturbance. It is therefore extremely important for the planning and economics of a successful breakwater project that information on the specific quarry is available at an early stage. One should first analyse a viable quarry and determine the potential yield curve for this quarry. Once a yield curve is determined, the cross section of the breakwater can be designed.

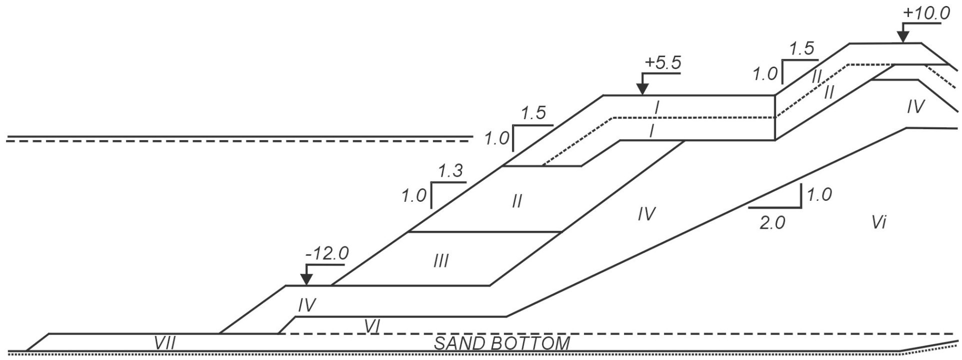


Figure 8.4: Multi-layer or non-homogeneous berm breakwater (Sirevåg in Norway) [PIANC-MARCOM, 2003]

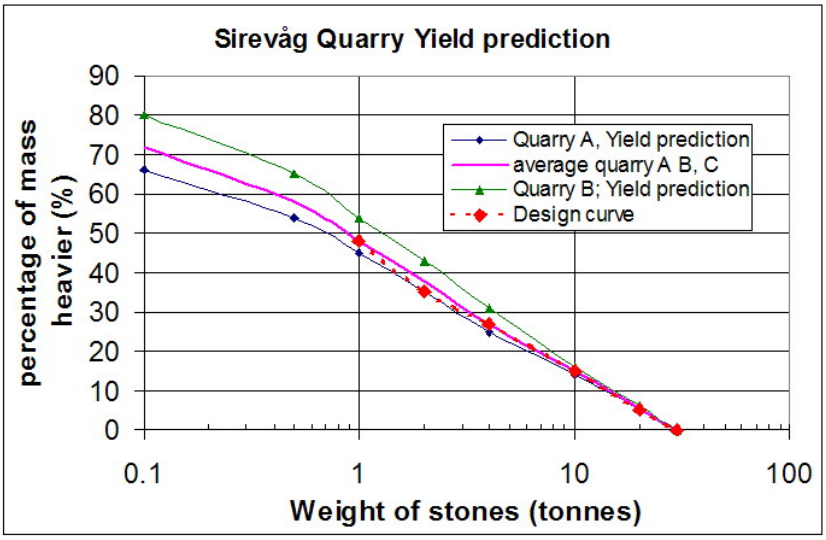


Figure 8.5: Quarry yield prediction curve and real production curve for Sirevåg breakwater [SIGURÐARSON, 2004]

By using appropriate blasting techniques (see Appendix 2.8) one may produce a relatively large percentage of big stones. A number of stone classes can then be defined and the amount of stone available in each class can be calculated. For example for the Sirevåg breakwater the quarry was able to provide the following classes:

Stone class	W_{min}-W_{max} (tonnes)	-W_{mean} (tonnes)	W_{max}/W_{min}	d_{max}/d_{min}	Expected quarry yield
I	20-30	23.3	1.5	1.14	5.6%
II	10-20	13.3	2.0	1.26	9.9%
III	4-10	6.0	2.5	1.36	13.7%
IV	1-4	2.0	4.0	1.59	19.3%
V	<1				52.5%

Table 8.2: Stone classes and quarry yield for the Sirevåg breakwater [SIGURÐARSON, 2004]

A cross section of the outer part of the Sirevåg breakwater is shown in Figure 8.4. The design fully utilises all quarried stones over 1 tonne and a 100% utilisation of all quarried material was expected for the project. For the Sirevag breakwater, a quarry yield prediction was carried out of three quarries for a 640,000 m^3 breakwater. The armourstone material is anorthosite gabbro rock of good quality (ρ_s= 2700 kg/m^3).

The quarry yield prediction, Figure 8.5, for a carefully worked quarry is about 50% over 1 tonne, about 30% over 3 tonnes and about 15% over 10 tonnes.

Research of Sveinbjörnson [2008] showed that the transition point between class I and class II (below the water level) stones should be at a depth of $h_{I-II} > 1.45\Delta d_{n,classI}$ and $h_{I-II} > \Delta d_{n,classII}$. The larger value of

Type		$H_s/\Delta D_{n50}$
Hardly reshaping Icelandic-type berm breakwater	HR-IC	1.7-2.0
Partly reshaping Icelandic-type berm breakwater	PR-IC	2.0-2.5
Partly reshaping mass-armoured berm breakwater	PR-MA	2.0-2.5
Fully reshaping mass-armoured berm breakwater	FR-MA	2.5-3.0

Table 8.3: Categorization of berm breakwaters (VAN DER MEER AND SIGURDARSON 2016)

these two should be used. Class II stones should reach down to at least the transition of the original and the expected reshaped profile, h_f.

8.5. VAN DER MEER AND SIGURDARSON 2016

In 2016, VAN DER MEER AND SIGURDARSON published a book on berm breakwaters that summarizes all research on the subject to date and provides new design guidance that partially supersedes the information given in the previous sections. The new edition of these lecture notes will be based on this book in particular, and the contents of the previous sections will be re-written. In the meantime, the following sections contain a summary of the main points from this book.

8.5.1. CATEGORIZATION OF RESHAPING BREAKWATER TYPES

In their book, VAN DER MEER AND SIGURDARSON divide berm breakwaters into two types:

- Icelandic breakwaters (IC) that consist of generally three classes of (very) large armour rock, and that are allowed to reshape only a little during design conditions
- Mass armoured (MA) breakwaters that consist of generally two classes of armour rock, smaller than for IC breakwaters, that are allowed to show significant reshaping during design conditions. Traditionally MA type berm breakwaters only had one single rock class as armour, but VAN DER MEER AND SIGURDARSON recommend to use two classes.

This gives rise to a total of four categories of berm breakwaters as given in table 8.3. These categories are defined by their stability number $H_s/\Delta D_{n50}$ where H_s is the design significant wave height, Δ is the relative submerged density of the armour rock material and D_{n50} is the median armour stone diameter.

This table forms the basis of the design method and can also be used to get a first impression of the feasibility of berm breakwaters. For $H_s/\Delta D_{n50} > 3$ the situation is no longer described as a berm breakwater but as a gravel beach, and the methods given in the previous sections (like the Breakwat method) must be used. For lower $H_s/\Delta D_{n50}$ numbers, the breakwater is a conventional non-reshaping rock armoured breakwater and the appropriate design methods (such as the Van der Meer formulas) of Chapter 7 must be used.

8.5.2. BERM RECESSION

Difference is made between design conditions (called 100 year conditions in the book but understood to be equally valid if the design return period has another value than 100 years) and overload conditions (taken in the book as 120% of the design conditions).

The most important parameter of a berm breakwater is its berm width. This must be larger than the recession (reshaping) during design and overload conditions. For design conditions the formula for berm recession is:

$$\frac{Rec}{D_{n50}} = 1.6\,(H_s/\Delta D_{n50} - 1.0)^{2.5} \tag{8.9}$$

This relatively formula takes the place of the more complex formulas described earlier, and the reshaping is now a function of the stability number $H_s/\Delta D_{n50}$ only instead of the period-dependent $H_o T_o$ number.

Van der Meer and Sigurdarson report that using equation 8.9 for overload conditions will overpredict the recession. Their practical advice is to calculate the recession using equation 8.9 for both design and overload conditions, but change the result for overload conditions to the value *halfway between* the two results. So if for example the recession from the formula would be 10 m in design conditions and 15 m in overload conditions, the end result would be *10 m in design conditions and 12.5 m in overload conditions.*

Hardly reshaping berm breakwaters HR			
lower slope	$\cot\alpha_d = 1.5$	$\cot\alpha_d < 1.5$	score: -
berm level	$d_b/H_{sD} \geq 0.6$	$d_b/H_{sD} < 0.6$	score: -
toe depth	h_t/H_{sD}	no influence (hardly any reshaping)	
A double negative overall score gives about $1D_{n50}$ more reshaping.			
Partly reshaping berm breakwaters PR			
lower slope	$\cot\alpha_d = 1.5$	$1.33 < \cot\alpha_d < 1.5$	score: -
		$\cot\alpha_d < 1.33$	score: - -
berm level	$d_b/H_{sD} \geq 0.6$	$d_b/H_{sD} < 0.6$	score: -
toe depth	$2.0 < h_t/H_{sD} \leq 2.5$	$h_t/H_{sD} > 2.5$	score: -
		$1.6 < h_t/H_{sD} \leq 2.0$	score: +
		$h_t/H_{sD} \leq 1.6$	score: ++
A double or triple negative overall score gives about $1.5D_{n50}$ more reshaping.			
A double positive overall score gives about $1D_{n50}$ less reshaping.			
Fully reshaping berm breakwaters FR			
lower slope	$\cot\alpha_d = 1.25/1.33$	$\cot\alpha_d < 1.25$	score: -
		$\cot\alpha_d > 1.33$	score: +
berm level	$d_b/H_{sD} \geq 0.6$	$d_b/H_{sD} < 0.6$	score: -
toe depth	$2.0 < h_t/H_{sD} \leq 2.5$	$h_t/H_{sD} > 2.5$	score: -
		$1.6 < h_t/H_{sD} \leq 2.0$	score: +
		$h_t/H_{sD} \leq 1.6$	score: ++
A double positive overall score, gives about $2D_{n50}$ less reshaping.			

Figure 8.6: Score table correction berm recession (VAN DER MEER AND SIGURDARSON 2016

This recession formula is valid for all berm breakwater types, but only if the cross section conforms to some 'standard' values in terms of berm level, slope angle etc. If the cross section deviates from the standard, the recession can be between 2 D_{n50} more or 1 D_{n50} less. For this purpose, Van der Meer and Sigurdarsson have established a score table in which the designer can look up the deviations from the standard and assign a '+', '++', '–' or '-' score to each deviation. Depending on the total number of plusses or minuses, the result from equation 8.9 can be adjusted, see Figure 8.6.

8.5.3. RESILIENCE AND SAFETY FACTOR

Van der Meer and Sigurdarson suggest to introduce a safety factor (which they call a resilience factor) over the calculated results for berm recession in design conditions. This value is a function of the berm breakwater type. They recommend the following:

- For hardly reshaping breakwaters (IC HR), let the recession in design conditions be 10-20% of the total berm length
- For partly reshaping breakwaters (IC PR or MA PR), let the recession in design conditions be 20-40% of the total berm length
- For fully reshaping breakwaters (MA FR), let the recession in design conditions be 70% of the total berm length

The final choice of berm length will depend on a careful consideration of the calculated recession in design conditions, the calculated recession in overload conditions, and the desired resilience factor. This choice is left to the designer.

8.5.4. OVERTOPPING

The overtopping over a berm breakwater can be calculated using the same formulae as for other rubble mound breakwater types (see chapter 10). The shape of the breakwater can be brought into account by in-

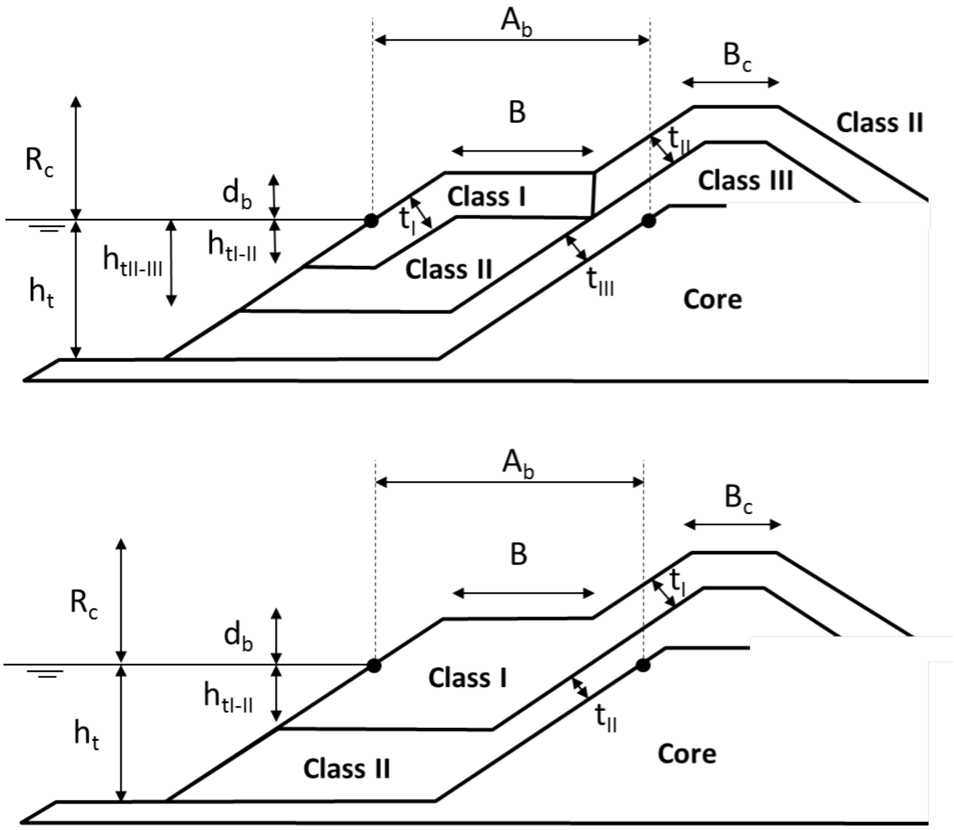


Figure 8.7: Principle sketch cross section IC (top) and MA breakwaters (bottom) and main dimensions (adapted from VAN DER MEER AND SIGURDARSON 2016

troducing an extra reduction factor γ_{BB} as follows:

$$\begin{aligned} \gamma_{BB} &= 0.68 - 4.5 s_{0p} - 0.05 B/H_{sD} \quad \text{for HR and PR} \\ \gamma_{BB} &= 0.70 - 9.0 s_{0p} \quad \text{for FR} \end{aligned} \tag{8.10}$$

This factor accounts for the reshaping of the berm during design conditions, so it could be argued that this formula is only valid for overtopping calculations during design conditions. For overtopping calculations during lower wave conditions (SLS limit states, for instance the 1 year conditions) it may be less suitable.

8.5.5. CROSS SECTION

The main difference in this context between IC and MA berm breakwaters is that IC breakwaters have three main rock classes and MA breakwaters have one or two. The typical cross sections and the main dimensions are sketched in figure 8.7. The berm width B can be calculated using the estimate of the recession and resilience described above. The required crest freeboard R_c follows from the overtoppning calculation. VAN DER MEER AND SIGURDARSON (2016) give recommendations with regards to the other dimenions; these are given below. These recommendations can be used for a first indication and can be seen as minimal requirements. The concept of a berm breakwater is that deviations from this typical sections are well possible, if this leads to a better optimization of the rock use. Also, the toe and rear slope details are not specified clearly and have to be designed like for a conventional structure, keeping the local conditions in mind.

Layer thickness The minimal layer thickness t for all rock classes is 2 times D_{n50} just like for a conventional structure.

Crest width The minimal crest width B_c is 3 times D_{n50} just like for a conventional structure.

Berm level The layer thickness of the class I material is 2 times $D_{n50,I}$. For practical constructability reasons it would be recommendable to let the top level of the undelying class II material be a safe margin (say 1 meter) above high tide level (or another reasonable operational water level) so it can be constructed safely with land-based equipment. This determines then the final berm level as 2 times $D_{n50,I}$ above this level.

Different berm levels can also be selected, based on practical engineering judgment by the designer. Some levels may have a small positive or negative influence on the recession length of the berm, see figure 8.6.

Total armour width / core position The stability and recession formulas for berm breakwaters assume implcitly that the pore sizes between the rocks in the berm are large, which has a positive influence on the stability (in a way that is broadly similar to the concept of notional permeability in conventional rock-armoured breakwaters). In order to ensure that this positive effect occurs, it is recommended that the total width between the outer slope and the start of the core (measured at the design water level) conforms to a certain minimum given by:

$$\frac{A_h}{H_s} = 2\frac{H_s}{\Delta D_{n50,I}} \tag{8.11}$$

Transition from Class I rock to Class II rock For this transition, VAN DER MEER AND SIGURDARSON give another practical recommendation and propose that this level is selected as:

- $h_{tI-II} = 0.4 H_{sD}$ below LW for IC-type breakwaters
- $h_{tI-II} = 0.6 H_{sD}$ below LW for MA-type breakwaters

where H_{sD} is the wave height in design conditions. The low water (LW) conditions to take into account is the lowest possible water level for which wave conditions can occur that are more or less equal to the design conditons (90% of H_{sD}). For the second transition $h_{tII-III}$ in the case of IC breakwaters there is no clear guidance.

Toe A berm breakwater may or may not have a toe structure at the seaward side, for instance in order to mitigate the risk of scouring. The level of the toe may have a small positive or negative influence on the berm recession, see figure 8.6. This toe structure can be built out of class II or class III rock, provided that the this material is stable at the depth of the toe. The design rules to check the toe stability are the same as for conventional structures, see chapter 7.

Slopes As a first indication, all slopes are typically 1:1.5. The seaward slope of the core can be set to a more gentle slope if class I/I/II material needs to the saved.

Core material The core material is not specified, this is left up to the designer and will depend on local circumstances like the expected quarry output and availability of materials. On all interfaces between the rock classes and the core material the same filter rules apply as for conventional structures.

Rock classes The selection of the rock classes I, II and III is the most important decision that the designer needs to make. In fact, the design process of a berm breakwater can be simplified to (1) selecting these rock classes, (2) calculate the minimum dimensions of the breakwater following from these rock classes, and (3) optimize by selecting other rock classes if necessary. This is where the skill and the experience of the designer becomes important. As a first indication, a selection can be made using the standard rock classes from the Rock Manual, or some non-standard classes that are often used in practice for berm breakwaters, as reported by VAN DER MEER AND SIGURDARSON (2016). An overview is given in table 8.4. The median mass M_{50} can be estimated as the average of the two class limits and the corresponding D_{n50} can be calculated using the defintion $D_{n50} = (M_{50}/\rho_s)^{1/3}$ where ρ_s is the rock mass density. As a starting point, a target value for $H_s/\Delta D_{n50}$ can be taken from table 8.3 as a function of the required breakwater category. This will give a first estimate of the required D_{n50}, and thus M_{50} for the largest rock class I. The next step is to select a rock class that roughly corresponds to this M_{50}. It is good practice to select a class II rock for which the upper limit is equal to the lower limit of the selected class I rock, and a class III rock for which the upper limit is the same as the lower limit of the selected class II rock. As an example, suppose that we want to make a PR-IC type breakwater for a design wave height $H_s = 6$ m, and suppose our rock density is $\rho_s = 2700 kg/m^3 (\Delta = 1.63)$.

Standard (Rock Manual)	Non-standard (examples)	
0.3-1 T	0.2-1 T	2-6 T
1-3 T	0.2-5 T	10-15 T
3-6 T	1-4 T	10-20 T
6-10 T	4-10 T	20-35 T

Table 8.4: Examples of rock classes (other choices are also possible)

We can select a target value $H_s/\Delta D_{n50} = 2.5$ from table 8.3, which gives us an initial guess of $D_{n50,I} = 1.47$ m and $M_{50,I} = 8.6$ T, so we take Class I = 6-10 T. Class II can then be for instance 2-6 T and Class III 0.5-2 T. From these initial guesses the dimensions of the breakwater can be calculated, after which the process of optimization can start. Please note that many other choices could have been made on the basis of the same starting points, so a designer has a lot of freedom here. (Many) other examples are given in VAN DER MEER AND SIGURDARSON (2016).

8.5.6. OPTIMIZATION OF QUARRY OUTPUT

Finally, after the initial rock classes have been selected and the dimensions of the breakwater have been determined, the required volumes of rock material (for each of the rock classes) can be calculated. This is the starting point for further optimization. In this context it is also important to predict the *quarry yield*, i.e. the volume of rock for each class that the quarry can produce. The predicted quarry yield will not only give an estimate of the largest rocks that can be produced, but also an estimate of the ratio of the produced volume for larger rock to the volume for smaller rock. For a given quarry, such a ratio will have a more or less fixed value, meaning that if a quantity X_1 of large rock is to be produced, a quantity X_2 of smaller rock will 'automatically' be produced as well. Any design that requires X_1 of large rock but does not use the full quantity X_2 of smaller rock will lead to losses. The trick of berm breakwater optimization is to compare the design volumes against the quarry yield and modify the design until they match as closely as possible. Generally speaking, three types of optimization are possible:

- Selecting different rock classes such that the total volume of the breakwater is reduced, for instance by selecting a larger class I rock so that the berm can be smaller etc. This is only possible if the quarry can produce sufficient quantities of the larger rock.
- Selecting different rock classes while keeping $M_{50,I}$ the same. For instance in our example, we could also have selected Class I 6-10 T, Class II 3-6 T and Class III 1-3 T. This probably does not change the overall volume too much, but it changes the volume distribution over the three rock classes. This new distribution might better match the distribution of the quarry yield, leading to smaller losses.
- Changing the overall design of the breakwater by deviating from the standard dimensions, for instance by modifying the rear slope or toe design, change the core material or core slope, et cetera. This will also give a (much) different distribution of rock volumes.

An example of how a quarry yield can be predicted, and how the design of a berm breakwater can be optimized, is given in appendix C.

8.5.7. CALCULATION EXAMPLES AND SPREADSHEET

Van der Meer and Sigurdarson complete their book with a large number of calculation examples for all berm breakwater types. These calculation examples are worked out using a standardized spreadsheet that is available for public use as well. The spreadsheet can be downloaded from the personal webpage of prof VAN DER MEER or from the course blackboard site.

The essence of the spreadsheet is that the designer chooses a set or rock classes and defines the dimensions of the cross section (berm width, crest level etc) and the spreadsheet then checks the cross section against the requirements. A rudimentary cross section is sketched by the spreadsheet which must be worked out into a more detailed design drawing by the designer.

9

STABILITY OF MONOLITHIC BREAKWATERS

The title of Chapter 9 may be a little misleading. Although it refers to monolithic breakwaters, in practice it deals mainly with vertical wall breakwaters or even caisson breakwaters. The subject is so full of uncertainties that it makes no sense at all to know formulae in a quantitative way. However, it is important to understand the difference between static, quasi-static and dynamic loads, and their effect on the stability.

9.1. INTRODUCTION

The problem of the stability of monolithic breakwaters has not yet been solved in a satisfactory and generally accepted way. Research efforts are under way, but have not resulted in a generally applicable theory or formula. Nevertheless, monolithic breakwaters are being built, and designers do use practical formulae. In this chapter, we will discuss a theoretical approach and a practical method developed in Japan. As the stability is a joint effect of wave load and subsoil resistance, some soil mechanics will be discussed as well. In addition to the stability of the monolithic breakwater, some other aspects of wave structure interaction will also be discussed. Because of the intense interest in many countries, rapid development of the knowledge of monolithic breakwaters must be expected, comparable with the evolution of rubble mound breakwaters between 1988 and 1993. For the reader this means that the most recent sources of literature must always be consulted in addition to this textbook.

9.2. WAVE FORCES AND THEIR EFFECTS

9.2.1. QUASI-STATIC FORCES

On the basis of linear wave theory, SAINFLOU [1928] developed a method to calculate pressures exerted on a vertical wall by non-breaking waves. RUNDGREN [1958] carried out a series of model experiments and concluded that Sainflou's method overestimates the wave force for steep waves. Rundgren then used and modified the higher order approach proposed by MICHE [1944]. This Miche-Rundgren method gives satisfactory results for steep waves, whereas the original Sainflou-method is best suited for long and less steep waves.

The main and most important aspect of the Miche-Rundgren approach is the definition of a parameter h_0, which is a measure for the asymmetry of the standing wave around SWL. This leads to the pressure diagrams shown schematically in Figure 9.1.

In this figure p_1 is given by:

$$p_1 = \left(\frac{1+K_r}{2}\right)\frac{\rho_w g H_i}{\cosh(2\pi h/L)} \tag{9.1}$$

and H_i is the (undisturbed) incoming wave height and K_r the reflection coefficient. Design manuals give graphs for the calculation of h_0 as a function of wave steepness, relative wave height (H/h) and reflection coefficient. They also give graphs to calculate integrated pressures and resulting turning moments for crest and trough of the wave. This leads to a relatively simple load diagram (Figure 9.2), in which the horizontal hydrostatic forces on the front and rear wall have been omitted because they eliminate each other. For stability, one must consider the resistance against translation and the resistance against rotation. *Here it is stressed*

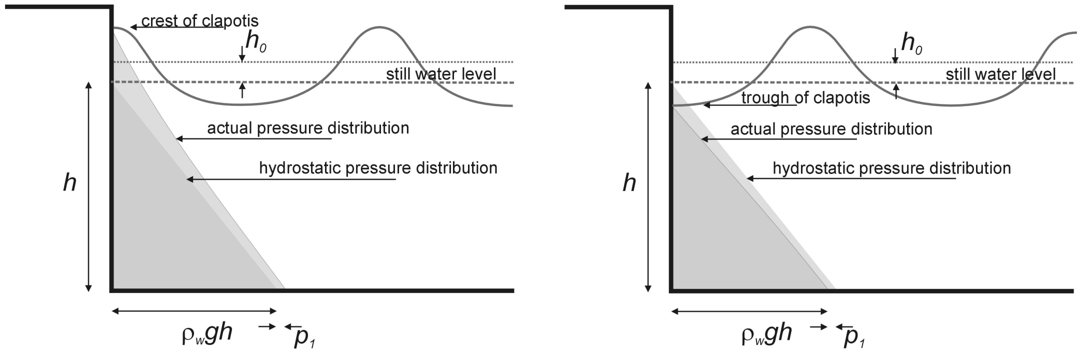


Figure 9.1: Schematic pressure distribution for non-breaking waves

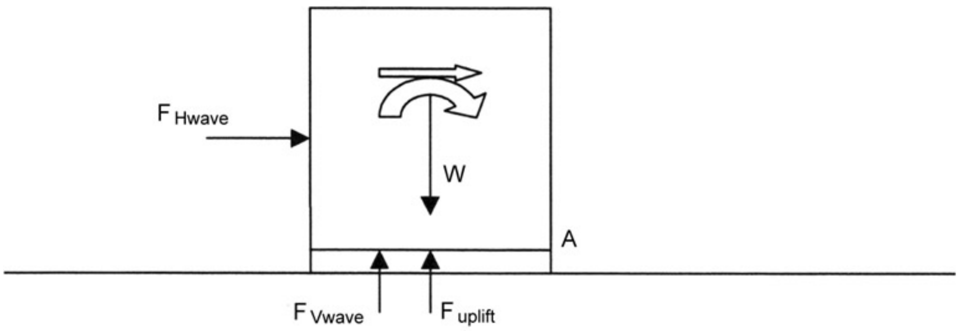


Figure 9.2: Load and equilibrium diagram

that the resistance against rotation cannot be taken simply as the sum of the moments around point A. Long before the structure starts to rotate, the pressure under point A has reached a value that leads to failure of the subsoil or failure of the corner of the structure.

Since these formulae have been derived for regular monochromatic waves, it is necessary to combine them with spectral theory and arrive at a statistical distribution of wave forces and overturning moments. It can then be decided what frequency of exceedance is acceptable during the lifetime of the structure. In this way, the design loads can be established.

The loads defined so far are called quasi-static forces, because they fluctuate with the wave period of several seconds and do not cause any dynamic effects. Inertial effects need not be taken into account.

9.2.2. DYNAMIC FORCES

In Section 0, we restricted ourselves to the forces exerted by non-breaking waves. However, when waves are breaking, we know that impact or shock pressures occur in the vicinity of the water surface. The duration of these pressures is very short, but the local magnitude is very large. The quasi-static pressures are always in the order of $\rho g H$, but the impact pressures can be 5 to 10 times higher. An example of a pressure record is given in Figure 9.3.

Many researchers have studied this phenomenon in the laboratory and none have come up with a satisfactory explanation that can predict the occurrence and the magnitude of a wave impact as a function of external parameters. BAGNOLD [1939] was the first of these researchers. He found that the impact pressure occurs at the moment that the vertical front face of the breaking wave hits the wall, and mainly when a plunging wave entraps a cushion of air against the wall.

Apparently, the deceleration of the mass of water in the wave crest, combined with the magnifying effect of the air cushion, causes the high pressures. Two models can be used to describe and calculate this effect:

- The continuous water jet hitting a plane yields a pressure: $p = (1/2)\rho u^2$ (u is the water velocity in the jet)
- The water hammer effect, resulting in: $p = \rho u c$ in which u = the water velocity in the conduct and c = the celerity of sound in water (1543 m/s)

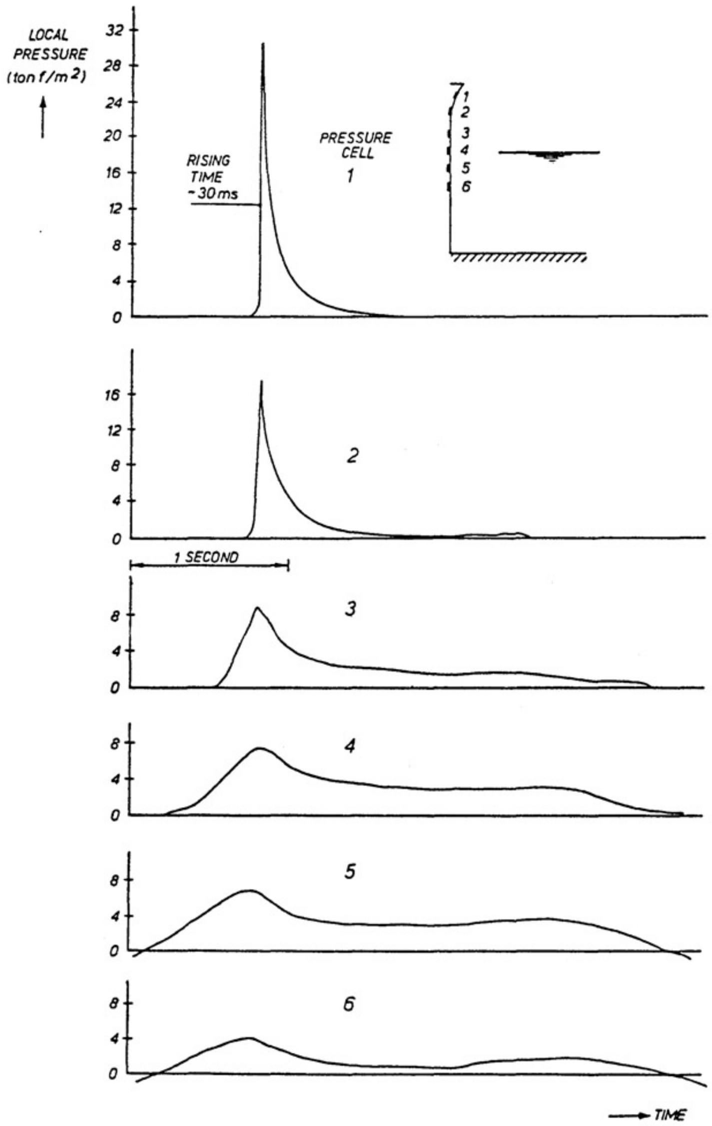


Figure 9.3: Example of a pressure record

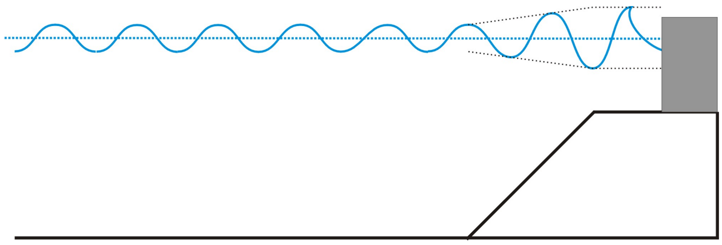


Figure 9.4: Changes to incoming wave front induced by high mound breakwater

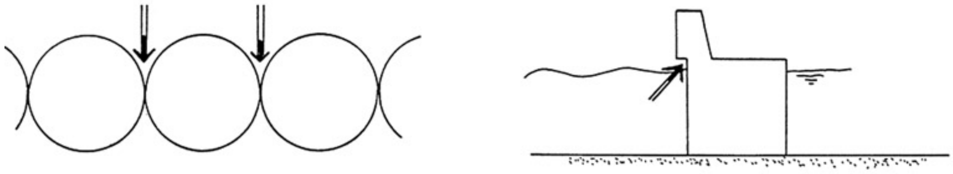


Figure 9.5: Risk of local impact forces

The water velocity in the crest of the breaking wave is equal to the wave celerity (in shallow water: $\sqrt{gh}$). Substitution of reasonable figures leads to a water velocity on the order of 10 m/s and impact pressures:

Continuous jet:	55 kPa
Water hammer:	16,000 kPa

In reality, we know that the impact pressures reach values between 500 and 1500 kPa. Measurement of the impact pressures in a model is complicated because the short duration of the load requires a very stiff measuring system to provide proper data. Moreover, the compressibility of the water (influenced by entrained air) is an important factor because it determines the celerity of the compression wave in water. Uncertainties about model conditions endanger scaling up to prototype figures. MINIKIN [1955 and 1963] has given a method to calculate wave impact pressures, but his method overestimates impact pressures and does not lead to satisfactory results.

From all experiments, however, it has become clear that the duration of the wave impact is short and the area where the impact takes place at the same time is small. This means that the wave impact forces cannot be used for a static equilibrium calculation. The dynamic effects must be taken into account, including mass and acceleration of the breakwater in conjunction with its elastic foundation and the added mass of water and soil around it. Preliminary analysis has shown that it is specifically the momentum connected with the breaking wave that determines the stability or loss of stability of the breakwater. Care must also been taken of potential resonance phenomena when the loading frequency coincides with the resonance frequency of the structure as a whole or of some individual members of the structure.

A sound method of design should establish a physical relation between the impact pressure, the hydraulic parameters and the structural parameters. On the basis of this, one should establish the exceedance curves of certain loads during the lifetime. Taking into account the response of the structure, one can then determine the probability of failure of the structure during its lifetime. Unfortunately, the physical description of wave impacts is insufficient to start this approach.

The most important lesson that can be learned from this section is the uncertainty that is connected with wave impact forces and their effects on the stability of monolithic breakwaters. It is therefore good engineering practice to try to avoid the exposure of monolithic breakwaters to breaking waves. In this context it is good to remember that even if no breaking waves are expected at the location of the breakwater, the breakwater cross-section itself may induce them, specifically when the monolith is place on a high mound of stone (see Figure 9.4).

It can also be concluded that the risk of local impact pressures increases for structural elements that entrap breaking waves. If water can escape sideways from the impact area, the pressures remain low (compare free jet), while if water cannot escape, the local pressures may become quite high (compare water hammer). In this way, certain features of monolithic breakwaters are relatively vulnerable (Figure 9.5).

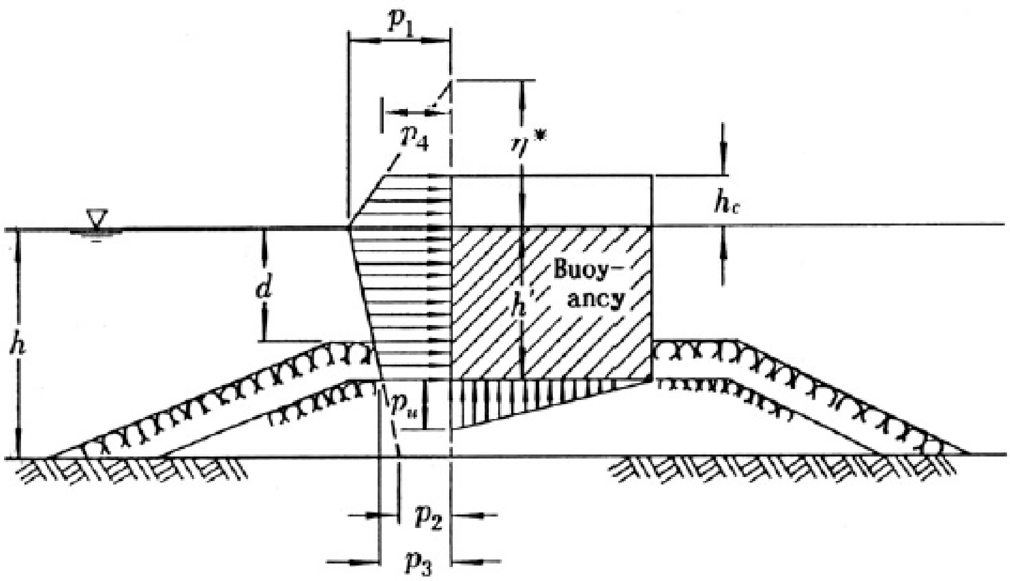


Figure 9.6: Definition sketch Goda method

9.2.3. A WORKING COMPROMISE: THE GODA FORMULA

While the uncertainties around the design of vertical breakwaters have reduced the number of such breakwaters in Europe and the USA, in Japan, construction has continued with varying success. Goda analysed many of the successful and unsuccessful structures and came up with a practical formula that can be used to analyse the stability of a monolithic breakwater. From a theoretical point of view, one can object that Goda is not consistent in his definition of design load and risk. In practice, the safety factors he proposes are apparently adequate, as long as one realizes that conditions with breaking waves should be avoided as much as possible. If this is not possible, extensive model investigations must be carried out, followed by a dynamic analysis of the structure and the foundation. In such cases one must take into account all inertial terms.

GODA [1992] has summarized his work in an article published for the short course on design and reliability of coastal structures. This article is added to this book as Appendix 4. Pending further theoretically based developments, the Goda formula can help to establish a preliminary idea about the stability of a monolithic breakwater. The Goda method is basically a deterministic method. For the design in a probabilistic way the same method can be followed as described in Appendix A1.4 for rubble mound structures. A detailed description is given in PIANC-MARCOM 28 [2003].

The basic elements of the Goda method are summarized below. For a more detailed overview see Appendix 4, or GODA [2000].

Design wave Goda defines a design wave height H_{max} as 1.8 times the significant wave height, or a maximum breaking wave height in case the caisson is located in the surf zone. Appendix 4 contains more details on how to calculate H_{max}. The corresponding wave period T is the significant wave period $T_{1/3}$.

Maximum elevation of wave pressure Goda defines a trapezoidal pressure figure on the seaward side of the caisson, see figure 9.6. The theoretical maximum level at which pressure is exerted is called η^* and can be calculated as:

$$\eta^* = 0.75\left(1+\cos\beta\right) H_{max} \tag{9.2}$$

where β is the angle of the incoming waves relative to the normal direction, i.e. $\beta = 0$ is the wave approach the caisson head-on. If the waves approach the caisson at an angle the wave pressures will be reduced. Goda (2000) recommends to always adjust the incoming wave angle by 15^o back towards the line normal to the breakwater, for safety in view of the uncertainty in estimating the incoming wave angle.

Wave pressures in front of the caisson The maximum wave pressure occurs at the design water level and can be calculated as:

$$p_1 = \frac{1}{2}\left(1+\cos\beta\right)\left(\alpha_1+\alpha_2\cos^2\beta\right)\rho g H_{max} \tag{9.3}$$

This pressure reduces linearly to a value at the bottom given by:

$$p_2 = \frac{p_1}{\cosh kh} \tag{9.4}$$

where $k = 2\pi/L$ and L is the local wave length based on $T_{1/3}$, at depth h. This wave length can be calculated from the dispersion relation at shallow water.

If the caisson does not extend all the way to the sea bottom but to a level h' below design water level then the pressure at the toe of the caisson is calculated by interpolation between p_1 and p_2:

$$p_3 = \alpha_3 p_1 \tag{9.5}$$

Above the design water level, the pressure figure theoretically reduces linearly to $p = 0$ at level η^*. If the crest level h_c of the caisson is lower than that level then this triangular figure is cut off and the pressure at the crest of the caisson can be calculated as:

$$\begin{aligned} p_4 &= p_1\left(1-\frac{h_c}{\eta^*}\right) && \text{if } \eta^* > h_c \\ &= 0 && \text{if } \eta^* \le h_c \end{aligned} \tag{9.6}$$

The values of the model coefficients are given by:

$$\begin{aligned} \alpha_1 &= 0.6+\frac{1}{2}\left(\frac{2kh}{\sinh 2kh}\right)^2 \\ \alpha_2 &= \min\left\{\frac{h_b-d}{3h_b}\left(\frac{H_{max}}{d}\right)^2, \frac{2d}{H_{max}}\right\} \\ \alpha_3 &= 1-\frac{h'}{h}\left(1-\frac{1}{\cosh kh}\right) \end{aligned} \tag{9.7}$$

Uplift The uplift pressure distribution is schematized as triangular with $p = 0$ at the heel of the structure and a value p_u at the toe that can be calculated as:

$$p_u = \frac{1}{2}\left(1+\cos\beta\right)\alpha_1\alpha_3\rho g H_{max} \tag{9.8}$$

Theoretically p_u should be equal to p_3 because they are evaluated at the same point, and water pressure is supposed to be isotropic. However, Goda proposes this slightly different value because it matches the datasets better. This shows again that this is an empirical design method.

Stability Now that the pressure figure around the caisson has been defined, the resulting horizontal force F_H and uplift force F_U can be calculated as:

$$F_H = \frac{1}{2}(p_1+p_3)h' + \frac{1}{2}(p_1+p_4)h_c^* \tag{9.9}$$

and

$$F_U = \frac{1}{2}p_u B \tag{9.10}$$

where B is the width of the caisson and $h_c^* = \min\{\eta^*, h_c\}$.

The resulting overturning moments around the heel of the structure are then:

$$M_H = \frac{1}{6}(2p_1+p_3)h'^2 + \frac{1}{2}(p_1+p_4)h'h_c^* + \frac{1}{6}(p_1+2p_4)h_c^{*2} \tag{9.11}$$

and

$$M_U = \frac{2}{3} F_U B \tag{9.12}$$

Finally, if the interior structure of the caisson and choice of ballast material is known, its mass M can be calculated and the safety factor (S.F.) against sliding and overturning can be calculated as:

$$\text{S.F. against sliding} = \frac{\mu(Mg - F_U)}{F_H} \tag{9.13}$$

$$\text{S.F. against overturning} = \frac{(Mgt - M_U)}{M_H} \tag{9.14}$$

where t is the horizontal distance between the center of gravity of the caisson and its heel ($t = B/2$ for a symmetrical caisson) and μ is the friction factor between the caisson and its foundation. In absence of further information $\mu = 0.6$ is a common estimate.

The required safety factors can be taken from local building codes or other standards, if available. Goda (2000) reports that in Japan it is requried that $S.F. \leq 1.2$ for both mechanisms.

Foundation load The caisson will exert an eccentric load on its foundation, which could lead to problems regarding the bearing capacity at the heel (the caisson can be 'pushed into the soil' even if it is not sliding or overturning. The bearing pressure at the heel can be analyzed using a formula given in Goda (2000):

$$\begin{aligned} p_e &= \frac{2W_e}{3t_e} && \text{if } t_e \leq B/3 \\ &= \frac{2W_e}{B}\left(2 - 3\frac{t_e}{B}\right) && \text{if } t_e > B/3 \end{aligned} \tag{9.15}$$

Goda (2000) recommends to design the caisson in such a way that the foundation pressure p_e remains below 400 to 500 kPa.

9.2.4. IMPULSIVE WAVE BREAKING

As discussed, the Goda method was developed as an empirical working compromise between breaking and non-breaking waves. The original recommendation by Goda (1992, see appendix 4) was to avoid constructing caisson breakwaters in impulsive wave breaking conditions as much as possible. These conditions to be avoided include steep foreshores, long waves and, in case the caisson is built on top of a rubble mound foundation, high rubble mounds. The width of the rubble mound also plays a role: if the width is very large or very small there is litte risk of impulsive wave breaking, but for some values in between the width can be 'just wrong'.

If the occurrence of impulsive wave breaking cannot be avoided, or if the situation needs to be examined more closely, the more recent work of Tanimoto and Takahashi can be used, see Goda (2000). In their work, the coefficient α_2 in equation 9.3 is replaced by $\max\{\alpha_2, \alpha_I\}$ where the new coefficient for impulsive breaking is defined as:

$$\alpha_I = \alpha_{IH}\alpha_{IB} \tag{9.16}$$

and the (sub)coefficients can be calculated as:

$$
\begin{aligned}
\alpha_{IH} &= \min\{H/d, 2.0\} \\
\alpha_{IB} &= \begin{cases} \cos\delta_2 / \cosh\delta_1 & \text{if } \delta_2 \le 0 \\ 1/(\cosh\delta_1 \cosh^{1/2}) & \text{if } \delta_2 > 0 \end{cases} \\
\delta_1 &= \begin{cases} 20\delta_{11} & \text{if } \delta_{11} \le 0 \\ 15\delta_{11} & \text{if } \delta_{11} > 0 \end{cases} \\
\delta_2 &= \begin{cases} 4.9\delta_{22} & \text{if } \delta_{22} \le 0 \\ 3.0\delta_{22} & \text{if } \delta_{22} > 0 \end{cases} \\
\delta_{11} &= 0.93\left(\frac{B_M}{L} - 0.12\right) + 0.36\left(0.4 - \frac{d}{h}\right) \\
\delta_{22} &= -0.36\left(\frac{B_M}{L} - 0.12\right) + 0.93\left(0.4 - \frac{d}{h}\right)
\end{aligned}
\tag{9.17}
$$

where B_M is the width of the berm in front of the caisson. If the resulting coefficient α_I is smaller than the original Goda coefficient α_2 from equation 9.7 then α_2 is to be used (interpreted as: there is no risk of impulsive wave breaking).

The risk of occurrence of impulsive wave breaking can be investigated by following the steps in figure 9.7, but these questions can be rather qualitative and non-conclusive. If the risk for impulsive wave breaking persists it may be recommendable to investigate other breakwater types, and/or investigate the situation using physical model tests.

9.3. INFLUENCING THE FORCES

It has been shown that the quasi-static forces and the dynamic forces have a tendency to translate and rotate the structure, resulting in the displacement of the structure and/or damage to the foundation and the bottom corners. The effect of the external forces can be reduced by changing the direction of the horizontal force or by spreading the force in space and in time. The first effect is easily understood if one realizes that the water pressure is always acting normal to a plane. When the front wall of the monolith is tilted, it means that the wave force is no longer horizontal, but directed towards the foundation. This reduces the horizontal component and strengthens the vertical component of the force. Altogether, the likelihood of sliding is reduced and the overturning moment is also reduced (Figure 9.8).

Another method involves the creation of a chamber in front or on top of the structure, so that the point of application of the force is spread over two walls, and a time lapse is created between the two forces. This reduces the maximum instantaneous force, although the duration is prolonged. JARLAN [1961] first applied such idea (Figure 9.9), partly to reduce forces and partly to reduce the reflection. In Japan, a large number of similar ideas have been developed and brought into practice. In a number of cases, the idea is combined with power generation (Figure 9.10). Many of the designs have been described by TAKAHASHI [2002]. Two examples of their typical designs are given in Figure 9.11 and in Figure 9.12.

Many other designers followed the ideas of Jarlan. One of these designs is given in Figure 9.13. However, the main problem of this type is that the efficiency in damping wave energy is quite sensitive to variations in the wave period. This means that such a system could damp waves quite effective for a given wave period, but when the period is only a few percent longer, the damping is considerably less.

9.4. FAILURE MECHANISMS

When designing a vertical wall breakwater, one has to check all failure mechanisms. The main failure mechanisms for vertical wall breakwaters are:

- Stability against sliding
- Stability against overturning
- Structural integrity of the caisson against wave impacts
- Slip failure stability of rubble foundation on the subsoil (sand or clay)
- Hydraulic stability of the toe berm rock armour
- Wave induced scour in front of the breakwater and the roundheads

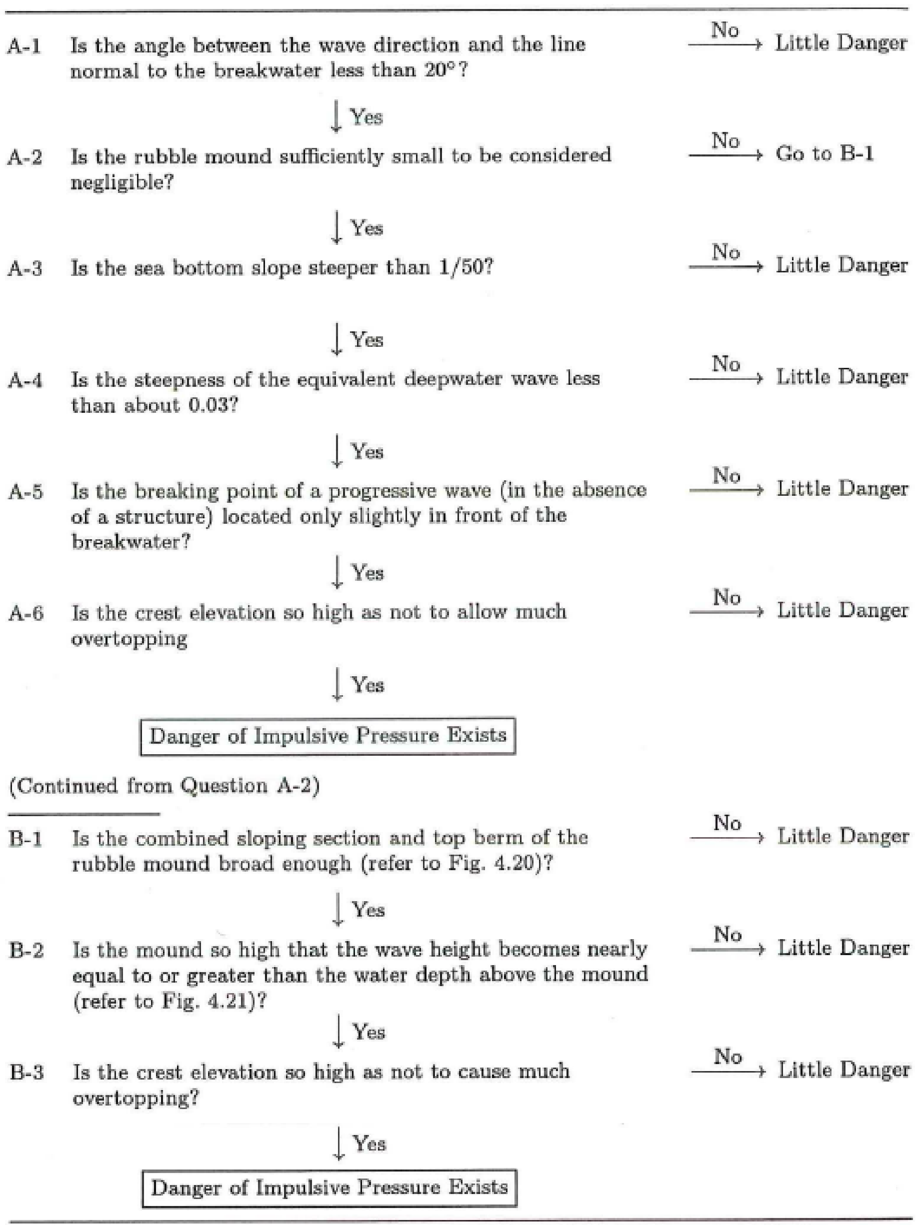


Figure 9.7: Flow chart for determination of risk of impulsive breaking (Goda 2000)

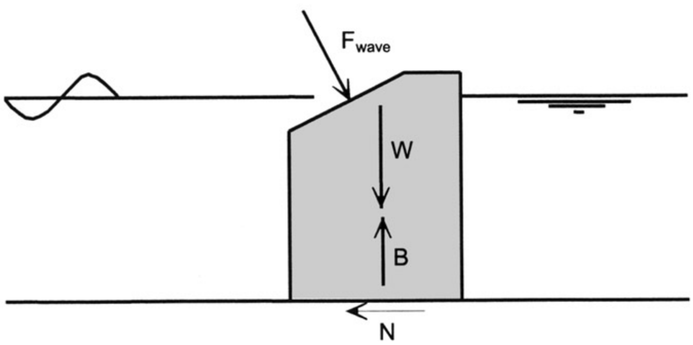


Figure 9.8: Hanstholm caisson

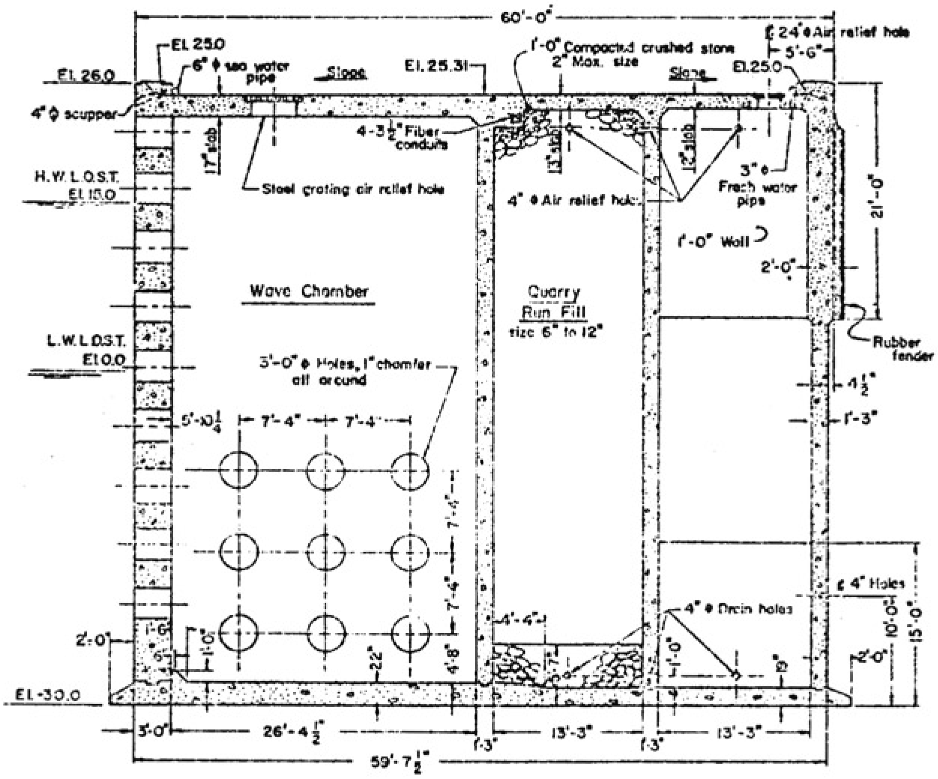


Figure 9.9: Jarlan caisson

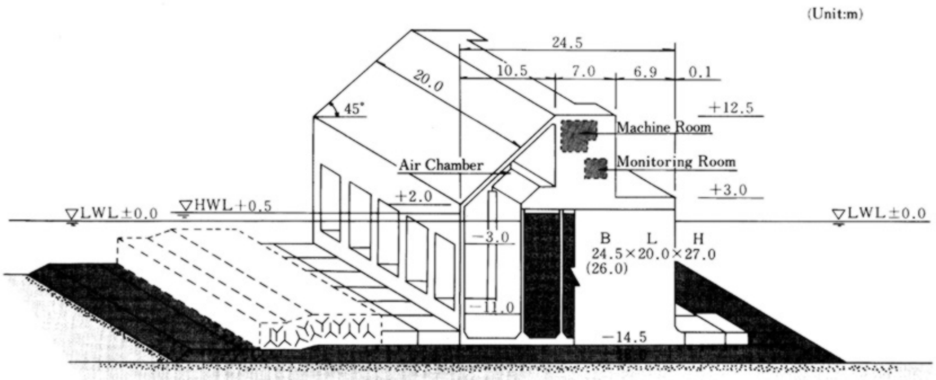


Figure 9.10: Breakwater with a wave power generating system

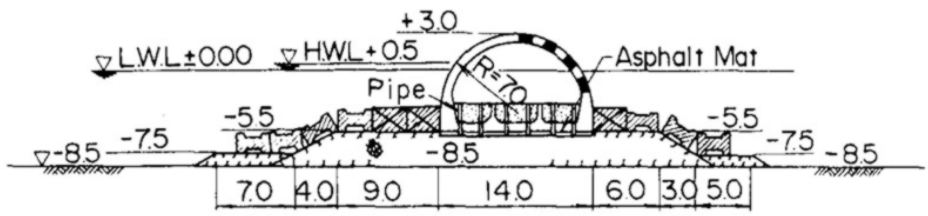


Figure 9.11: Possible cross-section of semi-circular caisson breakwater for extremely high breakers

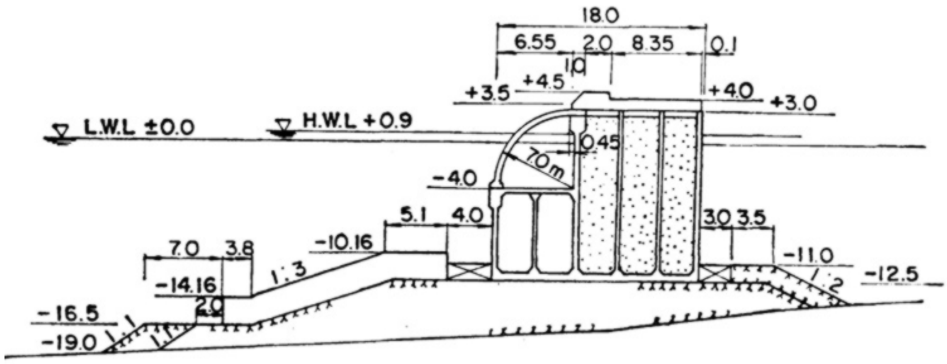


Figure 9.12: Cross section of curved-slit breakwater at Funakawa Port

Figure 9.13: Honey wall breakwater

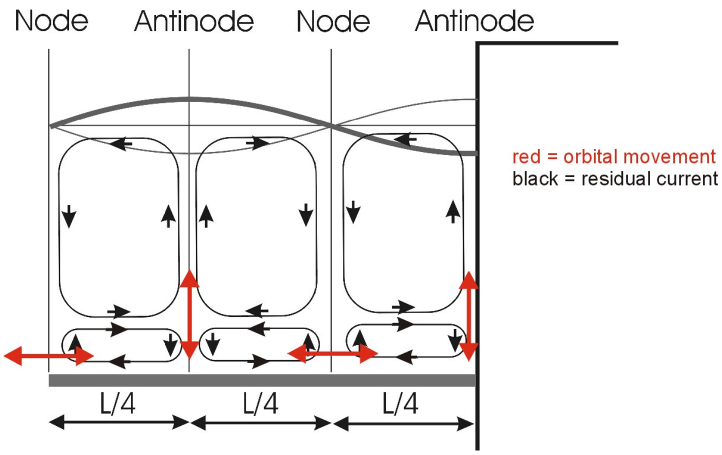


Figure 9.14: Steady streaming in the vertical plane in front of a breakwater

- In case of composite breakwaters: the stability and structural integrity of the armour units in the berm in front of the wall

In principle also the structural integrity of the caisson against static hydraulic loads has to be checked, but usually this is not a problem when the other conditions are fulfilled.

9.5. BERM STABILITY

If the caisson is placed on top of a rubble mound foundation, the stone size used for the armour of the foundation must be large enough to withstand the wave forces. A preliminary design for these blocks can be made using the Hudson-type formula proposed by Tanimoto (1982):

$$\frac{H_{1/3}}{\Delta D_{n50}} = N_s \tag{9.18}$$

where the stability number N_s is calculated as:

$$N_s = \max\left\{1.8, \left(1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left[-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right]\right)\right\} \tag{9.19}$$

in which:

$$\kappa = \frac{2k'h'}{\sinh 2k'h'}\sin^2(k'B_M)$$

and $k = 2\pi/L'$ where the local wave length L' is evaluated at the water depth h'.

9.6. SCOUR

Due to the standing wave or clapotis in front of a vertical wall breakwater, the orbital velocities just above the seabed become quite high at the location of the nodes ($(1/4)L \pm (1/2)L$) in front of the structure. This can lead to unwanted scour.

The standing wave generates a field of steady streaming, a system of recirculation cells (consisting of bottom and top cells), as illustrated in Figure 9.14 [MEI, 1989]. The sediment on the bed will respond to these circulations. When the bed consists of course material, the lower cell is most important and the sediment will be transported to the nodes. When the sediment is fine, it will reach somewhat higher in the vertical and the circulation on the border between the two cells is relevant. Then, the material is brought to the antinodes.

This means that for fine material (suspension mode) accretion can be found near the antinodes and for course material (non-suspension mode) accretion can be found near the nodes. See also Figure 9.15, based on XIE [1981]. In view of the uncertainties, it is recommended that the seabed in front of the wall should be protected over the length that is essential to ensure soil mechanical stability, with a minimum of 10 to 15 m.

In Japan it is quite common to place heavy toe blocks in front of the caisson to prevent scouring. TAKAHASHI [2002] presents some practical rules to calculate the size and weight of such blocks.

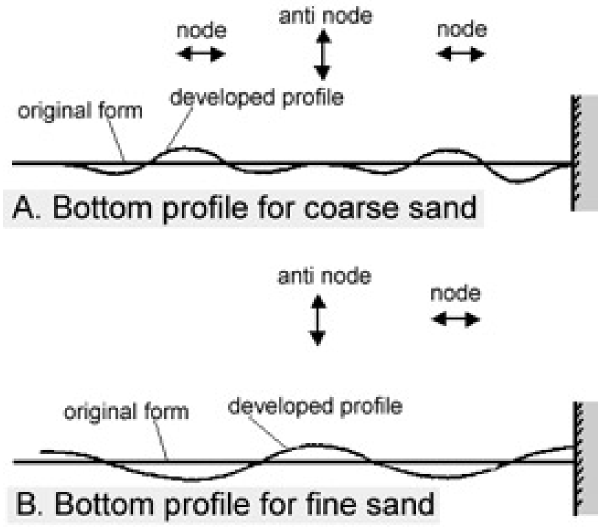


Figure 9.15: Erosion pattern depending on the grain size

9.7. FOUNDATION

The hydraulic forces exerted on the caisson, plus the weight, determine what the local pressures in the interface between the caisson and the foundation will be. It will be clear that these pressures must not lead to soil mechanical failure. Because the foundation is flexible to a certain extent, it is necessary to verify whether the mass-spring system formed by caisson (mass) and foundation (spring) gives rise to resonance phenomena. Depending on the outcome of this investigation, one may decide that a static stability analysis is sufficient (as is often the case). Soil-mechanical failure is nevertheless one of the most likely failure modes. Even if, after the analysis, it is decided a quasi-static approach is justified, the cyclic effect of the load may not be overlooked. In any case, the load will cause an increase in the total stress level (σ) and initiate compression of the subsoil. At first, this will lead to a higher stress level in the ground water (p). Depending on the permeability of the soil, the excess water will drain and gradually the effective stress (σ') will increase. This is all in accordance with one of the basic laws from soil mechanics:

$$\sigma = p + \sigma' \tag{9.20}$$

Because of the cyclic character of the load, it is possible that drainage of excess water is not complete when the next loading cycle starts. In this way, the water pressure may gradually increase due to rocking of the caisson. Eventually, this will lead to a condition in which the effective stress σ' becomes very low or even zero. A low effective stress will greatly reduce the resistance against sliding, while an effective stress equal to zero leads to liquefaction or the formation of quicksand. This is the main reason that care is recommended when designing monolithic breakwaters in areas that are sensitive to liquefaction: soil consisting of fine, loosely packed sand as in the SW part of the Netherlands.

It is possible, but expensive to use preventive methods against liquefaction. Soil replacement and compaction of the subsoil are the most commonly used methods. Widening the base of the caisson is also an effective measure.

Because of the possibility that high ground water pressures may occur under the corners of the monolith, large vertical gradients are also likely. It is therefore necessary to cover a fine-grained subsoil with an adequate filter. Because of the large gradients, it is recommended that the filter be designed as a geometrically impervious filter. Filter rules have been treated extensively by Terzaghi. The theory has also been covered in other textbooks [e.g. SCHIERECK 2001].

A granular foundation layer may also be required if the structure is placed on an uneven hard seabed. In this case, it is the function of the foundation layer to flatten the seabed and to avoid pressure concentrations and an unpredictable support pattern for the structure. Alternatively, one may create pre-designed contact areas in the bottom of the structure, so that the bending moments in the floor plate can be calculated.

To create a perfect homogenous contact plane between the foundation and the structure, a grout mortar is sometimes injected. This technique has been developed in the offshore industry for the foundations of gravity platforms, but the use has spread to other coastal engineering projects. To avoid loss of grout, a skirt is provided along the circumference of the bottom of the caisson. This skirt (usually a steel sheet) penetrates into the foundation and creates a chamber that can be filled with the grout mortar.

10

WAVE-STRUCTURE INTERACTION

There is a strong interaction between a wave and a wave damping structure, such as a breakwater. This interaction is visible in front of the structure (reflection), on the slope of the structure (run-up) and behind the structure (overtopping and transmission). This chapter summarizes these interactions.

10.1. INTRODUCTION

Even if a breakwater structure is stable under the action of waves, there is an interaction between the structure and the adjacent wave field. We discern various phenomena that lead to different wave patterns in the vicinity of the structure:

- Wave reflection
- Wave run-up
- Overtopping
- Wave transmission

Before using any of the expressions given in this chapter, it is useful to analyse which phenomenon influences the design problem in question. Wave reflection influences the area in front of the structure, wave run-up takes place on the slope of the structure, and overtopping and transmission are important for the area behind the structure. Overtopping focuses on the effect of one single wave on the inner slope of the breakwater, while transmission focuses on the amount of waves which may pass the breakwater. The latter is relevant for the tranquillity in the basin, while the first is relevant when designing the inner slope. Also, it is useful to categorize the phenomena to assess if one of the functions can be allowed on the inner side of the breakwater (i.e. if it can be used as a quay wall). Too often, formulae for run-up or overtopping are used when the designer wishes to address wave transmission.

10.2. REFLECTION

The wave motion in front of a reflecting structure is mainly determined by the reflection coefficient $K_r = H_r/H_i$. (See Figure 10.1.) If 100% of the incoming wave energy is reflected, one can safely assume that the reflection coefficient $K_r = 1$. This is generally valid for a rigid vertical wall of infinite height. The reflection coefficient for sloping structures, rough or permeable structures, and structures with a limited crest level is smaller, $K_r < 1$. In the European research programme CLASH [ZANUTTIGH AND VAN DER MEER, 2007], a new formula has been developed for application both on smooth and rough slopes.

The general shape of the reflection formula for straight slopes is:

$$K_r = \tanh\left(a\xi_{m-1,0}^{b}\right) \tag{10.1}$$

in which:

$$a = 0.167\left[1 - \exp\left(-3.2\gamma_f\right)\right]$$

$$b = 1.49\left(\gamma_f - 0.38\right)^2 + 0.86$$

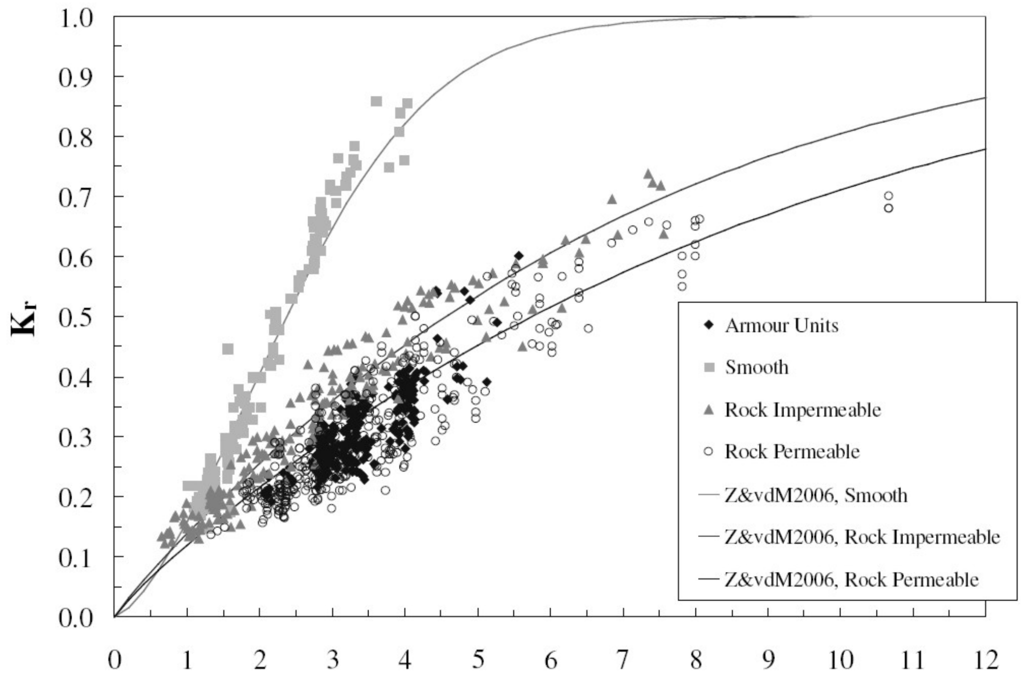


Figure 10.1: Reflection data for straight slopes [ZANUTIGH AND VAN DER MEER, 2007]

For composite slopes the coefficients a and b have different values. The value γ_f is the roughness coefficient. For smooth slopes, $\gamma_f = 1$. For slopes with some roughness, this value may decrease to values in the order of 0.5. In Table 10.1 an overview is given of the values a, b and γ_f for various types of structures with straight slopes.

Structure	a	b	γ_f
Rock, permeable	0.12	0.87	0.40
Rock, impermeable	0.14	0.90	0.55
Smooth slopes	0.16	1.43	1.0
Tetrapods, 2 layers	0.102	0.87	0.38
Core-Loc, 1 layer	0.113	0.87	0.44
Xbloc, 1 layer	0.112	0.87	0.45
Accropode, 1 layer	0.115	0.87	0.46
Antifer, 2 layers	0.115	0.87	0.47
Cubes, 2 layers	0.108	0.87	0.47
Cubes, 1 layer	0.120	0.87	0.50

Table 10.1: Correction factors for roughness

For composite slopes, the same approach can be followed but a correction for the slope angle has to be included. Details are given by ZANUTTIGH AND VAN DER MEER, [2007].

10.3. RUN-UP

10.3.1. STANDARD CASE: SMOOTH, IMPERMEABLE

Wave run-up is the phenomenon in which an incoming wave crest runs up along the slope up to a level that may be higher than the original wave crest. The vertical distance between still water level SWL and the highest point reached by the wave tongue is called the run-up z or R_u. From this definition, it is clear that we can only speak of run-up when the crest level of the structure is higher than the highest level of the run-up (Figure 10.2). Run-up figures are mainly used to determine the probability that certain elements of the structure will be reached by the waves. Run-up can be indirectly used to estimate the risk of damage to the inner slope of the structure.

In the Netherlands, research on run-up has always attracted a lot of attention [BATTJES, 1974]. This was

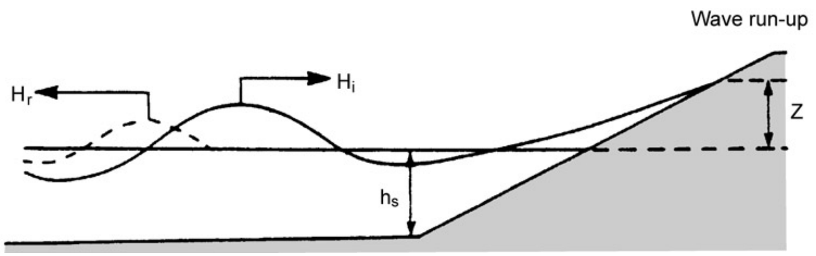


Figure 10.2: Definition of wave run-up

mainly because of the need to assess the required crest level for dikes and sea walls. Since most of the research effort was directed to run-up on dikes with a slope protection of asphalt or stone revetment, most results are valid for smooth, impermeable cover layers on the seaward slope.

Since the inner slopes of many dikes in this country are often covered with grass, it is not acceptable for a large percentage of the incoming waves to reach the crest and subsequently cause damage to the inner slope. Therefore, in most cases, the 2% run-up is required: the run-up level that is exceeded by 2% of the incoming waves. It is assumed that the grass on the inner slope of a sea dike can withstand this condition.

The run-up on a smooth impermeable slope is then expressed as:

$$\frac{R_{u2\%}}{H_{m0}} = A\gamma_b\gamma_f\gamma_\beta\xi_{0p} \tag{10.2}$$

with a maximum of

$$\frac{R_{u2\%}}{H_{m0}} = \gamma_f\gamma_\beta\left(B - \frac{C}{\sqrt{\xi_{m-1,0}}}\right)$$

, in which:

$R_{u2\%}$ = run-up level exceeded by 2% of the waves
H_{m0} = wave height based on the total energy in the spectrum
$\xi_{m-1,0}$ = the Iribarren number based on $T_{m-1,0}$

The parameters A, B and C are determined by curve fitting. In the table below two values are given; one set is the average value and the other set is the parameter to be used in designs. This is, in fact, the one plus the average value multiplied by the standard deviation. The average value can be used in probabilistic designs, while the second value can be used in deterministic designs.

Type of parameter	A	B	C
Average value	1.65	4.0	1.5
Value to be used in design (average+ 1σ)	1.75	4.3	1.6

Table 10.2: Fit factors for run-up calculations

The run-up level can effectively be reduced by designing a berm at still water level, by increasing the roughness of the surface or by increasing the permeability of the structure. This reduction is expressed in terms of reduction factors γ. For the roughness and the permeability, the factor γ_f is used. For rough permeable structures, Table 10.3 gives some values. For more impermeable slopes, some values can be found in Table 4-15.

Waves approaching the structure at an angle will also lead to reduced run-up levels. The effect of the approach angle is included by the factor γ_β:

$$\gamma_\beta = 1 - 0.0022\beta \tag{10.3}$$

in which β is the approach angle in degrees.

The reduction for a berm is included in the parameter γ_b. For the determination of γ_b, one is referred to the Overtopping Manual [PULLEN ET.AL., 2007].

For a permeable rubble construction basically the same equation as for smooth slopes can be used. However, as can be seen from Figure 10.4, for high Iribarren numbers there are some differences compared to

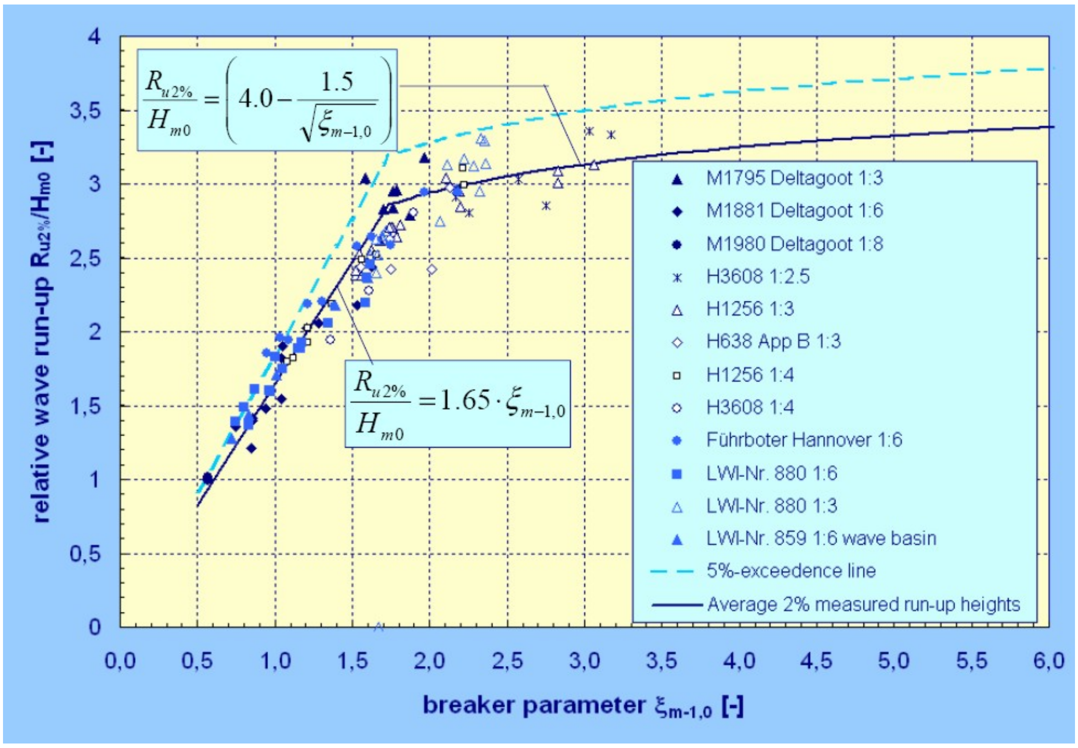


Figure 10.3: Wave run-up on a smooth impermeable slope [PULLEN ET.AL., 2007]

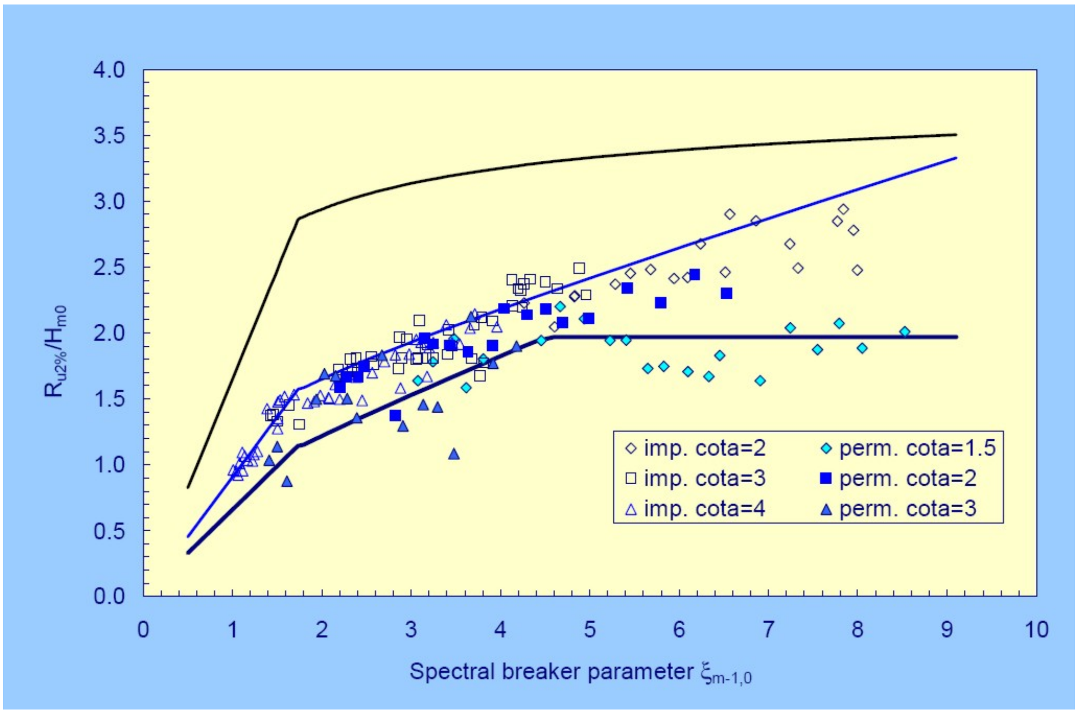


Figure 10.4: Relative 2% run-up on rock slopes [PULLEN ET.AL., 2007]

Structure	γ_f
Smooth, impermeable (like asphalt or closely pitched concrete blocks)	1.0
Open stone asphalt etc.	0.95
Grass	0.9 – 1.0
Concrete blocks	0.9
Pitched quarry stone blocks (granite, basalt)	0.85 – 0.9
Rough concrete	0.85

Table 10.3: Correction factors for roughness

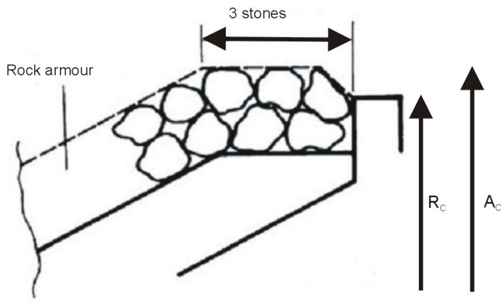


Figure 10.5: Difference between R_C and A_C

smooth slopes. For two layers of rock on a impermeable slope, one may use $\gamma_f = 0.55$. This reduces to $\gamma_f = 0.40$ for two layers of rock on a permeable core. This influence factor is used in the linear part of the run-up formula to approximately $\xi = 1.8$. For higher Iribarren numbers, this factor linearly increases to 1 for $\xi = 10$ and then remains constant. For a permeable core, however, a maximum is reached for $R_{u2\%}/H_{m0} = 1.97$. The physical explanation for this is that if the slope becomes very steep (large values for ξ) and the core is impermeable, the surging waves slowly run up and down the slope and all the water stays in the armour layer, leading to fairly high run-up. The surging wave actually does not “feel” the roughness anymore and acts as a wave on a very steep smooth slope. For a permeable core, however, the water can penetrate into the core which decreases the actual run-up to a constant maximum (the horizontal line in Figure 10.4).

For rough slopes one should also select either the coefficients for a deterministic design method or calculate the average values for including the figures in a probabilistic computation.

The probability of overtopping can be calculated by using the Rayleigh distribution:

$$P_{ov} = \frac{N_{ov}}{N} = \left[- \left(-\sqrt{\ln - 0.02} \frac{R_c}{R_{u2\%}} \right)^2 \right] \tag{10.4}$$

in which N_{ov} is the number of overtopping waves and N the number of waves in a storm.

One should realise that R_C is the crest height (usually a wave wall behind the armour layer) which may be somewhat lower than the actual height (A_C) of the breakwater. For various armour units, one may combine several formulas leading to one expression including the wave height and the unit dimension.

$$P_{ov} = \frac{N_{ov}}{N} = \left[- \left(\frac{A_C d_n}{0.19 H_{m0}^2} \right)^{1.4} \right] \tag{10.5}$$

This equation is based on work of DE JONG [1996] and various CLASH-tests.

10.4. OVERTOPPING FOR RUBBLE MOUNDS

Overtopping is defined as the quantity of water passing over the crest of a structure per unit of time. It therefore has the same dimensions as the discharge Q [m^3/s]. Because this quantity of water is often a linear function of the length of the structure, it is expressed as a specific discharge per unit length [m^3/s/m].

When designing breakwaters, the quantity of overtopping may be important to determine the capacity of the drainage facilities required for port areas directly protected by the breakwater or to assess the risk to people or installations on the crest of the breakwater.

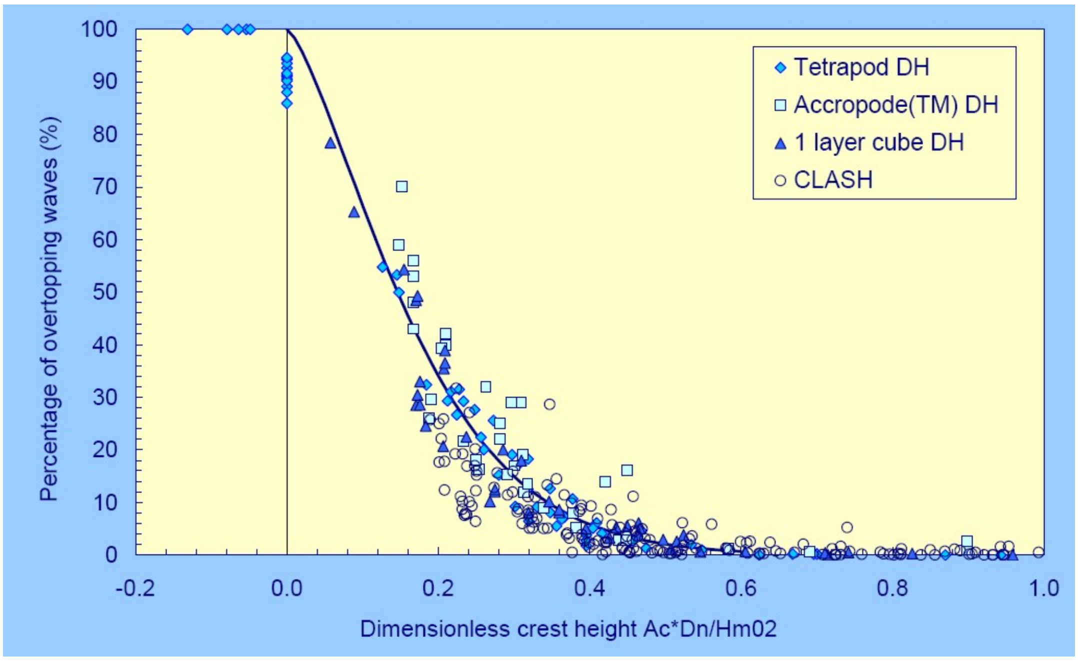


Figure 10.6: Percentage of overtopping waves for rubble mound breakwaters as a function of relative crest height and armour size ($R_C < A_C$) [PULLEN ET.AL., 2007]

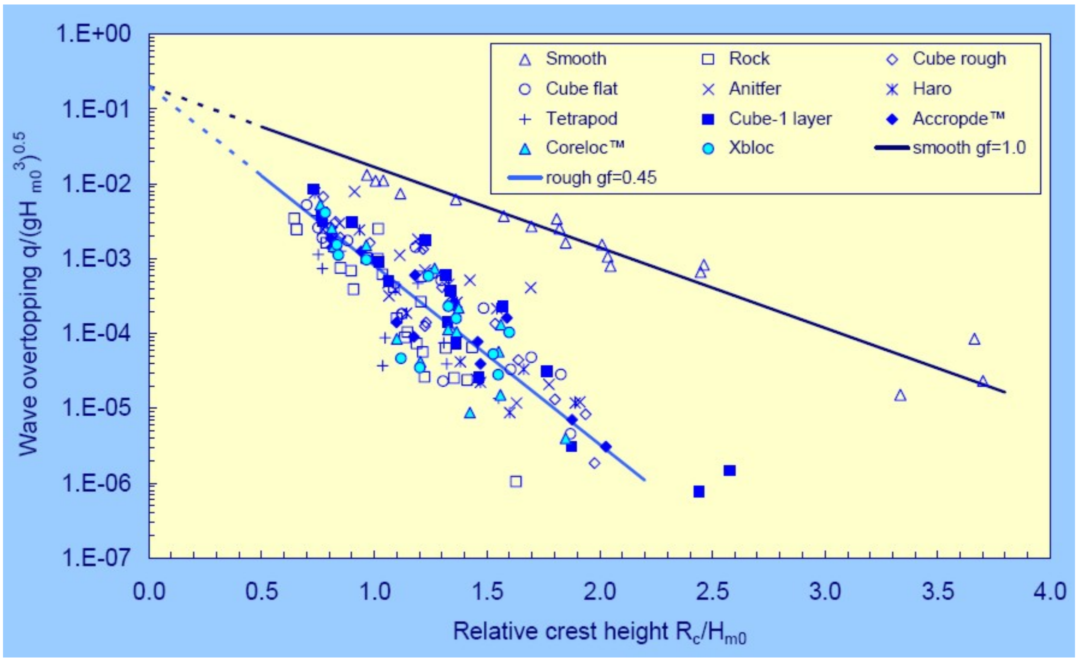


Figure 10.7: Mean overtopping for 1:1.5 slopes (smooth and rough slopes) [PULLEN ET AL, 2007]

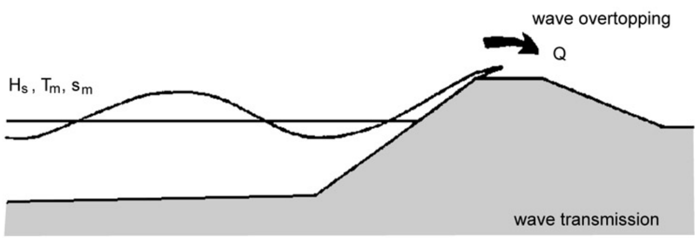


Figure 10.8: Typical wave overtopping

The EurOtop manual [PULLEN ET AL, 2007, 2016] gives the following recommendations:

Hazard type and reason	Mean discharge q (l/s/m)	Max volume V_{max} (l/m)
Pedestrians: Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1-10	500 at low level
Pedestrians: Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet wider walkway	0.1	20-50 at high level
Vehicles: Driving at low speed, overtopping by pulsating flows at low flow depths no falling jets, vehicle not immersed	10-50	100 - 1000
Vehicles: Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01-0.05	5-50 at high level
Property: Significant damage or sinking of larger yachts	50	5000-50000
Property: Sinking small boats set 5 - 10 m from wall; damage to larger yachts	10	1000-10000
Property: Building structure elements	1	-
Property: Damage to equipment set back 5 – 10 m	0.4	-

Table 10.4: Tolerable discharges

For unprotected inner slopes or slopes with natural vegetation, also maximum overtopping discharges have been defined (order 0.1 –10 l/s/m). For more details, one is referred to SCHIERECK [2001].

Systematic research on overtopping for smooth and impermeable slopes had led to specific design formulas, similar to the run-up formulas as defined before. This overtopping is discussed in detail in SCHIERECK [2001]. By introducing a roughness factor $\gamma_f = 0.55$ one receives quite accurate results also for riprap structures. For steep slopes (as for a breakwater) the resulting equation is:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-2.3\frac{R_C}{H_{m0}\gamma_f\gamma_\beta}\right) \tag{10.6}$$

See also Figure 10.7. The roughness factors from Table 10.3 can be used.

For berm breakwaters one might use the berm-reduction factor as defined for smooth slopes, however this may not always lead to correct answers. The EurOtop manual [PULLEN, 2007, 2016] gives some empirical formulae for special cases.

10.4.1. SPATIAL DISTRIBUTION OF OVERTOPPING

Part of the water which overtops the outer crest line will infiltrate into the crest, while another portion of the water will reach the inner slope. For the calculation of the stability of the inner slope only the water which reaches the inner slope is relevant.

STEENAARD [2002] has investigated how the overtopping quantity q_1 is split into a part that is infiltrating into the crest (q_2), and a part that is transmitted towards the inner slope (q_3). For regular waves he concluded that the ratio between q_2 and q_1 is given by:

$$\frac{q_2}{q_1} = \begin{cases} \left(\frac{Q^*_{tot} - Q^*_d}{Q^*_{tot} + 0.07}\right) & \text{for } {}^*_{tot} \geq Q^*_d \\ 0 & \text{for } Q^*_{tot} < Q^*_d \end{cases} \tag{10.7}$$

in which:

q_1 = overtopping in m^3/s per meter breakwater at the seaward edge of the crest
q_2 = overtopping in m^3/s per meter breakwater infiltrating into the crest
Q^*_{tot} = dimensionless overtopping, given by:

$$Q^*_{tot} = \frac{q_1}{\sqrt{gB^3}}$$

B = crest width (m)
Q^*_d = threshold value, with a constant value of $8.2 \cdot 10^{-3}$
g = acceleration of gravity (m/s^2)

10.5. OVERTOPPING FOR VERTICAL WALLS

Also in the framework of CLASH all existing tests on vertical wall breakwaters were reanalysed. For wave impact under non-impulsive conditions this resulted in the following equation:

$$\frac{q}{\sqrt{gH^3_{m0}}} = a \exp\left(-b\frac{R_C}{H_{m0}}\right) \quad \text{for } 0.1 < \frac{R_C}{H_{m0}} < 3.5 \tag{10.8}$$

in which:
q = unit discharge ($m^3/m/s$)
a, b = experimental coefficients;
in case of a deterministic design one should use $a = 0.04$ and $b = 1.8$;
for probabilistic design (for the mean value) $b = 2.6$

The variation of the results is indicated in Figure 10.9.

Under impulsive conditions, the overtopping is much more and is given by:

$$\frac{q}{h^2_* \sqrt{gh^2_s}} = a\left(h_* \frac{R_C}{H_{m0}}\right)^{-3.1} \tag{10.9}$$

in which:

$$h_* = 1.35 \frac{h_s}{H_{m0}} \frac{2\pi h_s}{gT^2_{m-1,0}}$$

h_s = water depth at toe of the structure
a = empirical coefficient,
for deterministic computations $a = 2.8 \cdot 10^{-4}$,
for the mean value $a = 1.5 \cdot 10^{-4}$.

By means of a parapet, a wave return wall, a bullnose or a similar structure, the overtopping can be reduced considerably. The reduction depends very much on the exact shape of this structure. It must be kept in mind that a vertical face breakwater causes a lot of spray when hit by a wave. The spray may also be blown over the breakwater. This effect is not included in the above formula, but can be included using a wind adjustment factor. This factor is relevant for low overtopping discharges (average discharge less than 0.1 l/s/m) and may increase the overtopping up to a factor 5. For more details, one is referred to the Eurotop manual [PULLEN ET AL, 2007].

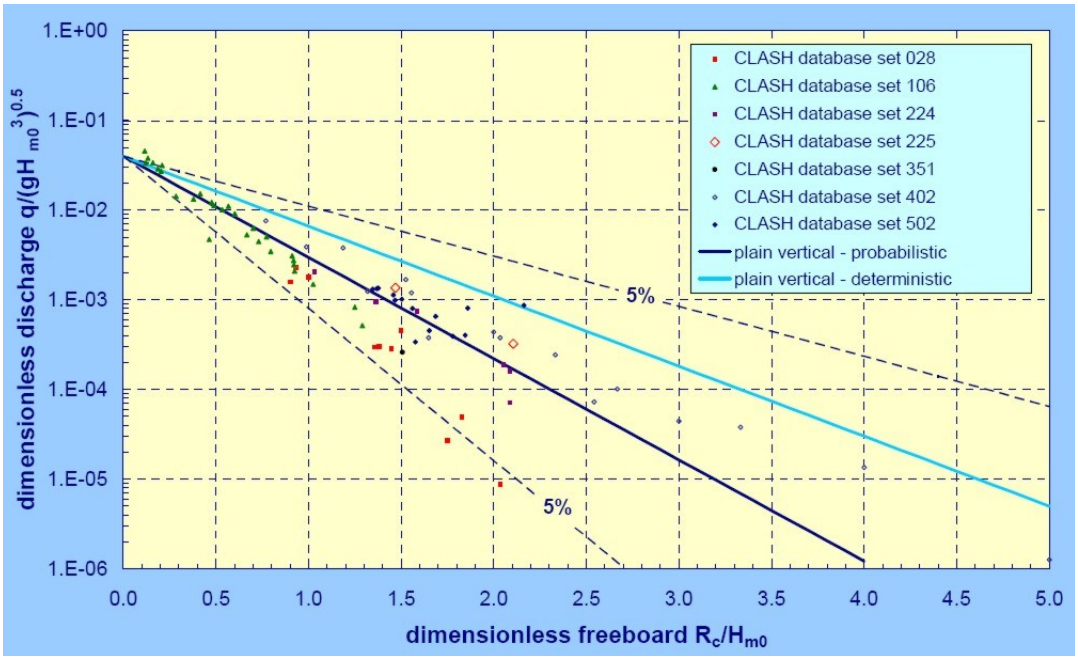


Figure 10.9: Wave overtopping for a vertical breakwater [PULLEN ET AL, 2007]

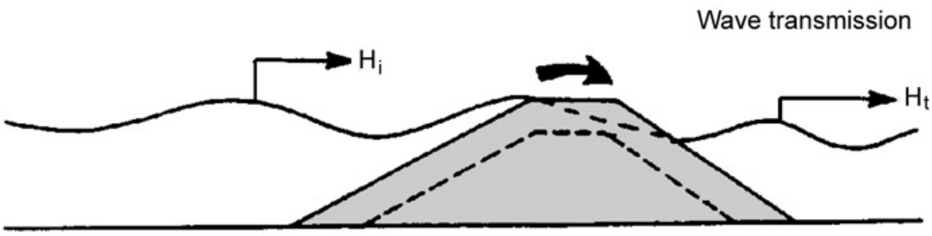


Figure 10.10: Typical wave transmission

10.6. TRANSMISSION BY RUBBLE MOUNDS

Wave transmission is the phenomenon in which wave energy passing over and through a breakwater creates a reduced wave action in the lee of the structure (Figure 10.10). This will certainly happen when considerable amounts of water are overtopping the structure. Wave transmission is also possible, however, when the core of the structure is very permeable and the wave period is relatively long. It is specifically the influence of these two factors that for a long time has prevented the derivation of an acceptable formula for wave transmission by rubble mound breakwaters.

Many authors (SEELIG [1980], POWELL AND ALLSOP [1985], DAEMRICH AND KAHLE [1985], VAN DER MEER [1990]) have investigated the effects of wave transmission. This has resulted in the diagram presented in Figure 10.11. In practice, limits of about 0.1 and 0.9 are found.

It is remarkable that for $R_C = 0$, which represents a structure with the crest at SWL, the transmission coefficient is in the order of 0.5. This means that a relatively low structure is already rather effective in protecting the harbour area behind the breakwater. In combination with the requirements for tranquillity in the harbour, the designer can decide on the minimum required crest level.

Eventually, DAEMEN [1991] (see also VAN DER MEER AND D'ANGREMOND [1991]) in his MSc thesis was able to produce an acceptable formula that relates the transmission coefficient to a number of structural parameters of the breakwater. To account for the effect of permeability, Daemen decided to make the freeboard R_C of the breakwater dimensionless dividing it by the armourstone diameter. This eliminates much of the scatter that was present in previous approaches. For traditional low crested breakwaters the Daemen formula reads as follows:

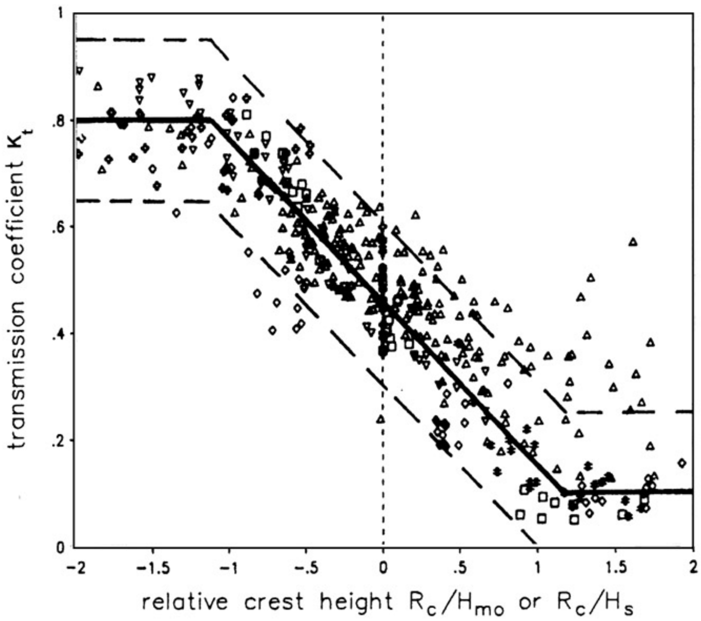


Figure 10.11: Wave transmission for low crested structures [D'ANGREMOND ET AL., 1996]

$$K_t = a\frac{R_c}{d_{n50}} + b \tag{10.10}$$

with:

$$a = 0.031\frac{H_i}{d_{n50}} - 0.24$$

$$b = -5.24 s_{0p} + 0.0323\frac{H_i}{d_{n50}} - 0.0017\left(\frac{B}{d_{n50}}\right)^{1.84} + 0.51$$

in which:

K_t = H_{st}/H_{si} = transmission coefficient
H_{si} = incoming significant wave height
H_{st} = transmitted significant wave height
R_C = crest freeboard relative to SWL
d_{n50} = nominal diameter armourstone
B = crest width
s_{op} = wave steepness

The use of the Daemen formula is complicated when it is decided to use a solid crown block or to grout armourstones with asphalt into a solid mass. Therefore, DE JONG [1996] (see also D'ANGREMOND ET AL. [1996]), reanalysed the data and came up with a different expression. He chooses to make the freeboard dimensionless in relation with the incoming wave height:

$$K_t = -0.4\frac{R_c}{H_{si}} + 0.64\left(\frac{B}{H_{si}}\right)^{-0.31}\left(1 - \exp\left(-0.5\xi\right)\right) \tag{10.11}$$

The factor 0.64 is valid for permeable structures; it changes to 0.80 for impermeable structures. The lower boundary is $K_t = 0.075$, the upper boundary is $K_t = 0.8$. See also Figure 10.12.

However, both equations have a rather limited range of application. In the DELOS research project therefore a combined formula was looked for, on the basis of an extensive database with observations. The conclusion was that for narrow-crested breakwaters ($B/H_s < 8$) one should apply Equation 10.11. For wider crests this formula is not valid. For wide crested breakwaters ($B/H_s > 12$), one may use

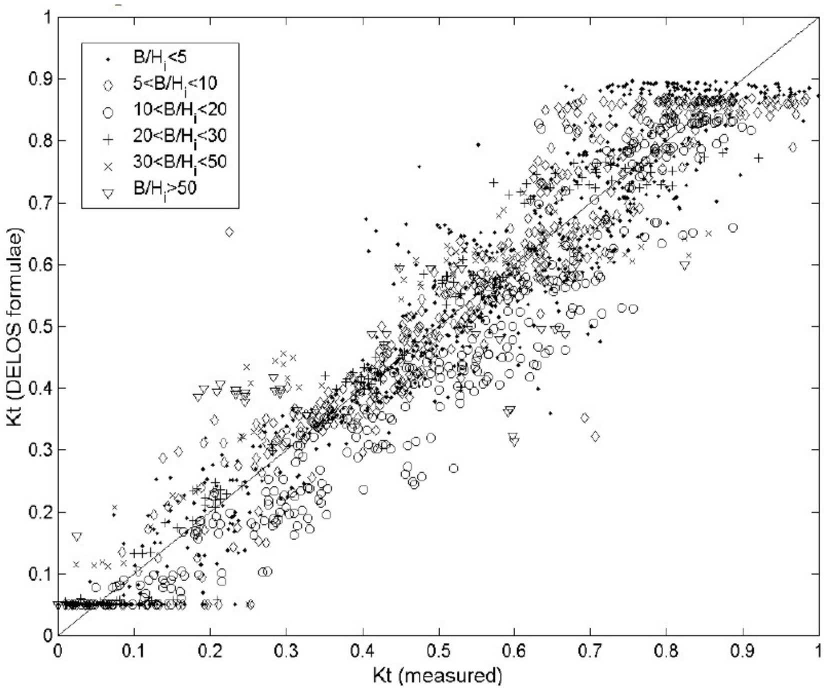


Figure 10.12: Comparison of computed and observed transmission coefficients with Equations 10.11 and 10.12 (from DELOS database)

$$K_t = -0.35\frac{R_c}{H_{si}} + 0.51\left(\frac{B}{H_{si}}\right)^{-0.65}\left(1-\exp(-0.41\xi)\right) \tag{10.12}$$

The lower boundary is $K_t = 0.05$, the upper boundary is $K_t = 0.93 - 0.006B/H_s$. Interpolation between the two equations is done for $8 < B/H_s < 12$. See also Figure 10.12.

10.7. NEURAL NETWORKS

As followed from the previous sections, many formulas mainly consist of a basic format based on dimensionless parameters and some curve fitting. A more advanced tool to perform this fit is the Neural Network. The advantage of a Neural Network is that there is no need to predefine the relations between the individual parameters in the equations. However, because the Neural Network does not use physical relations, the amount of data required to fit needs to be large, and also the quality of the data needs to be good. Further on it is necessary to make the database homogeneous. In the framework of the program CLASH [VERHAEGHE ET AL., 2003] as well as in additional work [VAN OOSTEN and PEIXÓ MARCO, 2005] a database has been set-up including nearly all existing observations of overtopping, transmission and reflection, both from model tests as well as from prototype observations. In total there are more than 10000 observations in the database. A Neural Network can be trained. With a trained Neural Network anyone can make calculations in a similar way as applying a classical equation. Neural Networks are available for overtopping, transmission and reflection. The additional advantage is that the Neural Network also indicates the reliability of the answer.

For example in Figure 10.13, with the Neural Network developed by VAN OOSTEN and PEIXÓ MARCO [2005], a calculation is made for a given breakwater. The transmission coefficient is plotted while varying the wave period. The short-dashed line in the figure is the result of equation 10.6. The thick line is the prediction of the Neural Network. But also the reliability is given with long-dashed lines. It is clear that for periods larger than 11s, one should be very careful with the found results. Because both the Neural Network as well as the Equation equation 10.6 are based on the same data, the unreliability for periods longer than 11 s found by the Neural Network is also applicable for equation 10.6. This scatter can also be seen in Figure 10.7, however there it is unclear that the scatter is more for larger periods.

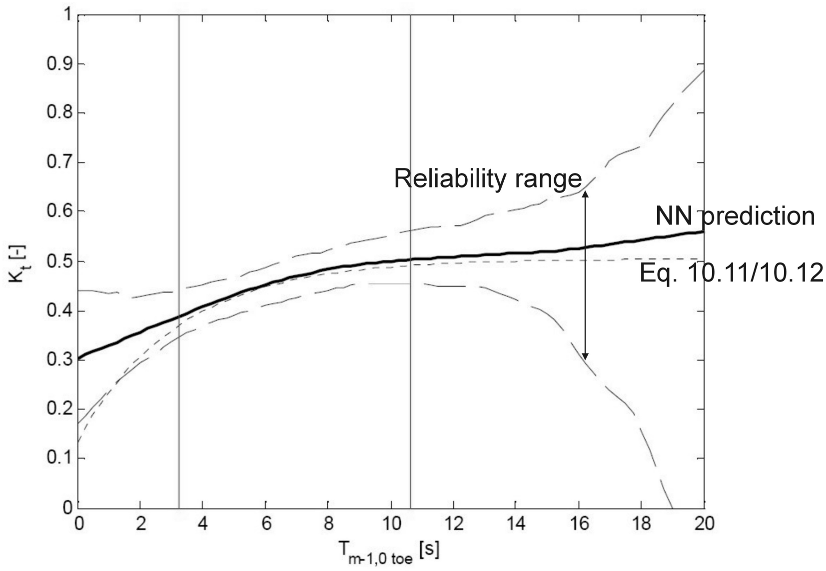


Figure 10.13: Result of a Neural Network calculation [VAN OOSTEN and PEIXÓ MARCO, 2005]

11

DESIGN PRACTICE OF BREAKWATER CROSS-SECTIONS

The design of a breakwater is not only an application of theory but is also largely based on experience. Therefore this chapter attempts to combine a set of simple rules into a design manual. However, it is felt that a set of design rules can never satisfactorily solve all design problems, so it is essential to look into cross-sections of existing breakwaters, while reading this chapter. Studying this chapter with some degree of success is only possible if one analyses the examples given and at the same time, contemplates why the specific designs have been followed.

11.1. INTRODUCTION

In the preceding chapters we have used terms such as 'armour layer' and 'core', we have discussed the stability of concrete armour units and we have seen the principles of a berm breakwater. It is now time to discuss some practical rules for the design of cross-sections. Many of these rules are simply rules-of-thumb, and only gradually has an experimental basis been created on which acceptance or rejection of these rules can be based. The present chapter is thus a mixture of research, experience and plain engineering judgement.

It must be kept in mind that natural rock is usually obtained by blasting, and that the size of the stone obtained (yield curve) can only be influenced to a limited extent. It is much easier to increase the percentage of fine material than the percentage of coarse material. Any material that is blasted must be handled in the quarry, whether or not it is used in the breakwater. If it is not used, it must be left somewhere, and in many countries, the deposition of waste material requires special licences that are difficult to obtain. It is therefore an element of good design to try to use all materials produced by the quarry.

In principle, we can follow two options.

- Split the quarry production into two or three categories (filter material, core and armour). This almost automatically leads to a berm breakwater. The gradation of the stone categories is rather wide (d_{n85}/d_{n15} up to 2.5 or even 3).
- Classification of the quarry output into a larger number of categories, each with a narrower gradation (d_{n85}/d_{n15} up to 1.5). In this way, it is possible to select the proper stone size for a specific function. It leads to a more economical (and thus more economic) use of material.

The berm breakwater with its larger volume has an advantage when the production cost is low and the quarry is located near the site of the breakwater. When quarry stone is more costly and the quarry is at greater distance, it is more economical to build a multi-layered breakwater. However, also in this case, it is advantageous to keep the design of the cross-section as simple as possible.

11.2. PERMEABILITY/POROSITY AND LAYER THICKNESS

11.2.1. PERMEABILITY/POROSITY

When rock or concrete blocks are placed in the cross-sections of a breakwater, it is important to have an idea of the permeability/porosity and the layer thickness. The permeability/porosity is important because it influences the hydraulic response of the structure, and it effects the stability (P in the Van der Meer formula).

During construction, knowing the porosity is important because it determines the bulk density. Quarry stone is often paid for per ton of material. When the contractor is paid per m3 for placing the material in the cross-section, as is often the case, to make a proper cost estimate it is essential to know the bulk density of the material.

The volumetric porosity n_v is defined as follows:

$$n_v = 1 - \left(\frac{\rho_b}{\rho_r}\right) \tag{11.1}$$

in which:
ρ_b = bulk density as laid
ρ_r = density of rock

Determination of the bulk density is not a simple task because of the errors made at the boundaries of the measured volume. Preliminary data can be found in the Rock Manual [2007], or in BREGMAN [1998].

Table 11.1, with data from the Rock Manual indicates porosity levels between 35% and 40% for quarry stone placed in thin layers. Bulk handling may lead to a porosity that is up to 5% higher than the values in Table 11.1. A wider gradation, however, may lead to lower porosity. Because of the uncertainties in the determination of the porosity and the bulk density, it is recommended that some in situ tests to ascertain the actual values should be carried out. It is emphasized that special placement of quarry stone (with the longest dimension either parallel or perpendicular to the slope) has a big effect on both layer thickness and stability.

In Table 11.2 the porosity of concrete units is also given. Here again, the method of placement may cause large differences in porosity and in stability.

Layer and Placement type	Parameter	Spherical foot staff method		Highest point survey method	
		Blocky rock	Irregular rock	Blocky rock	Irregular rock
Single dense	k_t	0.84	0.77	0.89	0.82
Single dense	n_v	0.32	0.35	0.36	0.40
Double standard	k_t	0.91	0.87	0.96	0.92
Double standard	n_v	0.32	0.35	0.36	0.40
Double dense	k_t	0.91	0.87	0.96	0.92
Double dense	n_v	0.31	0.35	0.35	0.36

Table 11.1: Thickness and porosity in narrow gradation armour layers [ROCK MANUAL 2007]

Type and shape of units	Layer thickness n	Placement	Layer coefficient k_t	Porosity n_v	Placing distance	
					Horizontal $\Delta x/d_n$	Vertical $\Delta x/d_n$
Cubes	2	Random	1.10	0.47	1.70	0.85
Tetrapods	2	Random	1.02	0.50	1.98	0.99
Dolos	2	Random	0.94	0.56	2.19	1.10
Core-Loc	2	Random	1.52	0.62	1.85	0.92
Accropode	1	Special	1.29	0.53	1.82	0.91
Xbloc	1	Special	1.40	0.61	1.92	0.94

Table 11.2: Thickness and porosity for concrete units[ROCK MANUAL 2007]

11.2.2. LAYER THICKNESS AND NUMBER OF UNITS

For armour layers, it is important to know the effective layer thickness for single, double or triple layers of material. This is essential information when designing and constructing a cross-section. The crest level is determined on the basis of the required protection on the lee side of the breakwater. Given this required crest level, one must know at what level the core must be finished. If the crest of the core is too high, this indicates that too much material has been used, which will probably not be paid for by the client. If the crest of the core

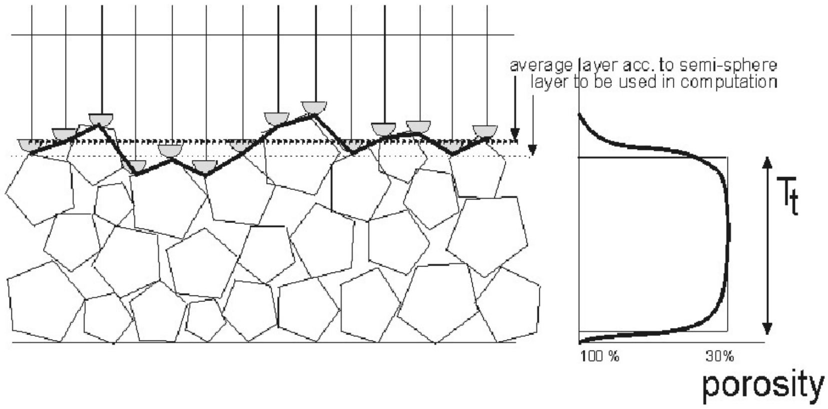


Figure 11.1: measuring the thickness of a rock layer

is too low, and the client still wants the given crest level, this means that extra armourstone has to be used. Since armourstone is generally more expensive than core material, this again will cause a financial loss to the contractor [1]. See also appendix 7.

For relatively thin layers, there is a variation in nv at the boundaries of the various layers. First of all, the boundary is not defined easily. Usually one measures the top layer with the so-called hemisphere method or spherical foot method. See Figure 11.1. From the work of BOSMA [2001], it follows that there is a difference in the measured layer thickness with the hemisphere method and when using a volumetric method. Sometimes only the highest points are measured. In that case one gets even more different values. See also Table 11.1. The difference is in the order of 5% of d_n, which can become a large value when using thin layers and large stones. This difference is relevant because the designer gives a layer thickness, and the contractor has to buy stones by weight (and thus by volume).

The effective layer thickness is discussed extensively in the Rock Manual [2007]. The thickness of a layer t (in m) is calculated as:

$$t = nk_t d_{n50} \tag{11.2}$$

in which n is the number of stones across the layer. In the American literature, k_Δ is often used instead of k_t, and r instead of t. Values for k_t are also given in Table 11.2. It is also important to know the number of armour units N required to cover a certain area A. This number is:

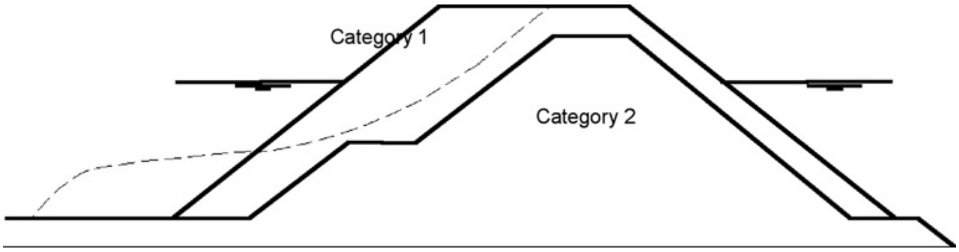
$$N = nk_t A(1-n_v)d_{n50}^{-2} \tag{11.3}$$

Although Table 11.2 gives values of k_t and n_v for concrete armour units as well, it is emphasized that those values can fluctuate considerably. This depends on the interpretation of the qualification “random placement”. It is possible for instance to place cubes much more densely than indicated in the table. This will improve the stability, but it will also influence the reflection, the run-up, the overtopping, and the transmission. Therefore, care is required when data from inexperienced researchers are used.

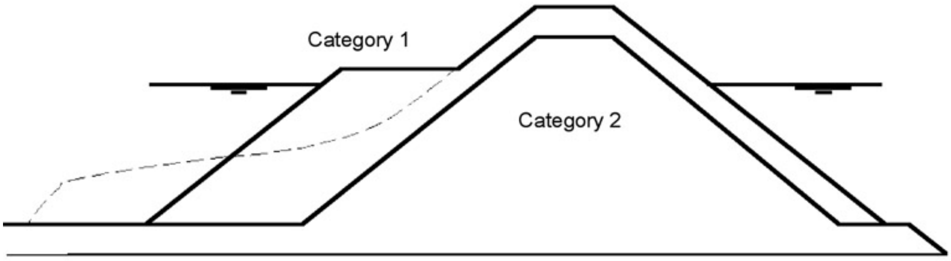
11.3. BERM BREAKWATER

In principle, the cross-section of a berm breakwater consists of two materials, core material and armour material. The armour material is the coarsest fraction of the quarry yield and the core material is the finer fraction. The armour material is located in a berm along the outer slope of the breakwater. The quantity of armour material is chosen in such a way that after a series of storms predicted during the lifetime of the breakwater, the core be always covered by at least a double layer of armour material. This applies to the front slope, the crest, and the exposed part of the inner slope (i.e. to a little below low water level).

[1] For reasons of construction the contractor often wants the crest of the core at HW + some freeboard On top of this the armour has to be placed, which results sometimes in an unnecessarily high breakwater. On the other hand, a crest which is below SWL might give too much wave transmission.



(a) Berm breakwater with high berm



(b) Berm breakwater with berm at MSL

Figure 11.2: Berm breakwater types

CREST LEVEL

In Chapter 8 (Dynamic Stability), it was explained that the crest level determines the stability of the inner slope. Given a reasonable ratio between wave height and nominal stone size, the minimum crest level follows from the accepted level of damage rather than from the functional requirements. When designing the cross-section it is good engineering practice to create a safe level at which trucks, bulldozers and cranes can work without much interference from waves. This also requires a minimum width of the crest.

FILTER LAYER

On the front slope, the incident and reflected waves create a complicated pattern of orbital velocities and pressure fluctuations. This will cause larger stones to sink slowly into the seabed, unless the latter consists of rock. The same hydraulic conditions will enhance the risk of scour and erosion of the seabed just in front of the breakwater. Eventually, both phenomena together will lead to the loss of material and potentially to the loss of the stability of the entire breakwater. Therefore, to prevent the formation of a scour hole close to the structure, it is good engineering practice to provide a filter layer under the toe of the breakwater and to extend this filter some distance in front of the toe. In the case of a berm breakwater, it is also wise to apply such a filter in the area that will be covered by armourstone after reshaping of the seaward slope.

Protection of the seabed may consist of a granular filter, or a geotextile with a cover of quarry stone. Filter rules are beyond the scope of this book; the reader is referred to the Rock Manual [2007] or to SCHIERECK [2001].

SLOPES

Since wave action will reshape the profile of a berm breakwater anyway, it makes no sense to construct the cross-section to a particular slope. It is generally accepted that the material should be allowed to find its natural angle while it is being deposited by barge, dump truck and/or bulldozer.

This leads to two frequently used basic cross-sections; one with the berm at crest level and one with the berm just above MSL. These two cross-sections are presented in Figure 11.2a and Figure 11.2b.

11.4. TRADITIONAL MULTI-LAYERED BREAKWATER

11.4.1. CLASSIFICATION

Although there are some standard cross-sections for traditional multi-layered breakwaters, numerous variations can be made which makes classification quite difficult. The first, and most logical classification, is done by crest level. A second type of classification, is by type of crest. The third type refers to the kind of armour units.

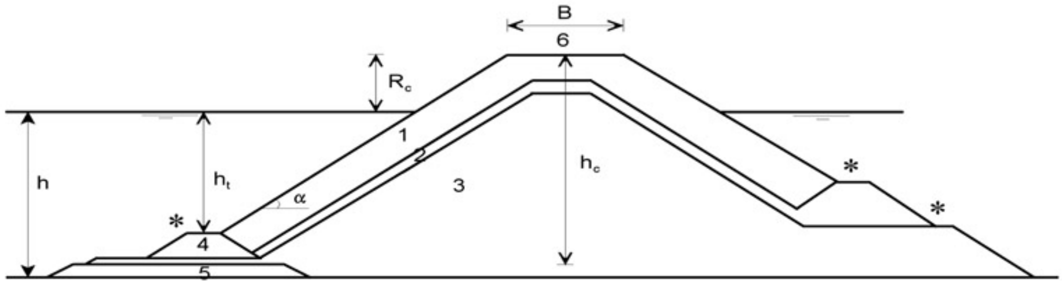


Figure 11.3: Definition sketch of cross-section

HIGH / LOW CREST

In this context, a high crest is considered so high that the inner slope is not severely attacked by the action of overtopping waves. The inner slope is designed for waves generated by passing ships or waves locally generated in the harbour basin. A low crest is so low that the inner slope is severely attacked by waves passing over the crest. To protect the inner slope, usually the same armour type is used on the inner slope as on the seaward slope. The crest level is generally determined on the basis of the functional requirements (wave tranquillity on the lee side). It must be clear that the selection of too high a crest level leads to excessive use of material because the volume of the structure is proportional to the square of its height. Selection of a low crest level may have serious consequences for the construction method and the maintenance. A low crest may also lead to the exclusive use of floating equipment.

CREST DESIGN

A second type of classification is possible with respect to the nature of the breakwater crest. This can consist of armour units or of a solid cap block. If the crest consists of armour units, the crest is usually inaccessible to people or equipment. This means that maintenance is only possible by using floating equipment. If the crest is formed by a solid block it is common practice to design it in such a way that it can be used as a road, both during construction and subsequently for maintenance or other purposes.

ROCK OR CONCRETE ARMOUR UNITS

A third type of classification is possible on the basis of the type of armour material. Since the maximum size of quarry stone is limited, it is not uncommon to reduce the seaward slope to obtain sufficient stability. (Note: the inner slope can be steeper!). If concrete armour units are used, it can easily be demonstrated that a steep slope of 1V:1.5H ($\cot\alpha = 1.5$) leads to the most economic design.

11.4.2. GENERAL DESIGN RULES

To explain the various design rules, a definition sketch of a multi-layered cross-section is given in Figure 11.3. It indicates the main elements of such a breakwater and their respective names.

TOLERANCES

Regardless of the construction element under consideration, it must be clear that significant size deviations can be expected. Not only deviations between size of the design and actual breakwater, but also deviations in size from one location to another location in the breakwater. The construction of mounds of stones with units up to 1m or 2m diameter has an inherent inaccuracy in the order of magnitude of the size of the stones. The design of the structure must take into account these tolerances. At different levels and locations, berms must be provided in which deviations in the actual measurements can be taken into account. These locations have been marked with an asterisk in Figure 11.4a through Figure 11.4e.

ARMOUR LAYER

It is evident that the armour layer must be able to withstand the wave attack during design conditions. The severity of the stipulated conditions follows from economic considerations. In general, the armour is placed in a double layer ($n = 2$), since this allows a few armour units to be displaced before the underlying material is exposed. The armour layer consists of concrete units or quarry stone. In the case of quarry stone, it is generally the heaviest fraction of the quarry yield curve. This has a narrow grading ($d_{85}/d_{15} < 1.5$). If quarry stone is used, it is possible to reduce the slope in order to improve the stability. When concrete armour units are used, it is more economical to increase the block weight if this is needed for stability.

CREST

If the crest consists of loose armour units, its width (B) must be at least 3 stones, or in the form of a formula:

$$B = n k_t d_{n50} \tag{11.4}$$

where $n = 3$, and k_t is taken from Table 11.1.

If the crest is formed by a concrete cap block, it must be ensured that the layer on which the block is placed or cast in situ is wider than the cap-block. It is never possible to fill voids under the cap-block after it has been placed. To protect the cap-block, it is recommended that armour material be placed at the seaward side to the full height of the breakwater. Parapet walls extending above the level of the armour units will be loaded heavily and in many cases, such walls have suffered extensive structural damage. To ensure that the armour layer shields the cap block properly it is recommended that a horizontal berm is maintained in front of the cap that is least as wide as one armour unit (see Figure 11.4a).

FIRST UNDER-LAYER

The layer directly under the armour layer is called the first under-layer. It is obvious that the units forming this layer must not pass through the voids in the armour layer. In many guidelines one finds a rule that the weight of units in the under-layer should not be less than 1/10 of the weight of the armour proper. This seems a very strict rule if compared with the filter rules of Terzaghi. These rules allow a ratio of 4 to 5 in diameter between two subsequent filter layers, which is a ratio of 43 - 53 (= 64-125) in weight. However, one must remain a bit on the conservative side because of the consequences of failure of the filter mechanism. The filter must therefore be “geometrically impermeable”. It is recommended that the weight ratio of subsequent layers of quarry stone be kept between 1/10 and 1/25. (d_{n50} ratio between 2 and 3). For more information, one is referred to Appendix 5. It is noted that in this context, choosing finer material for the first under-layer influences the notional permeability parameter P in the Van der Meer formula. This leads to the need for heavier armour material. A second consideration for the selection of a certain size for the material in the first under-layer may be the stability during construction. Depending on the construction sequence, the first under-layer may be exposed to a moderate storm during construction.

If the armour units are concrete blocks, the first under-layer is the heaviest fraction of the quarry yield curve. When the armour units are quarry stone, the first under-layer is composed of an intermediate fraction of the quarry yield which generally has a narrow grading.

TOE BERM

The toe berm is the lower support for the armour layer. In traditional literature, one finds a recommended weight that is equal to the weight of the first under-layer. With the most recent data, presented in the chapter on stability formulae, the designer can find a balance between the level of the toe berm and the size of the stone in the berm.

CORE

In most cases, the material of the first under-layer is such that the core can be situated directly under it. Again assuming a weight ratio between the first under-layer and the core of between 1/10 and 1/25, this means that the core material is a factor 100 to 625 lighter than the armour material. Usually it is not necessary to apply a second under-layer between the core and the first under-layer.

For the core, a material called “quarry run” or “tout venant” is typically used, indicating that it is meant to represent the finer fractions of the quarry yield curve. It must be noted that under no circumstances can overburden (degraded or weathered rock) be mixed with the quarry run. Quarry run generally has a wide ($1.5 < d_{85}/d_{15} < 2.5$) to very wide ($d_{85}/d_{15} > 2.5$) grading.

The use of large units in the core is not a problem from a stability point of view, although it does have a negative influence on wave transmission and sand tightness.

FILTER

Specifically under the seaward toe, large pressure gradients that tend to wash out material from the seabed through the structure may exist. Even extension of the core material under the toe berm may not guarantee the integrity of the structure as a whole. Loss of material in this region is a large threat to the stability of the armour layer. There are ample examples in the literature which show that substandard filters have initiated the failure of a breakwater.

It is therefore recommended that a geometrically impermeable filter should be placed under the seaward part of the breakwater. This filter may consist of a number of layers of granular material, or of a geotextile or another mattress. The pressure gradients under the centre of the structure and under the inner toe are generally much lower. Here, the quarry run may often act as a filter of sufficient quality. Care must be taken, however, when land is reclaimed directly behind the breakwater. Internal reflection may then again cause filter problems at the inner boundary of the breakwater. In such a case, special investigations are required to determine a satisfactory solution (e.g. reverse filter).

Since the layers of a granular filter are constructed at a considerable depth under water, it is necessary to make any separate layer thick enough to guarantee the stability of that particular sized material at any location. It will also be useful if the presence of the required material can be ascertained by inspection. In practice, this means that no layers thinner than 0.5m should be designed. This may lead to a relatively thick filter bed if a granular filter is used.

Scour protection

Just in front of the breakwater, the seabed may be eroded owing to a concentration of currents, or to a partially standing wave. Since the loss of bed material directly in front of the toe may cause a soil mechanical stability problem, it is recommended that a rubble blanket should be placed in front of the breakwater as scour protection. The width should be determined on the basis of local conditions, but should not be less than 5 to 10 m for a rubble mound breakwater and 10 to 15 m for a vertical wall breakwater.

11.4.3. Standard cross-sections

In Figure 11.4a, to Figure 11.4e, examples of standard cross-sections based on the considerations given in Section 0 are shown. These cross-sections show the changes of the leeward slope for increasing crest level and thus for decreasing overtopping and transmission. Examples of cross-sections with a concrete cap-block are also given. This feature is rarely seen in low-crested or submerged breakwaters, probably because of the difficulty of placing the block.

It must be noted that owing to the relation between local water depth and local significant wave height, the cross-section (including the size of the armour units) will vary considerably along the alignment of the breakwater. This gives the designer an added opportunity to match the quarry output to the over all demand of the project.

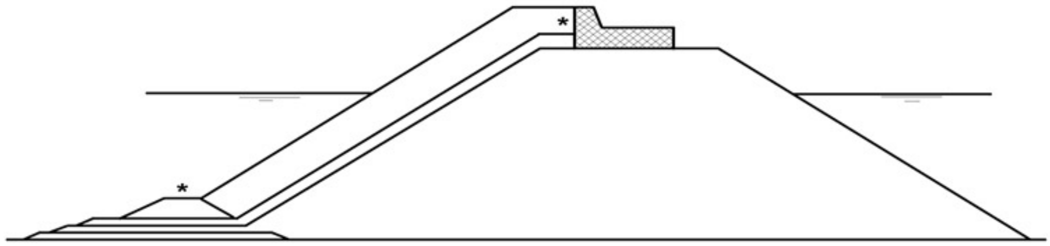
In the shallow water close to the shore, the standard design with a granular filter under the toe, will be difficult to construct. Owing to the thick filter bed, the level of the toe berm becomes too high. The problem can be solved by dredging a trench for the toe, by replacing the granular filter by geotextile or by modifying the toe berm. These four solutions are sketched in Figure 11.5a through Figure 11.5d.

Although the dredging of a trench seems expensive and a rather academic solution, it is not. In many cases, the bearing capacity of the subsoil is insufficient to create a safe foundation for the heavy load presented by the breakwater. This will be demonstrated by low safety coefficients in slip circle calculations, specifically when the soil is compressible and impermeable (consolidation time). In such cases, it is good engineering practice to apply soil improvement. Placing the toe in the dredged trench creates the intended soil improvement.

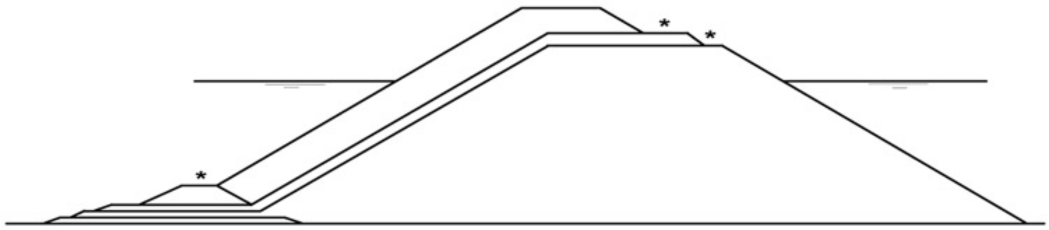
Although the figures presented in this chapter give a good impression of possible cross-sections, the reader is recommended to study cross-sections of actual breakwaters as well. This applies to both successful designs that have survived and to the unsuccessful examples that failed. When studying cross-sections in handbooks, it must be kept in mind that sometimes for the sake of simplicity, essential features are not shown.

11.5. Monolithic breakwaters

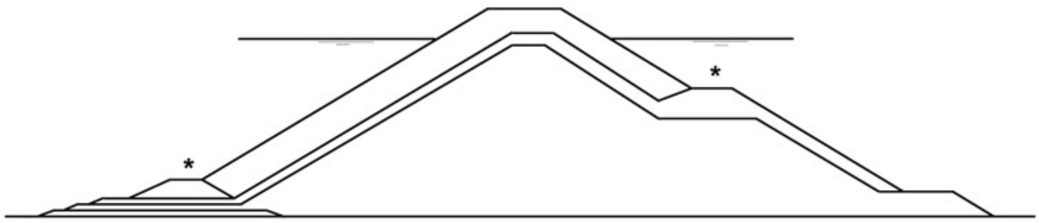
A basic difference between monolithic breakwaters and rubble mound breakwaters is that damage to monolithic breakwaters usually also means failure of the breakwater, while a rubble mound breakwater has a more ductile character. Because of this, safety levels for monolithic breakwaters are different. For details, one is referred to PIANC/MARCOM 28 [2003] and TAKAHASHI [2002]. The PIANC report suggested to use a level 1 probabilistic design (using partial safety coefficients). In the report, methods are given to determine the required coefficients.



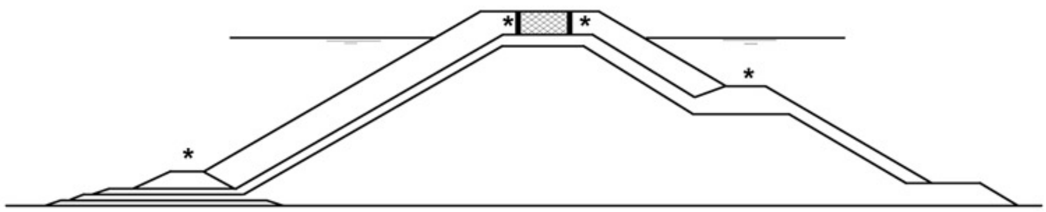
(a) Rubble mound breakwater – light overtopping (with cap block)



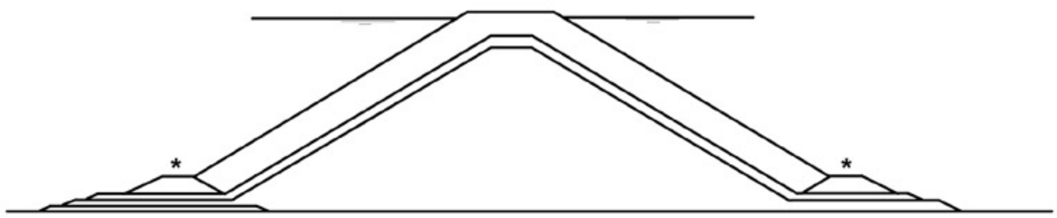
(b) Rubble mound breakwater - light overtopping



(c) Rubble mound breakwater - moderate overtopping

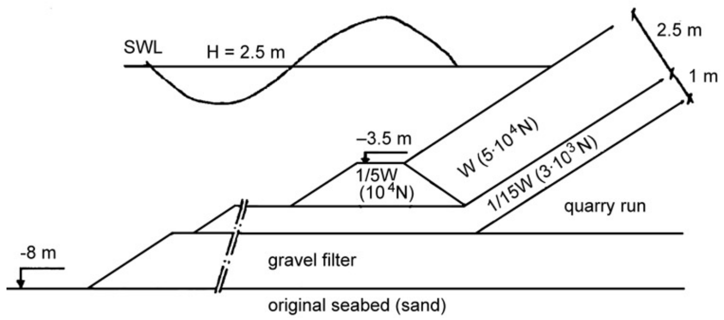


(d) Rubble mound breakwater - moderate overtopping (with cap block)

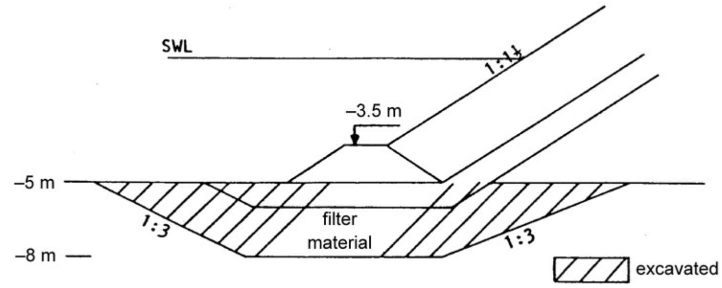


(e) Rubble mound breakwater - severe overtopping

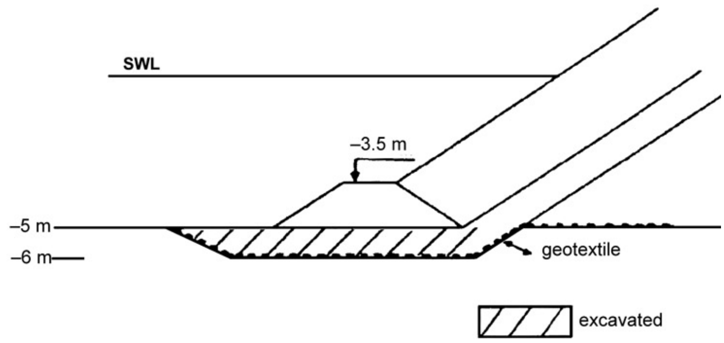
Figure 11.4: Standard breakwater cross-sections



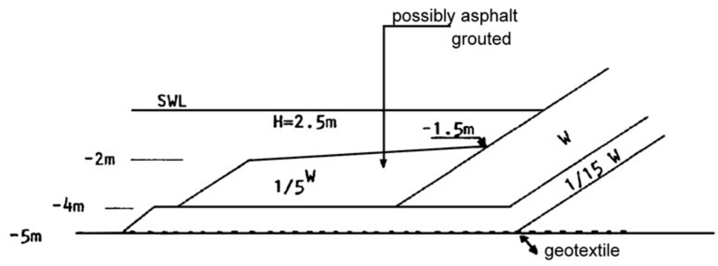
(a) Deep water, dredged trench



(b) Shallow Water dredged trench gravel filter



(c) Shallow water, dredged trench geotextile



(d) Shallow water (no excavation, geotextile and increased berm)

Figure 11.5: Standard breakwater toe details for shallow water

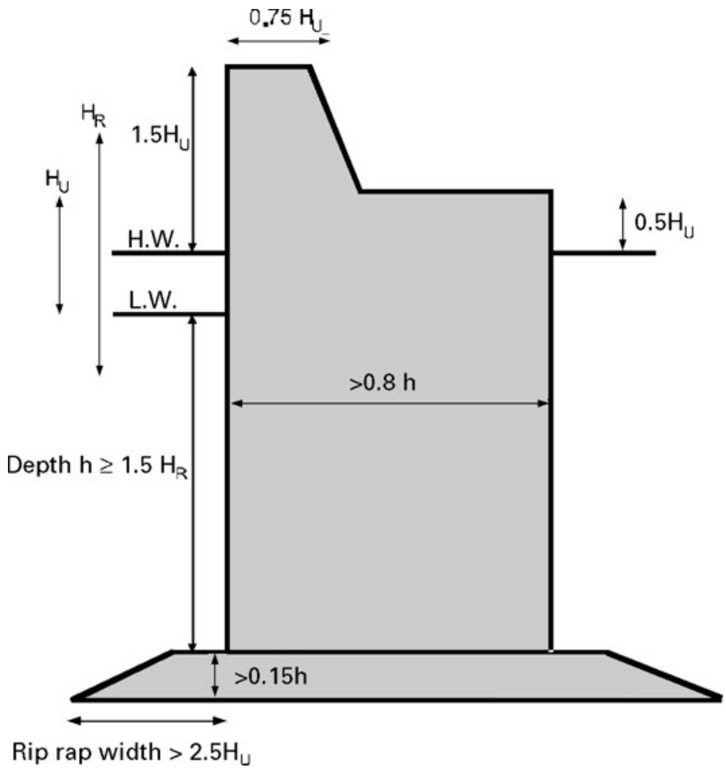


Figure 11.6: Recommended cross-section for a preliminary design

Applying this method one has to make a preliminary design and subsequently check if the design fulfils the requires safety requirements. For a preliminary design one could follow the recommendations from previous PIANC reports (Figure 11.6). This means that one has to define two design wave heights: a value Hr which is the design wave height for the ULS condition and a value Hu which is the design wave height for the SLS condition.

- Select a wall that presents a free height[2] that is at least $1.5H_r$ below low water
- Select a caisson width of at least 0.8 times the free height
- A toe protection against undermining; the thickness of which is at least 0.15 times the free height. (This places the seabed at least $1.72H_r$ under LW)
- Crest rising to an elevation of 1.3 to 1.5 times H_u above HW on the sea side and 0.5 times H_u on the harbour side
- Parapet wall the thickness of which is about $0.75H_u$
- Scour protection extending at least 2.5 H_u in front of the wall, with a minimum of 10 to 15m

This leads to a basic cross-section as sketched in Figure 11.6. This sketch can only be used as a first approximation in a design and its stability must be verified thoroughly for every new application. This can be done with the level 1 method as described in PIANC/MARCOM 28 [2003].

[2] Free height is the distance between the top of the stone berm foundation and the low water level.

12

CONSTRUCTION METHODS FOR GRANULAR MATERIAL

Chapter 12 is meant to give students an impression of the various available construction methods. The chapter requires careful reading and an attempt to understand the background.

12.1. INTRODUCTION

Although for a first approach theoretical and analytical considerations are indispensable to designers of breakwaters, the method of construction is equally important. Based on theoretical considerations, a preliminary design is put on paper. For a breakwater this leads to the details of the cross-section, the crest level, the outer slope, the weight of armour and sub-layers, generally starting the calculation from top down to bottom. For a closure it is the materials needed to cope with the tremendous change in flow-conditions during the progressive stages of horizontal and vertical closure that influence the design. Next, an analysis has to be made of the conditions in all construction phases and of the equipment required to operate in those conditions, from start to finish and from bottom to top. The main problems will be the stability (or vulnerability) of every construction phase, the accessibility of the work front and the safety of the equipment. In order to optimize the feasibility, alternatives have to be generated by making small or even radical changes to the design, in order to find an acceptable compromise between design requirements and the possibilities of the available equipment and materials.

Closure dams and breakwaters have one important aspect in common: their massive character. Construction of the structure requires a tremendous amount of material that must be acquired, for instance from a quarry, transported to the location of the structure, and then placed within the profile along the alignment. This is a logistical problem that covers much more than the handling of the material alone. It concerns opening up the various working sites, mobilizing and maintaining equipment, mobilization and accommodation of personnel, and last but not least, the actual handling of millions of tons of material. Optimizing the solution of this logistical problem may create savings that are much larger than the little extra cost of improving a sub-optimal design of the cross-section.

It is impossible to make a complete description of all options for the construction of closure dams and breakwaters in a book like this. In practice, consultants and contractors have a special and very well documented department to do this kind of work in the tender and pre-tender stages. This book attempts to give students an idea of possibilities and problems. Reference is made to the Rock Manual [2007], where more details are given, and from which some information has been included here.

In this chapter, we follow the materials like sand and blocks from source to destination. Various materials are used to create structural elements, such as the geotextile plus bamboo plus quarry stone used to make a bottom-mattress. They have to be obtained, brought to the site, reworked or stored, transported to the actual site and placed by appropriate equipment in a variety of wave and flow conditions. This mainly concerns:

- Bottom fixation or bottom protection: Mattresses made of geotextiles, branches or bamboo, sunken or placed onto the bottom in open water and ballasted with quarry stone (or alternatively: granular filters)
- Shore-connections and initial dam sections: Sand-fill or quarry-run dams with embankment protection, made by dredging or by dry earth-moving equipment

- Abutments: Sheet-piled walls or a caisson to provide a vertical connection with the large caisson units placed afterwards
- Breakwater core and dumped sills: Quarry stone, “all in” or with selected gradation, transported and positioned by dump trucks or dump vessels
- Cover layers and armour: Selected quarry stone or artificially made concrete units, placed by hydraulic equipment and cranes

Caissons and monolithic structures have such specific problems of execution that they will be treated in a separate chapter (see Chapter 14). A more thorough knowledge is required to satisfy some aspects of the above items. Details of the construction and use of mattresses, the provision and handling of quarry stone, the use of rolling and floating equipment in various conditions and finally some very specific techniques and ancillary equipment are given in the next sections. Some details of construction equipment are given in Appendix 7.

12.2. SCOUR PREVENTION BY MATTRESSES

Every closure of a watercourse leads to a situation in which the flow accelerates, to flow contraction around dam heads and to flow over materials with different hydraulic roughness. Each of these flow changes results in an increase of the capacity to erode. In the sand closure process the scour is accepted since its magnitude is limited because of the restrictions to the flow velocities which offer the possibility to apply the method. For all other methods and dependent on the resistance of the bottom material against erosive action, scour holes may develop, which can endanger the stability of the closure dam. This has to be prevented by the placing of bottom protection means at all relevant locations. These do not prevent the scour completely but shift its bearing towards less vulnerable locations and may reduce the scour depth. Scour prevention therefore is part of most closure processes and generally one of the first actions in practice.

Generally speaking the scour resistance of the bottom material is difficult to predict. Rock and stiff clay will be very resistant, soft clay is rather resistant; peat may stand the attack quite long and then suddenly break out in large lumps. The behaviour of sand has been investigated intensively and several formulae have been derived to predict the scour hole development. Since in practice a sandy subsoil is but seldom homogeneous, the actual scour may still deviate from the predicted values.

In short, scour holes can be expected at places where:

1. the flow velocity increases in course of time
2. the flow distribution over the vertical changes
3. the flow is not saturated with sediment
4. the turbulence intensity increases

These aspects occur in closure processes for instance:

- when diminishing the gaps profile (item 1)
- at the end of a stone protection as consequence of change in roughness (item 2)
- due to reduction of the discharge quantity in the approach gulley (item 3)
- around dam heads, structures and obstacles (item 4)

Of course combinations of these four often occur.

In one-directional flow, the scouring process creates a hole which is characterized by its steep slope at the upstream side, its depth and its gentle downstream slope. In tidal areas, where the flow changes direction in every tidal cycle, the shape of the hole will be different. The reversing flow smoothes the hole out slightly by which the slopes become more gentle. The development of the hole goes quickly in the beginning but gradually slows down. By creation of the hole, the bottom topography adapts itself to the flow’s eroding capacity and in the end an equilibrium state is reached. The depth of the scour hole develops in an exponential relation with time. In many cases the equilibrium state is reached so quickly that the intermediate stages are of no importance. However, if a number of caissons are placed one after the other in a couple of weeks’ time, the flow pattern changes stepwise in short periods and so does the scouring capacity. The scour hole then develops as a summation of intermediate successive stages (see Figure 12.1).

The development of scour holes in itself is not dangerous. Only in cases where they come too close to either the closure dam or adjoining dams or structures do they endanger the stability of these structures.

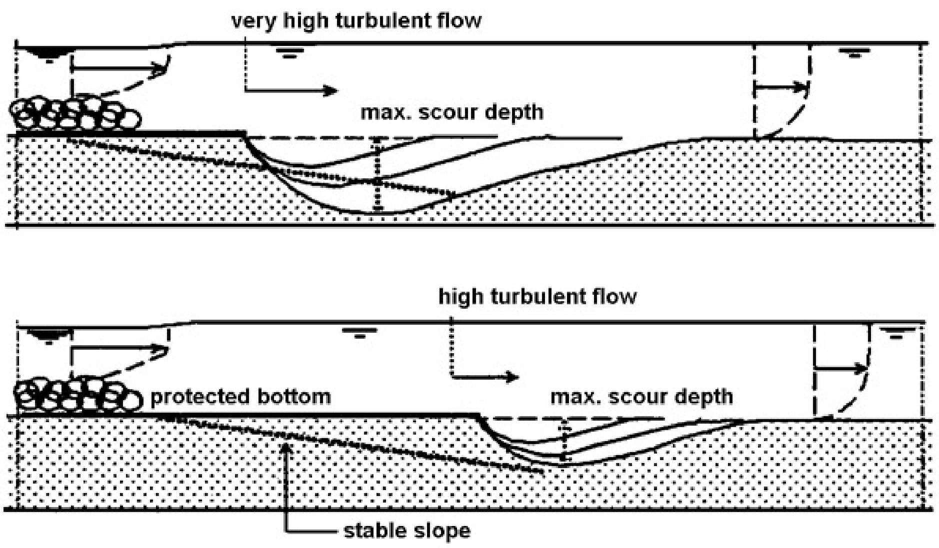


Figure 12.1: Development of scour hole

Then uncontrolled scour should be prevented. A scour protection by a bottom mattress or a filter layer will be required. Since the costs of these protections in closure works generally are considerable, minimizing the dimension is important. However, the installation has to be done in advance of the determining situation (construction phasing) and a too short protection may give a large risk. The longer the protected area, the further away the hole develops and since on that spot the attack will be less, for instance because of spreading of the flow or diminishing of the turbulence intensity, the equilibrium depth of the hole will be less. Both aspects, further away and lesser depth, improve the stability consideration of the endangered structure. Usually, as a first approximation, a protected length of about 10 times the water depth is considered safe. For detailed engineering in case large costs are involved, the optimization requires physical model investigation in a hydraulic laboratory.

For the stability considerations the dam in the closure gap and the joining bottom protection act as a total structure. Consequently groundwater flows and potential head differences will build up over this protection. Therefore the protection has to:

- be flexible to follow changes in bottom topography
- be well connected to the bottom, leaving no room for piping
- be sufficiently heavy to prevent flapping in the flow
- be extra ballasted at its end to prevent turning up when the tide turns, be impermeable for the soil material underneath
- be stable in all flow conditions in all relevant construction phases
- be permeable for water to prevent high pressures underneath (sometimes the requirement for an impermeable part is combined with the scour protection, in spite of the pressure increase).

12.3. CONSTRUCTION AND USE OF MATTRESSES

As soon as artificial structures for breakwater or dam construction disturb the governing flow and wave attack, fixation of the bottom by taking protective measures becomes necessary. A granular filter can be used but sometimes more coherent structural materials are required. The structure has to cover a rather large area and needs to be sand-tight but permeable to water. Moreover, it has to be stable under all prevailing flow and wave conditions.

Usually, mattresses, which are floated to the site or rolled onto a sinkable cylinder, then stretched out onto the bottom and subsequently ballasted, are used. Each mattress overlaps its predecessor. In the old days thick willow mattresses were the only suitable structures. However, since the introduction of geotextiles, adapted structural designs are used. The sand-tightness, formerly obtained by the thick layer of osiers can now be acquired by using a geotextile sheet.

This modern version of bottom protection is based on splitting up the functions:

1. Strength is obtained by using geotextile of the desired material
2. Sand-tightness is obtained by using geotextile of the correct mesh
3. Floatation is obtained by adding willow or bamboo bundles (as the old version)
4. Rigidity during the sinking operation is obtained by adding one willow or bamboo grating (as compared to two in the old version)

The items 1 and 2 are usually combined in one sheet, but sometimes, when the bottom material is very fine grained (silty), a double sheet is required that consists of a non-woven (felt type) sheet and a strong woven sheet.

The items 3 and 4 are not required for a sheet, which is unrolled from a cylinder straight onto the bottom.

In all cases a considerable quantity of ballast is needed to fix the sheet in position. This is usually provided after the sheet has been initially ballasted during the placement operation. Initial ballasting is used to sink the sheet when the protection is floated in place. In addition, this keeps the sunken or unrolled sheet on the bottom after sinking. This initial ballast has to be immediately followed by part of the final ballast to secure the protection against the next flow attack.

The final ballast should remain stable on the sheet and not roll or slide away. This is not only a matter of the size of the ballast material (Shields) but also a matter of friction. A bottom protection, generally drawn nicely horizontally on the design-drawings may in practice cover an undulating or sloping area. The above mentioned items 3 and 4 therefore have another important function in providing fixation for the ballast material. This is a complication for the unrolling system. Though the sheet is thin and can simply be rolled-up, some sort of structure has to be added to provide resistance. This is usually achieved by the mechanical fixation of the initial ballast to the sheet (by pins or glue) before it is rolled up onto the cylinder. Such a cylinder is therefore large and heavy.

A very special type of bottom protection was designed for the Eastern Scheldt Storm Surge Barrier. The protection used in this case had to be of the granular-filter type and needed to be laid in a single operation per mattress. The three-layer filter was wrapped up in geotextile sheets that were kept in the correct shape by a steel mesh (like gabions). Special equipment was built to handle these mats. This type of mattress was chosen for that part of the sill where loss of bottom material would cause structural failure. Geotextile alone was not considered safe enough over the design life time of 200 years.

Mattress-type of scour protection has a number of advantages and disadvantages when compared with granular filters. Advantages are:

- Limited construction height
- Applicable on steep slopes

Disadvantages are:

- Difficult to remove
- Presence of structural joints that tend to be vulnerable
- Vulnerable for mechanical damage
- Life time is restricted (ultra violet radiation for geotextiles, pile-worm (teredo) for willow or bamboo)

12.4. CONSTRUCTION OF GRANULAR FILTERS

The use of granular filters is more common in breakwaters than in closure dams. This is a consequence of the different working conditions that are present. Breakwaters are generally constructed in places with a relatively flat bottom, whereas a scour protection for a closure dam will often be extended along the steep slopes of the gullies. The material that is meant to form the granular filter tends to roll down the slope in that case.

The granular filter is composed of several layers of filter material, of which the finest material is used to provide a sand-tight filter over the virgin bottom material. Subsequent layers are stepwise coarser to prevent underlying layers from washing out, until such grain size is obtained that the upper layer can withstand the external forces by waves and currents.

The construction of such filter is complicated since the filter has to be built up layer after layer. To ensure the required sand-tightness, it is unacceptable that layers are interrupted locally. The presence of each (designed) layer must be guaranteed and proved by the contractor. In view of the tolerances on one hand and the measuring accuracy on the other hand, a layer thickness of at least 0.5 m or $2d_{n50}$ is recommended when the filter material is barge-dumped.

The dumping itself is done by barges that are able to produce a controlled flow of material. These are either split barges (for bulk placing, for instance the core of the dam or breakwater) or side dumping barges that create a sort of curtain of falling material (for placing relatively thin rock layers). By hauling the side dumping barges slowly sideways, an even layer can be applied. It is necessary to control both, the discharge of material and the lateral velocity of the barge. This control is often easier at breakwater locations than in potential closure locations, where the current velocities tend to be higher in the construction stage.

The advantages of a granular filter are:

- Self-healing after minor damage (for instance by a dropped anchor)
- Absence of structural joints
- Simple to be removed (dredging)
- Towards the end of the area to be protected, they can be faded out gradually

Disadvantages are:

- Absence of structural coherence, they disintegrate on steep slopes
- The total construction height is considerable because of the minimum thickness per layer and the large number of layers

12.5. PROVIDING AND HANDLING OF QUARRY STONE

It has been indicated earlier which data must be collected before can be decided to open a quarry in a particular rock formation. Before starting the actual quarrying operations, some requirements must be met:

- Access must be ascertained
- Required permits must be available
- Protective measures against damage to human interests and ecology must be operational
- Accommodation must be available for personnel
- Maintenance facilities for equipment must be available
- Supplies of fuel, spare parts, explosives, etc. must have been arranged

The planning of the quarrying operation is mainly based on the expected fragmentation curve. The blasting and quarrying must be done systematically, following a pre-determined mining plan. During the blasting, bench floors that can be used for sorting the material, loading and transport are created. The width of the bench floor must provide adequate working space for these purposes.

In most cases, it will be difficult to obtain the larger fractions. In the beginning, this does not appear to be a problem, since in the first phases of the project, only finer material for filters and core is used. The larger fractions are required in the later phases of the project, when armour layers and breakwater heads are under construction. Then, however, it is too late to modify the blasting scheme and to obtain the required percentage of armourstone. It is therefore recommended that larger fractions should be produced right from the start of the operation, and efforts to obtain these should not be postponed to the last stage of quarrying.

The consequence is that all stone gradations must be sorted and stored separately, right from the beginning, even if some categories of stone are not required until later. This leads automatically to the need for a large stockpile area where stone can be stored until it is used. The dimensions of the stockpile are directly related to the quantity of armourstone, and the size of the stockpile area relates to the size of each grading. In restricted areas stockpiles may rise to considerable heights. Generally, dump trucks can drive on stockpiled material if the gradings are smaller than 5–40 kg. In such cases the height of the stockpile is determined by:

- the gradient of the access road to the stockpile, maximum slopes being approximately 1:15
- segregation – the problem of bigger stones rolling down the slope can be eliminated by building up the stockpile in 4–5 m layers
- windblown dust in exposed areas (the nuisance can be reduced by spraying with water) and other environmental impacts
- subsoil – bearing capacity and stability.

Stockpiles of gradings above 10–60 kg can only be as high as the reach of the available wheel loader or hydraulic excavator, which is typically 3–3.5 m for wheel loaders and 4.5–5.5 m for excavators. To avoid cross-contamination between different grades of material, sufficient space and/or partition screens are used to separate the various stockpiles. If light and heavy gradings are placed next to each other the difference in

Figure 12.2: Stockpile area with wide gradings

the size of the materials will be obvious and any mixing will be noticed immediately. An example of a well-organized stockpile area is shown in Figure 12.2.

It is recommended that the classification of stone in the quarry is facilitated by providing sample stones per category and by frequently using a weighbridge to check the weight of sample stones. More details about the quarry are given in Appendix 2.

The transport of material from the quarry to the work site can be done in four different ways:

- By road
- By rail
- By water (either inland or sea)
- By a combination of methods

It is impossible to indicate a preferable mode of transport as the choice depends on local conditions, available facilities and the extra investments required. In general, transport by water is far cheaper (4 to 5 times per ton kilometre) than transport by road or rail. This is valid only if a waterway of sufficient width and depth is available, or can be made available at little extra cost. Rehandling of material due to change of transport mode will create extra cost, which must be compared with the savings.

It is not certain that delays in the transport chain will coincide with delays in the quarry operation. This is a second reason to provide a stockpile area at the quarry site that has sufficient capacity to cope with irregularities in the production and delivery of stone.

12.6. USE OF ROLLING AND FLOATING EQUIPMENT

At the location of the dam or the breakwater, a relatively large construction yard is required. Space has to be provided for offices, accommodation, workshops, etc. In general, a stockpile for quarry stone and other construction materials is also required to act as a buffer when supply and discharge are not in balance. When concrete armour units are used, a concrete mixing plant is required as well as a block casting area and a storage area for the armour units. In principle, there are three methods to bring the material into the designed profile:

- By floating equipment
- By rolling equipment
- By a combination of both

It is evident that for detached breakwaters or for dam sections not yet connected to the mainland floating equipment or transhipment is the most logical choice.

With regard to the method of construction, the major differences between breakwater construction and closure dam construction are caused by the differences in the cross-section of the profile to be made. A breakwater generally consists of a core covered by several layers of different materials of various sizes. The cross-section of a closure dam is much more uniform, and typically, cover layers are not required. However, different materials may be used in the longitudinal profile for different dam sections because of the increasing flow velocities at the work front when the gap narrows. Different longitudinal cross-sections may also be required for the breakwater, usually because the exposure is in a different wave-environment (for instance in deeper water or near the breakwater head).

The main problem of breakwater construction is to build out core and cover layers consecutively in such a manner that every part which is not yet stabilized by its cover is not damaged by the environmental conditions (weather) during construction. All damage, which occurs during construction, has to be repaired according to the prescribed layer profile, as the functioning of the breakwater depends on the filter-design rules. Therefore, it is necessary to consider tolerances and to maintain a very strict position control during the construction of the breakwater.

The term tolerance relates to the extent of deviation from the ideal that can be accepted or tolerated. Different definitions can be put forward based upon the following criteria:

- *What is possible*: Virtually anything is possible, but sometimes this can lead to great expense, take a long time and lead to the use of over-sophisticated methods.
- *What is required*: Since the technology exists to construct to very small tolerances, the specified requirements may reflect this and over-emphasise certain aspects of the works.
- *What is necessary*: Specified tolerances should reflect what is necessary for the structure to perform its designed function.
- *What is affordable*: The effect of tolerances on economic considerations can be profound. Often accepting a standard of finish that is functional rather than precise can lead to savings without which the construction may not be viable.

The setting of tolerances and the scale of deviations from the prescribed profile requires a careful balance of the above factors. The acceptable tolerances for armourstone placement are determined primarily by the functional requirements of the structure so the strictness with which they are applied may vary. These requirements relate to:

- stability of the structure, e.g. currents and waves
- smoothness of the filter layer
- guaranteed navigation depth in the case of bed protection works
- visual aspects.

Environmental conditions (storm surge) may also damage a closure dam. Although the repair may require extra material to compensate for the loss, construction generally simply continues on top of the remains. In some cases continuation implies a completely different design. For instance a planned vertical closure may have to be completed horizontally or by using an obsolete vessel as a caisson. Basically, there is no need to keep to the original design, as long as the gap gets closed. There, strict adherence to the tolerance and positions is not as essential as it is in breakwater constructions.

Several aspects of using rolling equipment are identical for breakwaters and dams. However, breakwater construction, with its layers, is the more complex operation. For floating equipment, there is a distinct difference because of the environmental conditions. For breakwaters, the main problem is operating in wave conditions: “how to place the material dangling from a moving floating crane, on the right spot”. For closure dams, the flow characteristics determine the operations: “how to keep position or anchor in the flow or how to operate in the short moment of turning tides”. Of course, in some cases waves and flow may occur at the same time.

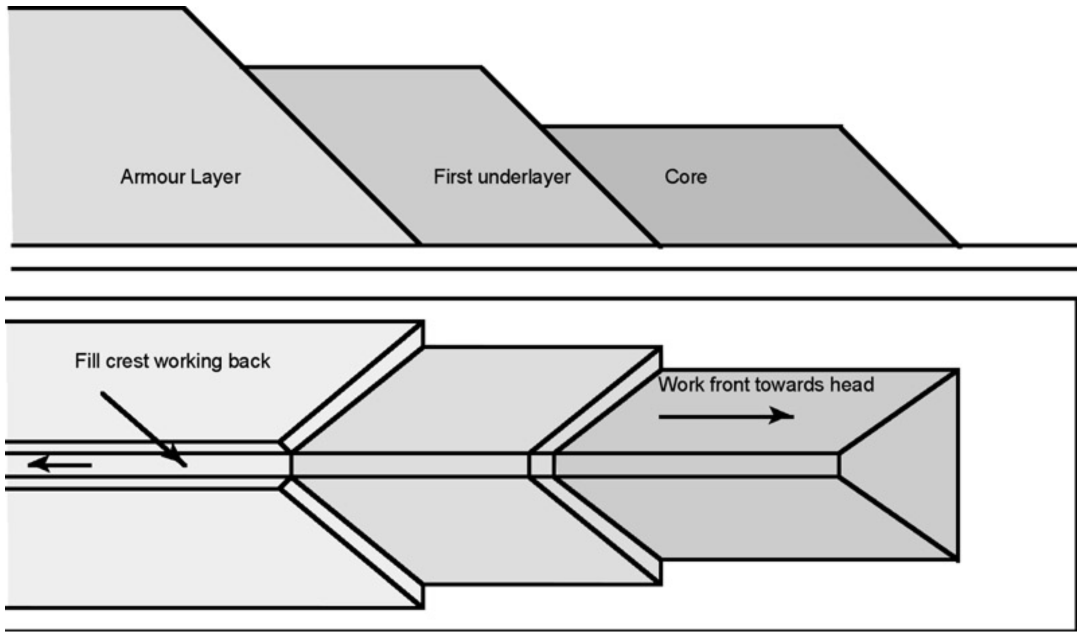


Figure 12.3: Subsequent work fronts

ROLLING EQUIPMENT

If rolling equipment is used, a dam is built out with a work front in several phases, for filters, core material, first under-layer, toe, etc. The crest of the dam is used as main supply road. It has a minimum width of 4m for one lane traffic. For two-lane traffic the crest width must be at least 7m. As an alternative, one can create passing places. Since it is virtually impossible to drive over the armour units (they are too large), the access road to the work front is often created on the crest of the core or at the crest of the first sub-layer. The level of this crest must be high enough above HW to guarantee the safety of equipment and personnel working there.

In this way, the full length can be constructed according to the design, except for the armour units on the crest. These units can be placed in the final stage, working backwards from head to mainland (see Figure 12.3). A picture of such breakwater under construction is given in Figure 12.4.

If a concrete crest block is used instead of a crest of loose armour units, one may position this cap-block building out the dam. The cap block can than be used advantageously to provide a better quality access road to the work front.

The filters and the core are often placed by bulk dumping. The armour units are generally placed individually by crane to avoid the risk of breakage or misplacement. If placing is done with hydraulic excavators provided with orange-peel or cactus grabs, or power forks then armourstone can be placed to a desired position and orientation. A tight packing density is possible in this way, which is important for the stability of the rocks.

The method used to place the first under layer depends on its size and the local conditions. For the material that is placed individually, nowadays the crane is fitted with electronic positioning equipment to place the units in the pattern prescribed in the specifications

Guidance of achievable vertical placing tolerances with land-based rolling equipment (cranes) is presented in Table 12.1.

A disadvantage of the dry construction method is the fact that all material must be transported over a rather primitive road with limited capacity (see Figure 12.5). This becomes ever more important as the length of the dam increases. When the crane at the work front prevents direct dumping, one may consider the use of a gantry crane. The required width and height of the work-road over the crest may lead to a much bulkier design.

The major advantage of the dry construction method is the potential use of cheap local equipment (see Figure 12.6) and the independence of working conditions at sea (fog, waves, swell, and currents). For execution of a closure, obviously, rolling equipment can only be used for the horizontal method or for finishing a combined closure horizontally. The crest height of the closure profile is taken at a storm safe level. The material size is identically for the full profile as the flow attack is at the work front, over the full height. In order

Figure 12.4: A breakwater under construction

Figure 12.5: Truck waiting on the breakwater

Depth of placing relative to LW	Bulk-placed armourstone		Armour layers and individually placed stones, with $M_{em} > 300$ kg	
	$M_{em} < 300$ kg	$M_{em} > 300$ kg (not armour)		$M_{em} < 300$ kg
Above LW = dry	±0.2 m	+0.4 m to -0.2 m	+/- $0.3d_{n50}$	+0.35 to $-0.25d_{n50}$
0 to -5 m	+0.5 m to -0.3 m	+0.8 m to -0.3 m	+/- $0.5d_{n50}$	+0.60 to $-0.40d_{n50}$
-5 to -15 m		+1.2 m to -0.4 m		
Below -15 m		+1.5 m to -0.5 m		

Table 12.1: Practical, achievable vertical placing tolerances with land based equipment [Rock Manual, 2007]

Figure 12.6: Use of cheap local equipment

to improve access for the rolling equipment, the crest layer is filled-out with small size material; a possibility that does not exist for the breakwater as it would disturb the layer characteristics. In exceptional cases the method is used on breakwaters, on condition that the fines are later removed.

FLOATING EQUIPMENT

When floating equipment is to be used, a work harbour is required from the start of the operations. In this harbour, the barges can be loaded. Side-dumping vessels may be used to place filter layers of a limited thickness. Bulk dumping can be used for the construction of the core, with bottom door barges, split barges, and tilt barges or flat deck barges and with bulldozers pushing the material over board. Intermediate layers may be applied along the slopes by using side-unloading barges.

As soon as the structure reaches a level higher than 3 to 4 meter below HW, the use of these barges and vessels becomes impossible. If one continues to use floating equipment, floating cranes or crane platforms are needed to finish the upper part of the profile. The use of cranes may also be necessary to trim the slopes of the core that are constructed in bulk dumping operations.

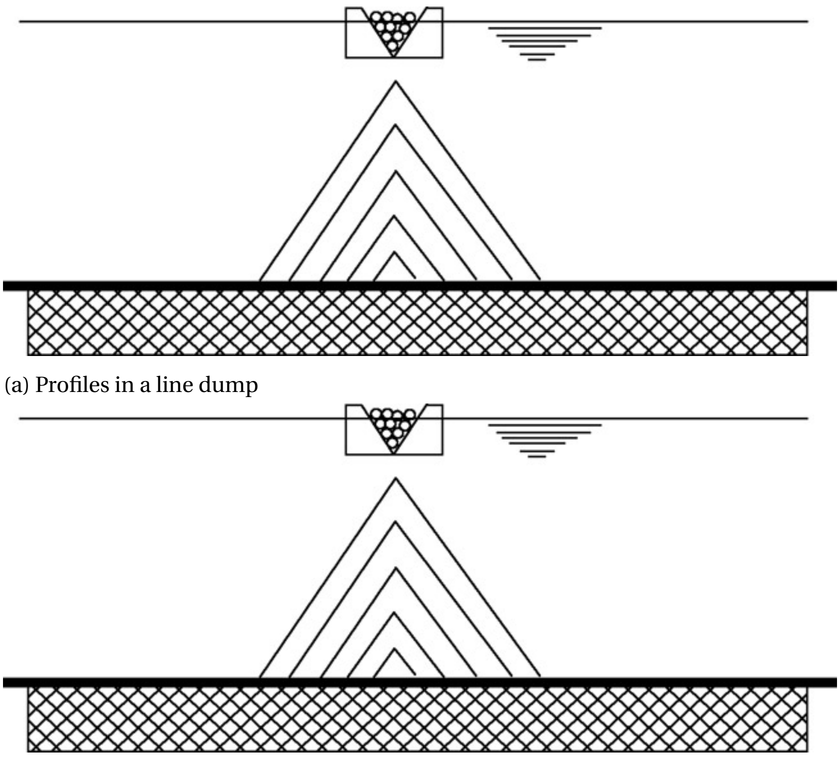
The main advantage of the “wet” construction method is the possibility to start construction at more than one work front, or to build detached breakwaters. Another advantage is that one can bring in the greater part of the material independently of the limited transport capacity over the crest. When material is transported by barge from the quarry to the site, it is an advantage to use the supplied material without stockpiling.

A disadvantage is the dependence on working conditions at sea and the need for a working harbour right from the beginning of the works.

Guidance of achievable vertical placing tolerances with floating equipment (side-dumping vessels and

Grading	Side stone-dumping vessel and crane barge	
	Individual	Bulk
Coarse	N/A	+/- 0.20 m
Light	N/A	+/- $1d_{n50}$
Heavy:		
300–1000 kg	+/- $0.8d_{n50}$	+/- $1d_{n50}$
> 1000 kg	+/- $0.8d_{n50}$	N/A

Table 12.2: Practical, achievable vertical placing tolerances with floating equipment [Rock Manual, 2007]



(a) Profiles in a line dump

(b) Profiles in horizontal layers

Figure 12.7: Dump profiles

crane barges) is presented Table 12.2.

For closure dams, floating cranes can be used for horizontal closure by positioning them at the down-stream side of the closure gap, in the area protected from the flow behind the dam head.

The side slopes thus depend on the individual dimension of every successive layer (Figure 12.7). This is an important stabilizing factor for the flow crossing over the sill. This may also be an advantage from the point of view of soil mechanics.

COMBINATION OF FLOATING AND ROLLING EQUIPMENT

A construction method in which dry and wet equipment is used in combination is often preferred. The main reason is that generally the unit transport cost over water is lower than by dump truck. As the largest part of the cross-section is the lower part of the profile, dump vessels are used for the bulk of the material in these sections. This is specifically true for deep-water dams, as dumping will not reach higher than about 4 metres below high-water level. The profile accuracy is low; both for the finished level and the width and for a breakwater, some reprofiling of the side slopes will have to be done later. This can be done, simultaneously with the placing of the next layer for stabilization, by a crane operating from above water, after further tipping by dump trucks has provided access (see Figure 12.8).

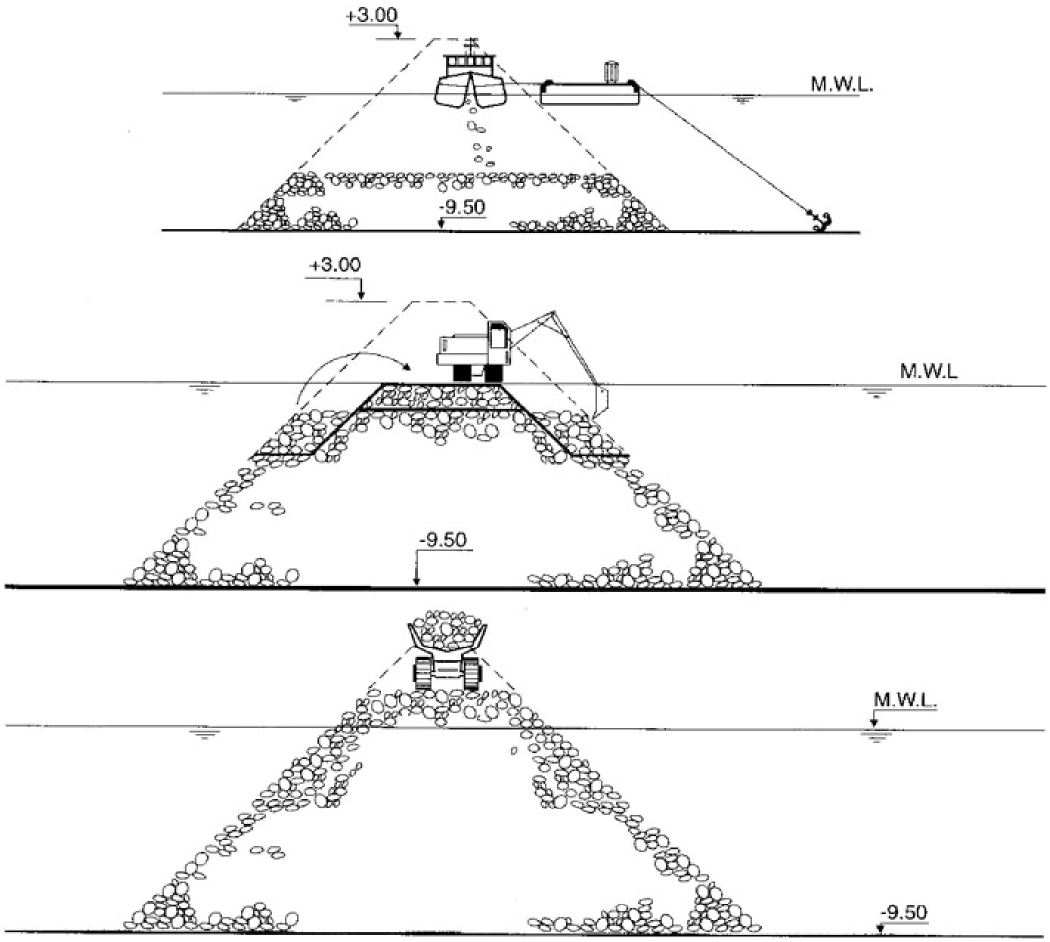


Figure 12.8: Construction of a closure dam by successively using waterborne equipment, grab and dump truck

In the combined method of closure dam construction, the dumping seldom creates a sill sufficiently high to reach the critical flow stage. But even if this is so, the horizontal continuation of the closure on top of it may change the levels such that the flow will change back to a sub-critical flow. In all cases, the flow velocity in the last part of the gap will be much higher than that occurring during the dumping process of the top layer. This layer therefore needs much heavier units (stone), certainly in the centre section of the gap, than when the vertical closure system is continued.

12.7. VERY SPECIFIC TECHNIQUES AND ANCILLARY EQUIPMENT

SPECIFIC CONSTRUCTION PERIOD

Sometimes it is possible to reduce the risks by limiting the construction period to the summer season (or to a particular monsoon season). If the whole structure can be completed in this calm period, the risk can be much smaller. Otherwise it may be possible to interrupt construction during the rough season. When the work fronts are well protected, it will be possible to resume construction the next year or season.

REDUCE EXPOSED LENGTH

In other cases, an option may be to keep the distance between the work fronts as small as possible. If damage occurs, it will be restricted to a small stretch of the structure. Since the contractor is still present with all his equipment, repair of the short damaged section need not to pose great difficulties.

PROTECTIVE BUNDS

Instead of the work sequence as sketched in Figure 12.9(left), it is possible to dump the first under-layer before the core material. This method can be compared with the construction of reclamation bunds around

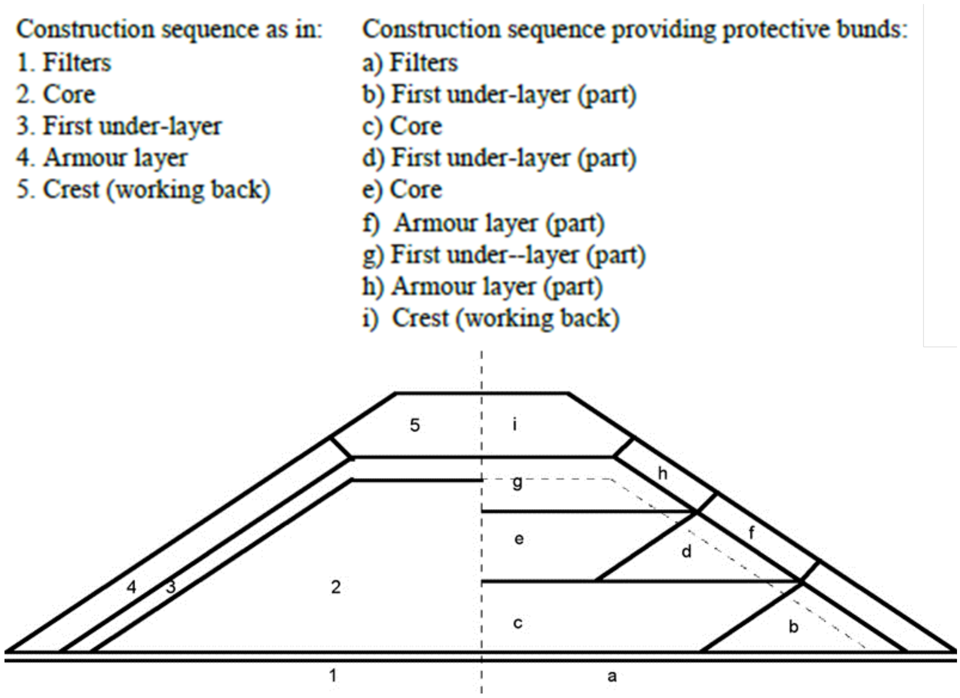


Figure 12.9: Varying construction sequence

a confined reclamation area in dredging. The disadvantage of the method is that more material is required for the first under-layer than strictly necessary on the basis of the theoretical two-layer design. Figure 4-85, the traditional method is compared with the alternative.

It is clear that the alternative method (on the right hand side of Figure 12.9) provides a better protection of the core material during construction phases. It is also clear at the same time that it requires more expensive first under-layer material and that the construction method is a little more complicated. Depending on the availability and cost of material versus the cost of handling, one can save some of the extra material by double handling.

12.8. SURVEY

Because of the direct relationship between survey techniques and payments, all parties to a works contract should ensure that an accurate, fair and pragmatic approach to surveying is adopted that will lead to the correct method of payment for the work done. To suit the requirements of the works, tolerance levels should be practical, sensible, achievable and affordable (see for definition of tolerance Section 13.6).

The most commonly used survey methods are briefly described below (see also the Rock Manual [2007]:

ABOVE WATER

Coarse and light armourstone gradings can be measured by using a probe with a spherical end of diameter $0.5d_{n50}$, which for a land-based survey will be connected to a staff, GPS antenna or EDM target. Measurements are generally carried out at intervals of between 1 m and 2 m across the measurement profile.

Heavy gradings are mostly measured by means of a staff linked to a GPS antenna or EDM target probe, which for a land-based survey will generally be connected to a staff or EDM target. For individually placed double-layered systems of armour, three different survey methods can be used (see Figure 12.10):

- highest points
- spherical foot staff
- conventional staff.

As can be seen in Figure 12.10, each method results in a different measurement of the layer thickness. Consequently, it is essential that before construction starts there should be agreement between the client and the contractor which method will be used.

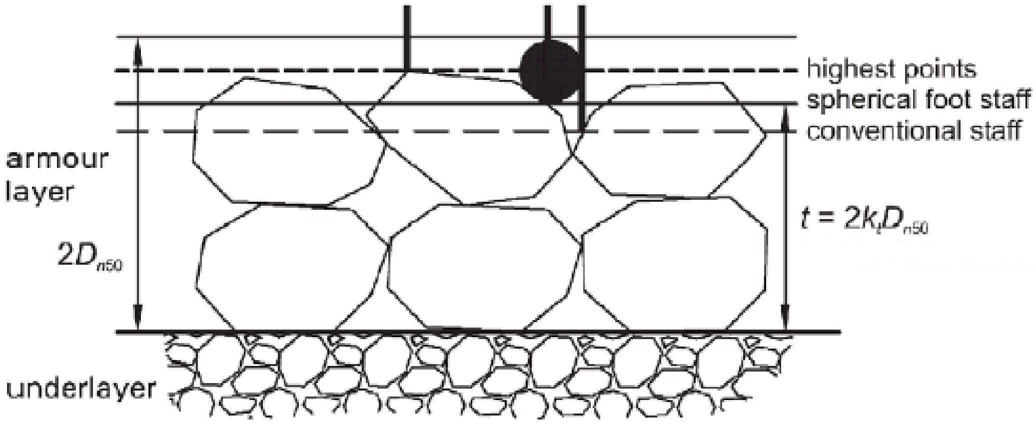


Figure 12.10: Effect of surveying methods on layer thickness for double armour layer

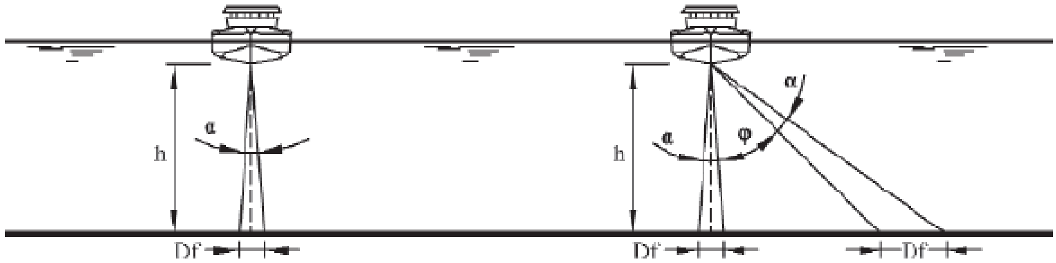


Figure 12.11: Footprint of single beam and multi-beam echo sounders [Rotterdam PWED, 2001]

Measurement profiles will be done generally with 10 m intervals along the breakwater, but may need to be more frequent where the profile is changing rapidly or on tight-radius curves. It is common practice in breakwater construction that a rock layer will be placed only after approval by the client/designer of the profile of the former layer.

UNDER WATER

Structural parts that are below the waterline can be surveyed by using a weighted ball on the end of a sounding chain. If they are too deep, surveys can only be completed by using echo sounders or side-scan sonar.

Echo sounders measure the water depth by determining the difference in time between the moment of sending the sound signal and the moment of receiving the signal after reflection from the sea bed. Using a preset value for the speed of sound under water, v_s (m/s), and the measured time interval, Δt (s), the water depth, h (m), can be calculated as:

$$h = 0.5 v_s \Delta t \tag{12.1}$$

There are two main echo sounder systems:

- single-beam
- multi-beam.

Single-beam systems make use of one sound beam so that only the sea bed directly underneath the survey vessel is measured. The circular section of the sea bed measured is called the footprint (see Figure 12.11). The diameter of the footprint, d_f (m), depends on the beam angle α (deg), and the water depth h (m), according to:

$$d_f = 2h \tan(0.5\alpha) \tag{12.2}$$

The beam angle (α) differs according to the system frequency but is in the range of 2.5 - 3.0^o.

Multi-beam systems use an array of sound beams allowing a line of points to be measured in one measurement sequence. This line of measurements is underneath and to both sides of the vessel but can be directed to one side if necessary (see Figure 12.11). The values of the sound/time measurements are calculated to depth values by the system software. This software is primarily designed for smooth surfaces. When rough or hard surfaces (i.e. those with bigger armourstones) are being measured, acoustic disturbances will occur, disrupting the processing of the sound beams. This can lead to systematic errors.

Multi-beam sound beams have a footprint that in many cases is smaller than that of single-beam systems. The beam angles α are in the order of 0.5–1.5^o. Towards the sides the footprint increases when further away from the vessel (see Figure 12.11).

The diameter, d_f (m), of the oval-shaped footprint is given below:

$$d_f = 2h\left[\tan(\phi + 0.5\alpha) - \tan\phi\right] \tag{12.3}$$

where ϕ is the direction of the beam relative to the vertical (deg) and h is the height difference between footprint and the ship's bottom (m).

In many systems ϕ can be varied in the range of -75^o to +75^o in steps of α = 0.5^o to 1.5^o. The size of the footprint furthest away from the survey vessel might be five times the footprint underneath the vessel.

In view of the relationship between measurement and payment, it is clear that measurement inaccuracies have a significant effect. Background information regarding the origin of measurement inaccuracies of single-beam and multi-beam echo sounding systems is essential.

Measurements may contain two types of errors:

- systematic errors
- random errors.

A systematic error will result in all measurements being biased to one side, either too low or too high. Random errors will cause the measurements to vary within a certain bandwidth, the average level being equal to the true value. An example of a systematic error made when surveying rock works is that which emerges from the penetration of the measurement system into layers consisting of large stones. The average footprint levels will be lower than the top of the stones. This problem will not occur with coarse gradings, so a relationship exists between accuracy, armourstone grading and beam width.

The number of measurements per unit of area is also important. Using a levelling staff and sphere will provide only scattered spot measurements. Although a single-beam echo sounder will deliver continuous profiles these are still separated by the distance between the survey lines. Measurements made with a multi-beam echo sounder provide full coverage of the survey area.

Other influences on the accuracy of the measurements are:

- errors in the positioning of the survey vessel
- errors in the depth measurement (wrong speed of sound setting of the echo sounder)
- poor or incomplete calibration of the system
- poor compensation of movement of the survey vessel
- inaccuracies arising from the system itself in relation to the measurement surface (smooth or rough, horizontal or sloping)
- the experience of the personnel.

These are mainly random errors and should not affect the average level.

13

CONSTUCTION METHODS FOR MONOLITHIC STRUCTURES

Like Chapter 12, this chapter gives a description of practical problems encountered when building monolithic breakwaters. It should be read as an example rather than be considered as absolute facts.

13.1. INTRODUCTION

In breakwater constructions it can be an advantage or even essential to use very large monolithic elements. As far as breakwaters are concerned, the monoliths may consist of loose elements that are assembled to form the final monolithic structure. Caissons have been widely used in monolithic breakwaters.

13.2. MONOLITHIC BREAKWATERS

13.2.1. MONOLITHIC BREAKWATERS CONSTRUCTED BY ASSEMBLING SMALL UNITS

These blocks were sawn in the quarry and placed in the breakwater according to a pattern comparable with the present brickwork techniques. The blocks were connected with dowels to ensure the monolithic behaviour of the structure.

This technique is still used, although the blocks are often cast in concrete nowadays, and steel reinforcement and cement mortars are used to connect them (Figure 13.1 and Figure 13.2)

13.2.2. MONOLITHIC BREAKWATERS CONSISTING OF LARGE UNITS CONSTRUCTED IN-SITU

The most common example of in-situ construction of large monolithic units is the construction of sheet pile cells. The main problem in constructing this type of structure is the closure of the slots between the individual piles. The workability during pile driving may also cause problems.

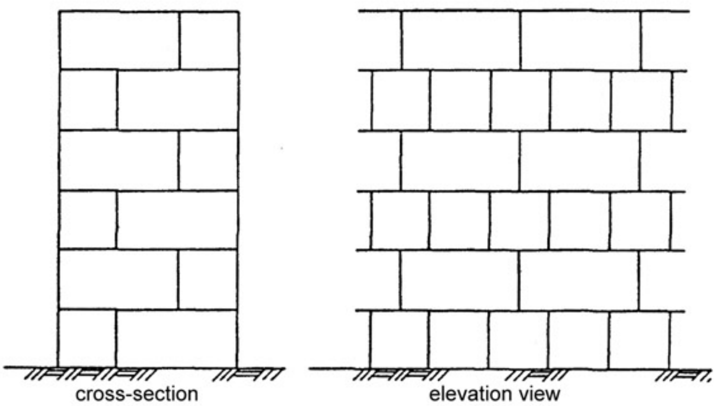


Figure 13.1: Typical block wall

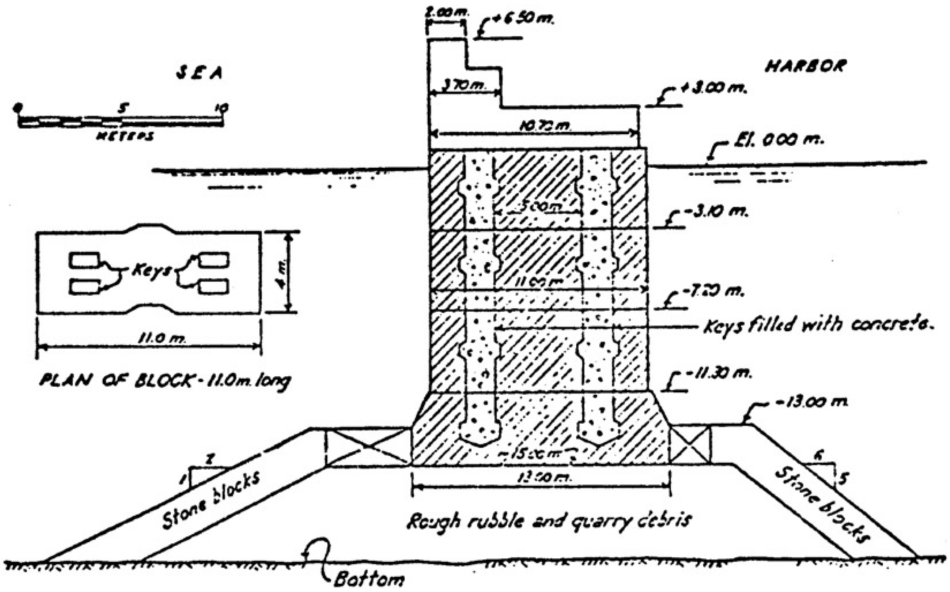


Figure 13.2: Breakwater Algiers, Algeria

The cells are filled with soil or stones. It must be assumed that owing to overtopping and spray, the fill material is saturated with water over its full height. Depending on the type of fill material, cyclic loading and wave impact forces may cause liquefaction of the fill material, which results in relatively high ground pressures on the sheet piles. In such cases, poor connections between the piles cause a serious complication.

13.2.3. PREFABRICATED LARGE UNITS

Prefabricated large units can be transported in different ways. The most elementary way is to use the buoyancy of the elements. In that case, we speak of caisson type structures. Because of their specific importance for both dams and breakwaters, they are treated in a separate Section 13.3.

However, it is not necessary that the prefabricated unit is fitted with a watertight bottom. It is possible to place circular or rectangular rings on a foundation bed and fill them with quarry run to act as a monolithic breakwater. The units can then be brought into place by using separate floats or lift barges. It is also possible to roll them out over the crest of the placed units and lower them into position with a gantry crane. This method was used in Hanstholm (Denmark) and Brighton (see Figure 13.3).

13.3. CAISSONS

13.3.1. BUILDING YARD

It is possible to construct caissons in different ways. The main differences lie in whether construction is completed in the dry, or only the bases of the caissons are constructed in the dry. In this case when their buoyancy is sufficient they can be launched and construction can be completed in a floating condition.

Whether only the lower part is constructed in the dry or the whole caisson makes little difference to the initial stage of construction. The construction yard must be kept dry until the structures are ready to float. For this purpose, one may use the following facilities:

- Dry-dock at a shipyard
- Slipway
- Lift deck
- Dredged special purpose dock

The first three facilities are common features of a shipyard. They can be used if available at an affordable cost. The disadvantage may be that the space is too small to construct the required number of caissons in a limited period. Then, one may consider floating out the caissons long before they are completed and finishing the upper part and the superstructure in the floating condition.

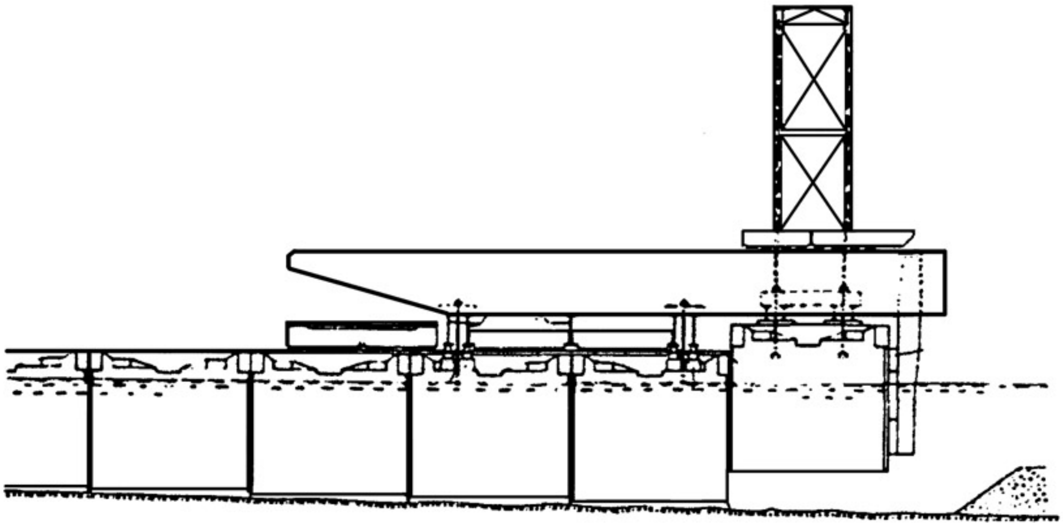


Figure 13.3: Construction method Brighton breakwater

Figure 13.4: Dredged dock for construction of caissons for the Brouwershavense Gat closure

The last option, a specially dredged large dock is a common solution in the Netherlands. The dock is kept dry by deep wells, and the closing dike can easily be breached by a dredge when the construction of the caissons has been completed. All caissons for the closures in the Delta project were constructed this way (Figure 13.4).

The advantage of such a special purpose dock is that it can be built close to the site where the dam is to be constructed. Moreover, the size of the dock can be made to comply with the specific requirements.

The construction site for the building of caissons is fitted out like any construction site for a large concrete structure.

13.3.2. Transport

After completion of the caissons, they have to be transported to the site of the dam or breakwater. It is essential that sufficient depth and keel clearance is available throughout the route from the dock to the site. Tugboats with sufficient power to overcome currents and to maintain a reasonable speed tow the caissons.

The force required to attain a specific speed depends on three parameters. One is the submerged cross-section of the caisson in the tow-direction and its drag coefficient. Next the friction force along the bottom and the sides, in fact the length of the caisson, has an influence. Third, the motions in the water that are caused by turbulence and waves have an impact on the speed. The acceleration of such a large mass requires a large force. Generally speaking, the drag coefficient for caissons will be in the order of about 4. It should be kept in mind that the pulling force is determined by the square of the relative velocity of the caisson in relation to the water. To calculate the actual speed over the ground, the relative velocity has to be compensated for the direction and velocity of the water flow. To generate the pulling force, often tugboats are used. As a rule of thumb, about 10 HP is required for every kN pulling force (1 HP (horsepower) = 0.75kW).

Finally, apart from the force in the tow direction, it is necessary to have lateral control to counteract yawing and power to slacken the tow. In consequence of these requirements, the total power needed is double the power calculated above. The worst case, is probably the moment when the caisson passes over the shallowest part of the transport route. Owing to the high return current under and alongside of the caisson in the narrow profile, the drag coefficient increases. The speed therefore drops and sideways control becomes more difficult. Increasing the power used (full ahead) brings a risk of dipping the caisson and touching the ground. This happens because the pressure underneath the caisson is less than the hydrostatic. Proper attention must also be paid to the stability of the caissons. This means that adequate calculations of the metacentric height must be carried out (see Section 0).

13.3.3. Preparation of foundation and abutments

All caisson structures have to be placed onto a prepared bed, which has to fulfil several functions. Before placement of the caisson, it is necessary to:

- bring the bottom to the desired level and smoothen it.
- keep it that way until the caisson is in place (flow and waves).
- provide proper connection between consecutive caissons

After the caisson has been put onto the foundation bed, four aspects are important:

- The load of the caisson should be well spread over the bed, which defines tolerance of the bed level in combination with the structural strength of the caisson. This is in particular the case, when the caissons have a permanent function, such as in breakwaters. Uneven (and unpredicted) supports may lead to failure of the bottom of the caisson by excessive bending moments.
- The caisson will soon be subjected to high forces caused by waves or hydrostatic head, which will try to either shift or overturn the caisson. Generally, a linear decline of the potential head underneath the caisson is assumed. However, if permeability of the bed is lowest at the downstream side, the upward pressure is higher than average. Moreover, seepage flow in the bed material concentrates along the lower edge.

The last function of the bed is performed by the part that acts as a scour-prevention in front of the caisson. Although the foundation bed and the bottom protection are not necessarily related, they are usually made simultaneously and of the same materials in order to prevent structural inconsistency at the boundary between them.

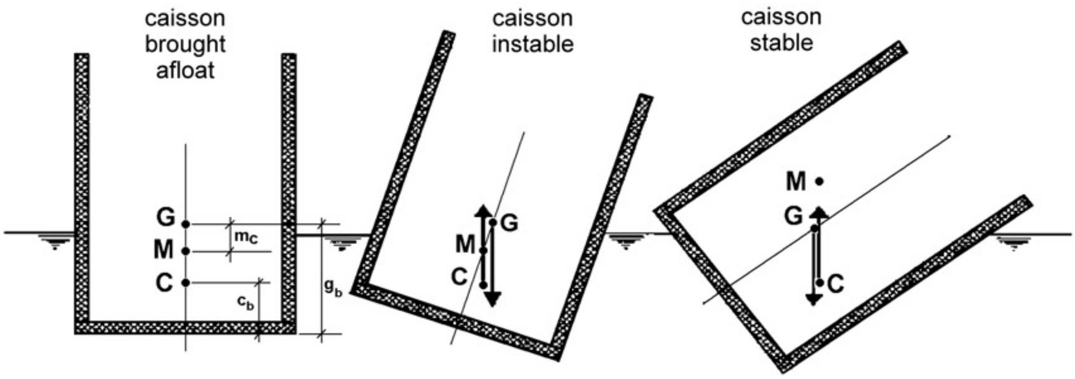


Figure 13.5: Stability of a floating caisson

The first (or only) caisson has to fit to the adjacent dam sections that have been made in a different way. This can be done in various ways. One method is to place a short caisson in advance in order to create a vertical side at its free end. This is linked to the dam section by encircling the other side with the other dam material, (clay, sand, quarry run or the like). The other method is to use a U-shaped sheet pile wall and link the dam to this structure by normal fill. This latter method requires a floating plant to hammer the piles, which may be an expensive operation in choppy water.

13.3.4. FLOATING STABILITY DURING TRANSPORT, POSITIONING AND BALLASTING

The transport of caissons can be an important factor in the design of a caisson. Although caissons are usually designed to withstand forces in their final placed state, it must also be possible to transport the caisson to the site and therefore the draught may be restricted. The dynamic stability of the caisson during transport is another important aspect.

For example, assume a caisson of infinite length, a width of 9 m and a height of 12.5 m. All walls are 0.5 m thick and are made of concrete with density 24 kN/m^3 .The caisson is made in a dry dock and floated by raising the water level in the dock. Then (per metre length) the characteristics of the caisson are:

weight of concrete (G)	396 kN
centre of gravity above bottom (g_b)	4.80 m
moment of inertia ($I = (1/12)LB^3$)	60.75 m^4
displacement (V)	39.6 m^3
draught of caisson (d_r)	4.40 m
centre of buoyancy above bottom (c_b)	2.20 m
$MC = I/V$	1.53 m
metacentre height (m_c)	-1.07 m

Since $MC + c_b$ (3.73 m) is smaller than g_b (4.80 m), the caisson is unstable and will tilt as soon as it comes off the ground. Figure 13.5 shows a clockwise tilted position; the centre C of the submerged part is situated to the left of G and the tilting moment will continue. Although the caisson capsizes, it will not be submerged. At an angle of about 45^o a stable situation is reached. The submerged part is triangular and the water line is much longer than the 9 m, which increases the I, and thus the MC. The MG (m_c) is positive and quite large; thus stability is very good. M is situated on a new stability axis. This is true as long as the vessel does not get water inside, for instance because of motion or wave action. In the figure, the tilt has been drawn just over its stability point to show the righting moment.

The tilt can be avoided by adding ballast to the caisson. Solid ballast (e.g. sand) is advantageous, but in the example shown below, a 2 m layer of water is pumped in and the data change as tabulated below:

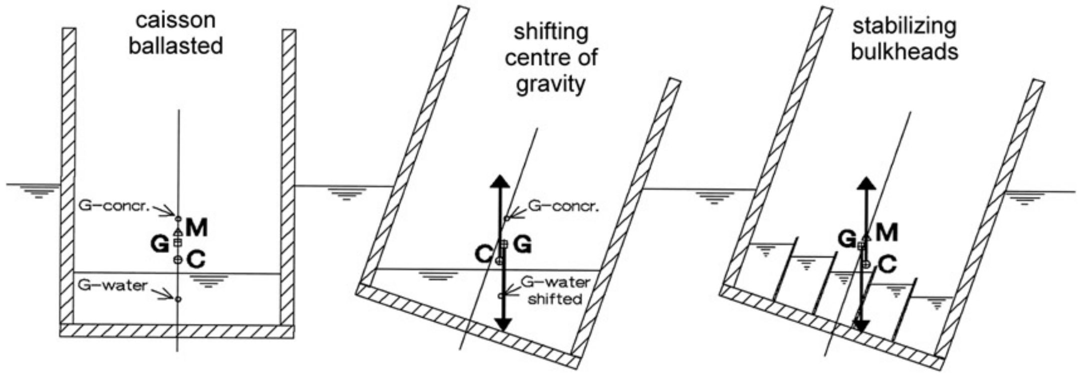


Figure 13.6: Stability of a caisson ballasted with water

weight of concrete and water (G)	556 kN	increased by 160 kN
centre of gravity above bottom (g_b)	3.85 m	down by 0.85 m
moment of inertia (I)	60.75 m^4	unchanged
displacement (V)	55.6 m^3	increased by 16 m^3
draught of caisson (d_r)	6.18 m	increased by 1.78 m
centre of buoyancy above bottom (c_b)	3.09 m	raised by 0.89 m
MC	1.09 m	decreased by 0.44 m
metacentre height (m_c)	0.33 m	increased by 1.40 m

Consequently, the $MC + c_b$ becomes 0.33 m larger than g_b and stability seems to be marginally reached. However, if the caisson gets a little tilt, the liquid cargo starts moving so the centre of gravity also shifts and the resulting moment is increasing the tilt. In the calculation a correction for the I is needed. The water area inside the vessel has to be subtracted. This can be avoided by using bulkheads to subdivide the caisson into compartments . Using solid ballast avoids the dynamic effects of the cargo (see Figure 13.6).

The sinking of a caisson in a closure gap is usually done by ballasting with water. Therefore, its stability has to be calculated for all stages of that operation. Although the sinking is intended to ground the vessel, a list will, at the very least, result in a position out of place.

The above calculations are valid for situations where the outer pressure around the vessel is hydrostatic (Archimedes). This may not be true for a vessel in a strong current flow, as for instance in a closure gap with too little keel clearance. A flow net around the vessel is then needed to give the outer pressure distribution and enable the determination of the position of the centre of buoyancy.

14

FAILURE MODES AND OPTIMIZATION

Chapter 14 looks in hindsight at the design and construction procedures for breakwaters and closure dams. An attempt is made to analyse the various ways in which the structures may fail, either during construction or after completion. This analysis requires a systematic approach in which each step is considered critically. The analysis is certainly not complete, and the reader is invited to participate actively. This certainly applies to the optimization procedures discussed in this chapter and elaborated in one of the appendices.

14.1. INTRODUCTION

For a long time, the design process used for rubble mound breakwaters was not very analytical. Often, a design wave height was selected on the basis of a limited number of field data. The final design was then tested in a hydraulic model. Usually a geotechnical study completed the efforts.

In the hydraulic model study, the hydraulic stability of the cross section when exposed to the design wave was verified. Although it was evident that this design wave height could be exceeded, waves higher than the design wave were seldom used in the model. Scatter in the model results and inconsistencies in the model procedures were seldom taken into account. Safety coefficients were not commonly used to cope with uncertainties in load or resistance of the structure.

In this way, it could happen that new armour units like the Dolos, with a very good hydraulic performance, were developed. However it was not recognized that the mechanical strength of such units was insufficient to withstand the impact forces under design conditions. In the same way, it was not always recognized that the margin between initial damage and failure of the armour was different for armour layers consisting of traditional quarry stone and the newer artificial armour units. This caused a reduction of the inherent safety factor of the traditional structure that went unnoticed.

The failure of a number of large rubble mound breakwaters triggered a more analytical approach to the design of such structures. In the recent PIANC publications on the design of breakwaters, but also in the Rock Manual [2007] and the Coastal Engineering Manual [2002] the design is fully based on probabilities (so including partial safety coefficients or a full probabilistic approach.

This method implies that a full overview of failure mechanisms is available.

14.2. FAILURE MECHANISMS

For a good insight into the behaviour and reliability of a structure under design conditions (and excess of design conditions), it is necessary to have a more or less complete idea of potential failure modes or failure mechanisms. A failure is defined as a condition in which the structure loses its specified functionality. This can be connected either to a serviceability limit state or to an ultimate limit state.

For breakwaters, the protective function is the most important one in most cases. Failure is thus related to any damage that leads to unwanted wave penetration into the harbour, followed by further structural and/or operational damage.

A general overview of failure modes of rock structures is given in Figure 14.1. BURCHARTH [1992] presents a slightly different review more focussed on breakwaters (Figure 14.2).

These overviews are given with some reluctance because it is not yet feasible to give properly defined limit states for each of the failure modes separately in terms of load, resistance and scatter of results. The

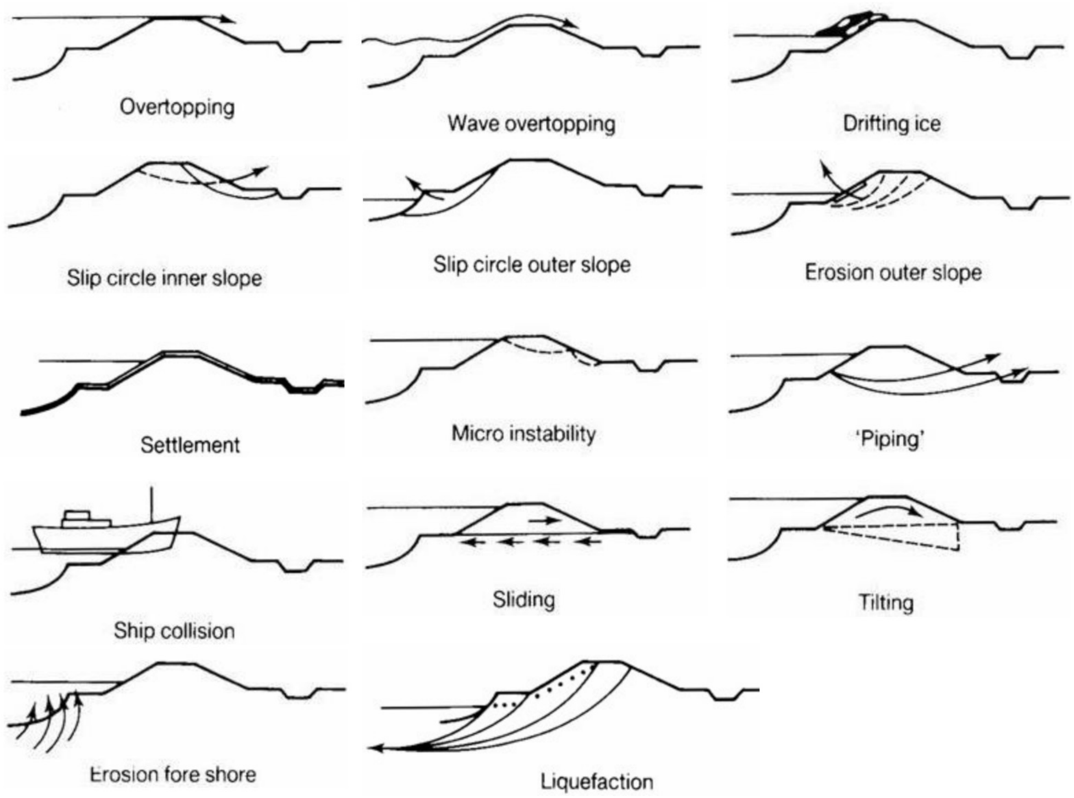


Figure 14.1: Failure modes of rock structures

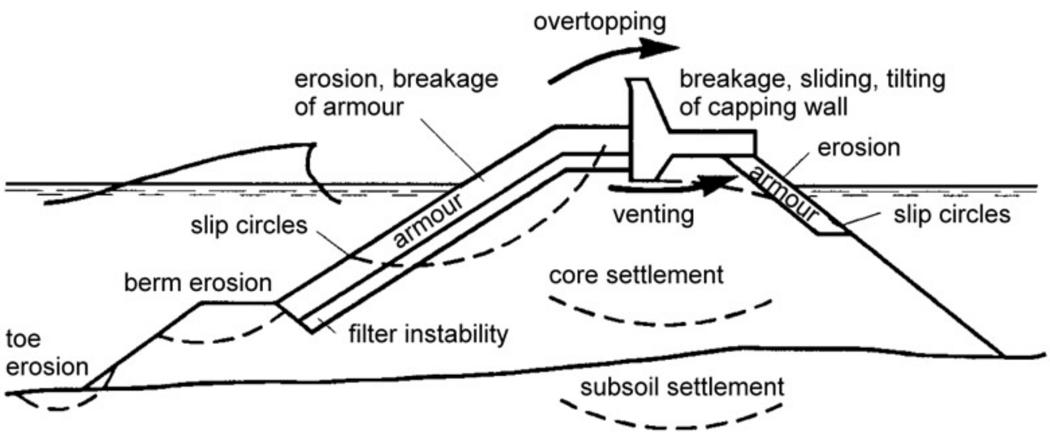


Figure 14.2: Failure modes for a rubble mound breakwater according to Burcharth [1992]

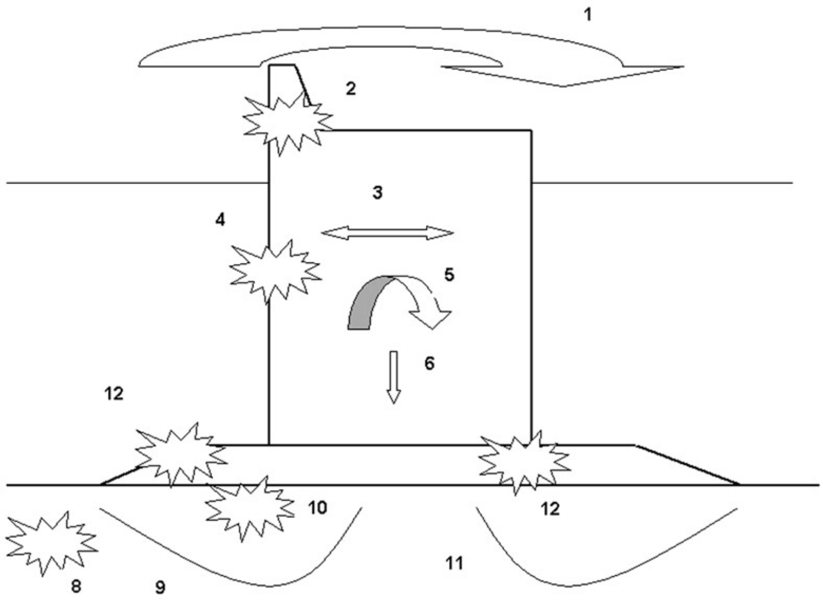


Figure 14.3: Some failure mechanisms for monolithic breakwaters

same applies to monolithic breakwaters. For this type of breakwater, some failure mechanisms have been assembled in Figure 14.3.

14.3. FAULT TREES

A structure can be schematized as a complex system consisting of many components, which may function or fail. A fault tree sketches the systematic relations between failure and malfunctioning of all components in their mutual, interactive relation. Failure of a component may or may not trigger the malfunctioning of a component at a higher level, until eventually the structure as a whole does not perform the functions for which it was meant. Failure of the structure as a whole may also occur if two non-correlated events happen at the same time. Considering these options, one can indicate "AND and "OR" gates, denoting parallel and serial relationships. By quantifying the probability of failure for each component, and by combining the various causes of failure, it is possible to assess the overall probability of failure of the system, be it a breakwater or a closure dam. It is beyond the scope of this book to enter deeply into the theory of fault tree analysis. The reader is referred to more specialized books on reliability theories. However, for illustration, a simplified fault tree and the related calculation of the probability of failure of a breakwater are given in Figure 14.4.

A complete fault tree analysis reveals the contribution of each failure mechanism to the overall probability of failure for the complete structure.

The probability of failure for each component of the system can be determined by making a preliminary design and assessing the uncertainties in load and resistance (strength) via a reliability parameter Z. This can be carried out at different levels of sophistication. A full probabilistic approach (level 3) using for example a Monte Carlo method is quite feasible. However, also a method with partial safety coefficients (level 1) is possible. The methods to find the partial safety coefficients can be found in design manuals (e.g. the PIANC guidelines from 1992 and 2003). For breakwaters, the uncertainty of the wave climate is often the most important contribution to the probability of failure on the loading side. On the structural side, however, the major contribution is provided by the scatter in the stability formulae and the inaccuracy of the nominal diameter of the armourstone (d_{n50}). In this way, it is possible to make analyses to determine the most promising measures to reduce the probability of failure if necessary.

For closure dams, the Rock Manual [2007] describes 3 fault trees for the events:

- Failure of cross-section of rock-fill dam
- Failure of construction equipment
- Failure of construction planning

Obviously, these fault trees and failure modes are only used to demonstrate the approach to be taken. For

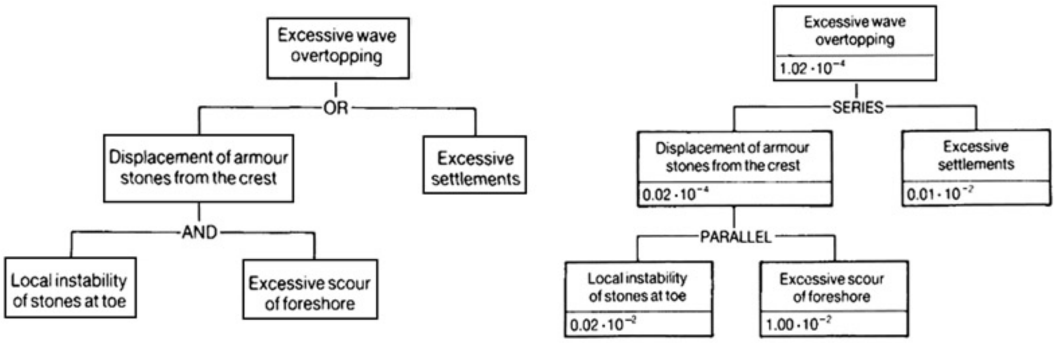


Figure 14.4: Fault tree and probability of failure

instance, for a sill the individual failure modes will differ but the general characteristics of such a fault tree will hold. The same holds for the transition structures.

In many cases, a risk analysis of possible failure modes will not prevent an event from happening. However, by means of this analysis, it should be possible to decrease the probability of its occurrence and/or to limit the consequences of such a failure. One way of achieving this is by diverting some specific construction elements from the critical path in the construction program.

The next question is whether the calculated probability of failure for the system is acceptable. After a lengthy study to quantify the probability of failure, it is highly unsatisfactory to make this decision on an irrational basis.

It is then wise to quantify the risk of failure in terms of the product of probability of failure and its consequences (damage). These consequences are not limited to the cost of the failing structure, but include the consequential damage. For a fully destroyed breakwater, the damage thus represents the cost of reconstruction of the breakwater plus the delays in the port operations resulting from this. The risk (being the product of probability and cost of damage) is expressed in terms of cost per unit of time (generally per annum).

It is possible to reduce the risk by strengthening the structure. Usually this can only be done at some extra cost. In this way, the construction cost of the structure increases, but the risk is reduced, mainly due to a lower probability of failure. Since the construction cost is expressed in actual monetary value at the time of construction, it is necessary to capitalize the annual risk due to failure over the lifetime of the structure and calculate its present-day value. The extra construction cost can then be compared with the savings on the capitalized risk.

In practice, the situation is more complicated, because it is not only the risk of failure that has to be accounted for, but also the risk of partial damage, resulting in the need for maintenance and repair. A second complication is that often there are several ways to improve the strength of a structure and it is not always clear what is the best (most economical) way to do this. These macro and micro optimization processes are discussed in the next section.

14.4. OPTIMIZATION

14.4.1. MICRO LEVEL

Optimization at micro level can best be explained by considering the deterministic design process. The objective of this process is to make a design that leads to the minimum total cost for a given strength level. To achieve this goal, it is necessary that all material in the structure fulfils its function, and is optimally used.

This can be compared with designing a frame. An attempt is then made to select the members in such a way that all are exposed to a stress level close to the maximum admissible stress. In the same way, an attempt can be made to ensure all elements in a closure dam or a breakwater are close to failure or partial failure when exposed to the design load. In a probabilistic design process, this means that one should avoid a very large contribution to overall failure by a single partial-failure mechanism while other mechanisms make no contribution to the probability of failure. It is wise to distribute the contribution to overall failure over a number of failure mechanisms. In fact, one should base this distribution on considerations of marginal cost. If a construction element is relatively cheap, over-designing it is not so much of a problem. If it is relatively expensive, over-designing leads to too high a cost in comparison with other elements

This means that the designer must attempt to make a balanced design, as can easily be explained when considering the cross section of a rubble mound breakwater. If the crest level designed is so high that no overtopping occurs even under severe conditions, it makes no sense to protect the inner slope with heavy armourstone. For a low crested breakwater on the other hand, it is essential to carefully protect the inner slope.

14.4.2. MACRO LEVEL

Optimisation at macro level can also best be explained by taking the deterministic design process, when only one failure mechanism with simple load and strength parameters is considered. When more mechanisms and parameters play a role, the calculations rapidly become more complicated, and one should be careful not to make mistakes that lead to false conclusions.

VAN DE KREEKE AND PAAPE [1964] developed the method for rubble mound breakwaters as early as 1964. The method is discussed below, and a sample calculation is given in Appendix 5. References to Tables and Figures refer to that Appendix.

The method starts with the assumption that there is a direct relation between one load parameter (the no-damage wave height, H_{nd}) and a strength parameter (the weight of the armour units, W). It is further assumed that the wave climate is known and available in the form of a long-term distribution of wave heights (Table A5-1). The interaction between load and strength is determined on the basis of laboratory experiments, which indicate that damage starts when a threshold value (Hnd) is exceeded. The damage to the armour layer increases with increasing wave height until the armour layer is severely damaged and the core of the breakwater is exposed. This occurs at an actual wave height $H = 1.45H_{nd}$. It is assumed that damage is then so extensive that repair is impossible and that the entire structure must be rebuilt. For intermediate wave heights, a gradual increase in damage is assumed, which is expressed as a percentage of the number of armour units to be replaced (Table A5-2).

The breakwater is designed for a number of design wave heights, where a higher design wave requires a heavier and more costly armour layer, whereas the core remains unchanged. The cost of construction is I. The cost of rebuilding the breakwater is assumed to be equal to the estimated construction cost, while the cost of repairing damage is assumed to be double the unit price of the armour units. It is then possible to list the construction cost and the anticipated cost of repair, still split over the three categories of damage (4%, 8% and collapse). Adding together the three categories of damage for a particular design-wave height yields the average annual risk anticipated for that design if all damage is repaired in the year the damage took place. If it is decided not to repair the breakwater except in case of collapse, the risk is only the risk caused by the category collapse.

Since the risk is still expressed in terms of a value per annum, it is necessary to assess what amount of money should be reserved at the time of construction to allow for payment of the average annual repair cost during the lifetime of the structure. Although money is regularly spent from this repair fund, the balance still accrues interest at a rate of ä % per annum. At the end of the calculated life time, the balance of the fund can become zero.

If the annual expense is s, the interest rate δ %, and the lifetime of the structure T, it can easily be derived that the fund to be reserved (S) is:

$$S = \int_0^T e - \frac{\delta}{100} t\,dt = s\frac{100}{\delta}\left(1 - e^{-\delta T/100}\right) \tag{14.1}$$

For $T = 100$ years, $S = s \cdot 100/\delta$ and for $T = 10$ years, $S = 0.35s \cdot 100/\delta$.

An interest rate in the order of 3.5% is usually set. By adding the initial construction cost (I) and the capitalized risk (S), one arrives at the total cost of the structure. When this total cost ($I + S$) is plotted as a function of the design wave height, it appears that there is an optimum design wave height or an optimum strength for the structure.

15
Review

Chapter 15 reviews the result of the design process as a whole. It repeats especially the main choices to be made during design.

15.1. Rubble or monolithic

The main choice facing the designer of a breakwater is the choice between a structure of the rubble mound type and one of the monolithic type. The advantages and disadvantages of each are therefore repeated here. Some of these are site specific and some are valid for the present time only. The designer must therefore carefully assess in which direction he should move.

Advantages of the rubble mounds are:

- Simple construction
- Withstands unequal settlements
- Large ratio between initial damage and collapse
- Many guidelines available for the designer

Disadvantages of the rubble mound are:.

- Dependence on the availability of adequate quarry
- Large quantity of material required in deeper water
- Large space requirement
- Difficult to use as a quay wall

Advantages of the monolithic breakwater are:

- Short construction time on location
- Can function as quay wall
- Economical use of material in deeper water

Disadvantages of the monolithic breakwater are:

- Sensitivity to poor foundation conditions (settlements and liquefaction)
- Uncertainty about wave loads in breaking waves
- Complete and sudden failure when overloaded
- Reflection against vertical wall
- Limited support for the designer from guidelines and literature

In practice, this means that the choice of the basic type of breakwater will largely be made on the basis of wave-climate and soil conditions.

15.2. QUARRY STONE OR CONCRETE ARMOUR UNITS

Generally, the use of quarry stone will be cheaper than the use of concrete blocks, even if the availability of quarry stone is limited. A problem of using quarry stone is the fact that it is a natural material, so its quality and properties are governed by nature. This means that neither density nor maximum size can be selected freely by the designer.

The main tool for the designer who is facing problems with the stability of quarry stone armour is reduction of the slope. The decision to reduce the slope is accompanied by a big increase in the volume of material required. At a certain point, the step towards concrete armour units becomes inevitable. In that case, the question that arises is whether to use simple blocks, cubes (or similar shapes) or more complicated shapes that rely on their interlocking capabilities. In the circumstances prevailing in the Netherlands and Belgium, preference is given to the simpler blocks, mainly because of the ease of construction and handling. Nevertheless, if a decision to use the more complicated units is made, the utmost care must be taken to avoid breakage.

15.3. WHICH DESIGN FORMULA?

There are many design formulae available to the designer of both rubble mound and monolithic breakwaters.

For rubble mound breakwaters, the approach of Van der Meer has gained worldwide support, although the structure of the formula set is not very satisfactory as it lacks a direct link with physical understanding. Therefore it is still recommended that a final design be checked in a physical model. Irregular waves must certainly be used in the model study. It is also recommended that the behaviour of the structure under overloading should be checked to establish the condition where it fails. If the ratio between no damage and failure is small, this must have repercussions on the choice of the Ultimate Limit State.

For the time being it is recommended that the Goda formula should be used for monolithic breakwaters. The disadvantage is that, like the Van de Meer formulae, it has an inadequate theoretical base. Moreover, the Goda formula has acquired far less experimental support from elsewhere in the world. In this case also, physical model tests are strongly recommended. Proper attention must be paid to dynamic loads and their effect on the structure and the foundation.

15.4. SERVICEABILITY LIMIT STATE

The design and the cost estimates are very sensitive to the level at which the breakwater must exercise its functions. It is therefore essential that the functional requirements are analyzed carefully and that a clear distinction is made between ULS (survival) and SLS (functioning). A frequent mistake is to overestimate the functional conditions, which results in structures with too high a crest level. Since the volume in the cross-section is proportional to the square of the height, this has serious cost consequences.

APPENDICES

A

Determination of the design storm

A.1. Introduction

For the design of a breakwater it is necessary to know the hydraulic boundary conditions, consisting of design values for wave height, wave period, water level and current velocity. These values are usually not readily available, so the first step in the design process is to study the meteorological conditions at the project site and determine these values. This document describes the basic steps that need to be taken. We will focus in particular on the determination of the design wave height. The steps in such a wave study are as follows:

- Selection of the required **return period**. The design will be made for some 'extreme' wave condition that does not occur very often, but how often is still acceptable?
- Analysis of an appropriate **dataset** of wave observations and transformation of this dataset into a format that can be used for further analysis
- The dataset will typically only contain a few years of data (maybe 10, up to 20 years) which is likely to be less than the selected return period. This means that the corresponding wave conditions cannot be read from the data directly and that some kind of statistical **extrapolation** of the data is needed.
- Is is very likely that no dataset can be found for the exact location of the breakwater, but rather for some location in the vicinity (usually in deeper water further offshore from the breakwater site). This means that an **offshore-nearshore transformation** is needed to obtain information at the breakwater site

Steps three and four can also be carried out in reverse order, depending on the project site and the preference of the designer. In this document we will follow the order given above. Sections A.2 through A.4 will treat the first three steps. The last step, wave transformation, is treated separately in the lecture notes.

A.1.1. General principle

The general principle assumes that we have a dataset of observed storm wave heights over a certain period of time. We can then assign an observed return period to each storm, plot the data on a graph, fit a line through the points, extrapolate this line to higher return periods and read the wave height corresponding to our design return period from the graph. Figure A.1 illustrates this principle.

Although this sounds straightforward there are a few things we have to keep in mind:

- We need to have a dataset of *storms*, not just observations of the wave height at random moments in time. How such a dataset can be assembled is treated in the next sections.
- We are talking about the *long term* statistics of the wave height distribution, not the *short term* statistics. The short term statistics describe the distribution of wave heights in a given wave field, during the same storm event, giving parameters like H_s, $H_{2\%}$, H_{rms} et cetera. The long term statistics describe how one storm is different from the next. We will describe the overall height of a storm wave field by its significant wave height 'Hs per storm' denoted H_{ss} and we will study the statistics of this parameter.
- We will assume the data points will follow an (unknown) underlying probability distribution. We can therefore not just fit any line through the data points, but we need to fit functions that correspond to the actual distributions that we are considering. In this document we will use four different distributions, being the Exponential, Weibull, Gumbel and Generalized Pareto distributions. The details of the fit

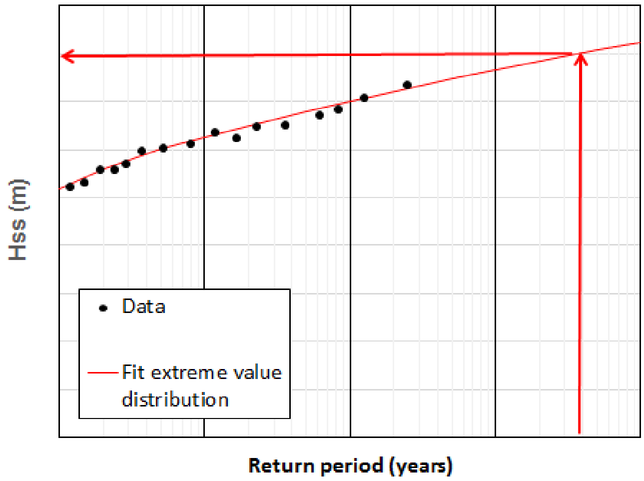


Figure A.1: Illustration of extrapolation principle

procedure depend on the mathematical properties of these distributions. There is no formal reason why the extreme wave height statistics should follow any of these distributions in particular, and there is no consensus in literature as to which distribution should be used. Sometimes also other distributions such as Fréchet or lognormal are being proposed. In practice it appears that the Weibull distribution is selected more often than the others. In this document we will adopt an engineering point of view in the sense that we will treat all four distributions and select the one that fits our data best. This can vary from one situation to another.

- An extrapolation is only statistically meaningful if the data is *homogeneous* and *independent*. While independence can be safely assumed for a dataset consisting of storms, homogeneity is another thing. This can be interpreted as a requirement that the underlying data all originate from similar meteorological events, just varying in magnitude. So a dataset that consists of a mix of swell waves and locally generated wind waves, which are clearly not the same phenomenon, cannot be used for extrapolation. The same is true for datasets consisting of waves from different directions. This usually means that the available dataset must be analyzed carefully and that a reasonably homogeneous subset of the data must be made before carrying out any further analysis.
- An extrapolation is a purely statistical procedure that assumes wave height can increase indefinitely *without any physical limitations*. For instance shallow water effects such as breaking, or potential geographic limitations of the fetch length, are not included. This means that the results of an extreme value analysis must always be combined with engineering judgment of the physical circumstances.
- The statistical analysis always takes place using probabilities *per storm*, but for design purposes we are interested in return periods and probabilities *per year*. Therefore we do not only need to know the actual wave heights but also their distribution in time. Especially the number of (observed) storms per year N_s is important. This parameter serves as the scaling factor between statistical probabilities and design probabilities per year.

A.1.2. THE POISSON EQUATION

The mathematical background of the number of occurrences of an extreme event (such as a storm) is given by the Poisson distribution. This distribution predicts the likelihood that a given *event* will occur *a certain number of times* during *a specific period of time*, and is valid for all processes where the duration of the event itself is much shorter than the period of time that is being analyzed. The classical example for processes that obey a Poisson distribution is the number of cars that pass in the street during one hour, or the number of people that show up at a shop during a day. In our application we will use it to model the number of storms of a certain magnitude that will occur during one year. For the sake of example, let's say that we define our event as "a storm occurs with a significant wave height H_s that is just high enough to destroy our structure", and let's call the number of times that this storm occurs in a year X. The probability that this storm occurs k times in a year is given by:

$$Pr\{X = k\} = \frac{\lambda^k e^{-\lambda}}{k!} \tag{A.1}$$

The distribution is uniquely defined by the parameter λ which is the *expected value of the number of occurrences per year*. This parameter is equal to the inverse of the return period R:

$$R = \frac{1}{\lambda} \tag{A.2}$$

In practical engineering we are not interested in the exact number of times that a storm occurs, (just once or two or three or ten times), because our structure will fail at the first occurrence and then it does not matter anymore if the storm occurs another time or not. In other words, our failure probability p_f is equal to the probability that the storm occurs *at least once*, so it follows that:

$$\begin{aligned} p_f &= Pr\{X \geq 1\} = Pr\{X = \text{not } 0\} \\ &= 1 - Pr\{X = 0\} \\ &= 1 - \frac{\lambda^0 e^{-\lambda}}{0!} \\ &= 1 - e^{-\lambda} \end{aligned} \tag{A.3}$$

This is the probability of failure during the selected period of analysis, which is one year. Usually we are interested in the probability of failure during a longer period, say the lifetime of the structure T_L. We can calculate this probability as follows:

The probability that the structure will fail during one year is:

$$p_f = 1 - e^{-\lambda}$$

The probability that the structure will survive (*not fail*) during one year is:

$$p_s = 1 - p_f = e^{-\lambda}$$

The probability that the structure will not fail T_L times in a row is:

$$p_{s,TL} = p_s^{T_L} = e^{-\lambda T_L}$$

And finally the probability that the structure will fail somewhere in T_L years is:

$$p_{f,TL} = 1 - p_{s,TL} = 1 - e^{-\lambda T_L}$$

This equation is usually called the Poisson equation because it is derived from the Poisson distribution. For design purposes it is more conveniently expressed in terms of R instead of λ, giving:

$$p_{f,TL} = 1 - e^{-T_L/R} \tag{A.4}$$

This is an important equation in the reliability-based design of (coastal) structures and it is important to realize its consequences. For instance, a common misconception is that it is safe to design a structure for a storm with a return period equal to the lifetime of the structure ("a 100 year storm for 100 years lifetime"). We can see from the Poisson equation that the corresponding probability of failure for $R = T_L$ is equal to $p_f = 1 - e^{-1} = 0.63$ of 63%.

For larger return periods the Poisson equation can be approximated to $p_f \approx T_L/R$ which can be convenient for discussions and quick calculations. For example, the probability of failure for a structure with a lifetime of $T_L = 50$ years that is designed for a $R = 500$ year return period is approximately $50/500 = 0.1$ or 10%. The "full" Poisson equation would give $p_f = 1 - e^{-50/500} = 0.0952$ or 9.5 %.

A.1.3. DEFINITIONS

The difference between probabilities per storm and probabilities per year can lead to some confusion in notation and understanding, so it is good to state our definitions clearly at this point. In this document we will use the following definitions:

- The **non-exceedance probability** P is defined as the probability that a given storm has a significant wave height that is smaller than (or equal to) a certain value. Mathematically, this corresponds to the definition of the cumulative distribution function $F_H(H_{ss})$ of the extreme value distribution that we are considering.

$$\begin{aligned} P &= F_H(H_{ss}) = Pr\{H \leq H_{ss}\} \\ &= Pr\left\{\text{Wave height } H_{ss} \text{ is not exceeded } \textbf{per storm event}\right\} \end{aligned} \tag{A.5}$$

- The **exceedance probability** Q is defined as the probability that a given storm has a significant wave height that is larger than a certain value.

$$\begin{aligned} Q &= 1 - P = Pr\{H > H_{ss}\} \\ &= Pr\left\{\text{Wave height } H_{ss} \text{ is exceeded } \textbf{per storm event}\right\} \end{aligned} \tag{A.6}$$

- The **expected number of exceedances per year** λ is defined as the average number of times per year that a storm with certain wave height or higher occurs. This is the parameter of the Poisson distribution that we saw above. Strictly speaking this is not a probability in the mathematical sense because the value can be larger than 1, but for small values of λ it can be interpreted as the probability per year that a storm is exceeded.
- The **return period** R which is the average time period between two consecutive storms with a significant wave height at least equal to H_{ss}. This is the most common way to express the desired design condition. In this document, the chosen design return period is denoted R_D and the corresponding storm wave height $H_{ss,D}$.

The first two parameters are defined in a statistical context, the last two are defined in a design context. They can be transformed into each other using the definitions below.

$$\begin{aligned} Q &= 1 - P \\ \lambda &= Q \cdot N_s \\ R &= \frac{1}{\lambda} = \frac{1}{QN_s} \\ Q &= \frac{1}{RN_s} = \frac{\lambda}{N_s} \end{aligned} \tag{A.7}$$

where N_s is the observed number of storms per year. For a given location, all four parameters P, Q, λ and R will be a function of H_{ss}, in other words there will be values for these parameters for all possible wave heights. The essence of an extreme value analysis as outlined in this chapter is to establish this relationship and then find the design wave height H_{ss} that corresponds to a given value of R.

A.2. SELECTING A RETURN PERIOD

The design return period can be obtained for instance from (local) norms or standards, client specifications or an economic optimization analysis. These methods are treated elsewhere in the lecture notes and are not repeated here.

In the absence of further specifications, the return period can be derived from an estimate of the acceptable failure probability of the breakwater during its lifetime $p_{f,L}$. If the lifetime is L years then the acceptable failure probability per year $p_{f,y}$ and the corresponding return period R can be calculated from the Poisson equation (equation A.4).

Please note that the definition of 'failure' in this context is strongly related to the calculation that is being made. For instance, if the boundary conditions are needed for a rock armour calculation with $S = 2$ then 'failure' means that there is initial damage to the rock armour and that some repairs are needed. This might be accepted a few times during the lifetime of the breakwater, depending of course on the maintenance strategy of the port owner and the accessibility of the armour slope for construction equipment. On the other hand, if boundary conditions are needed for calculations that relate to structural damage of important port infrastructure, or to issues that can cause major downtime to the port operations themselves, then 'failure' is more significant and a smaller associated probability is likely to be selected.

It is up to the designer to make a good analysis in this context. It might be necessary to distinguish between Ultimate Limit States (ULS) and Serviceability Limit States (SLS) and select different return periods for each of these, or distinguish between failure mechanisms and select for instance a different return period for armour stability than for overtopping or transmission.

In general the consequences of 'failure' for breakwaters are mainly economic and are not as extreme as for example the consequences of a dike breach. Usually no loss of life or major damages are expected. For this reason the return periods commonly seen in breakwater design are (much) lower than the return periods used in flood defenses. Typical values are in the range $R_D = 100 - 1000$ years. These choices are not further treated here, and this document should not be seen as a recommendation to select any given value. A project-specific analysis should always be made.

In our example we will select an arbitrary design return period of $R_D = 200$ years, which corresponds e.g. to a failure probability $p_{f,L}$ of just over 20% for a lifetime of $L = 50$ years.

A.3. ASSEMBLING A DATASET OF OBSERVATIONS

The next step is to assemble a good dataset of (H_{ss}, R) observations that can serve as the basis for the statistical extrapolation. A dataset like the one we are using in the example usually comes in any of the following two formats:

- As a timeseries of observations at a certain location, taken from either a wave buoy, satellite observations or model hindcast data. Figure A.4 shows an illustration of one month of observations from an example dataset. Please note that these observations are not continuous observations of individual wave heights, but rather registrations of the significant wave height of the wave field, usually a few hours apart. A good dataset will also contain other information such as wind speeds, wind / wave direction and wave period measured at the same time as the wave height.
- As a frequency table showing the number of occurrences of a given significant wave height in *bins*, sometimes split across associated wave periods and/or directions. Sometimes the number of occurrences are tabulated, sometimes only the percentages of the total number of observations are given.

A time series is somehow more useful for our purposes than a frequency table, because the latter can always be constructed from the former, but not the other way around. A frequency table on the other hand is more concise and easy to interpret. Also, it can be used directly to study the day-to-day wave climate at the site which can provide very valuable information for the construction phase of the project (downtime of equipment, workability) or the operations of the port.

A.3.1. FILTERING THE DATASET

In either case we will first have to modify the dataset in order to transform the individual observations to storms, and meet our criteria on statistical independence and homogeneity. The last criterion requires that we split the dataset into subsets containing data from the same *population*, which we will interpret as being caused by similar meteorological phenomena. Advanced procedures can be used here (such as spectral partitioning, see e.g PORTILLO ET AL (2015) but these are outside the scope of the present document. Here we will use common sense and engineering judgment. For instance, we can separate out wind sea and swell observations (for instance based on the wave steepness of an observation). Another good practice is to separate out waves coming from different directions. Apart from the statistical necessity to do so, this is also useful for direct design purposes if the breakwater site can clearly not be reached by waves from some directions. As an example, if the location of the dataset is due West from the breakwater location then waves coming from easterly directions do not have to be included in the analysis.

When a very stringent choice is made concerning filtering data to ensure statistical homogeneity, it may happen that the remaining dataset becomes too small for further analysis. In those situations the designer will have to make a compromise between quantity and quality of the data.

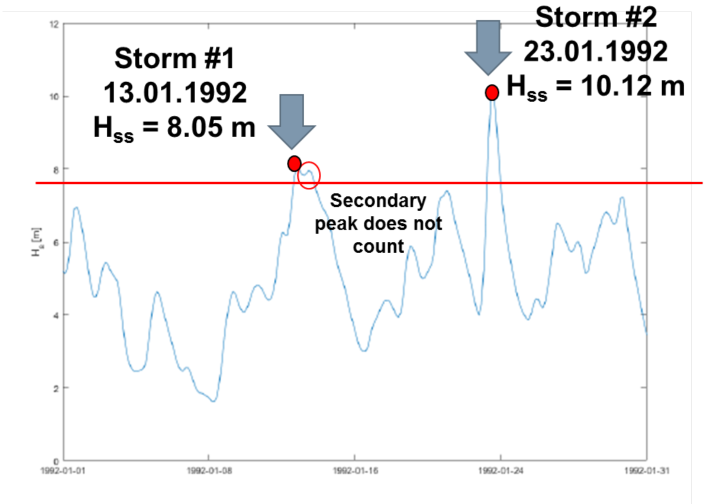


Figure A.2: Principle of a Peak over Threshold analysis

A.3.2. OBTAINING STORM OBSERVATIONS

A timeseries of observed wave data forms a good basis for an extreme value analysis, but it cannot be used without any further pre-processing. The most important thing to realize is that the dataset will consist of a series of *observations*, while for design purposes we are interested in the statistics of *storms*.

For instance, several consecutive observations could be minutes or hours apart and could well belong to the same storm. Such a storm should be counted only once in our statistical analysis. Also, from a design point of view we are usually concerned only with the wave height during the peak of the storm, not in the wave heights during the few hours before the peak (when the storm is building up) or after the peak (when the storm is subsiding again).

The most common way to process a timeseries of observations into a dataset of storms is by means of a peak-over-threshold (PoT) analysis. In such an analysis a storm is arbitrarily defined as a period of time in which the observed wave height is higher than a certain threshold. The time series is scanned time step by time step, for instance by a simple `Matlab` or `Python` script, looking for periods in which the threshold is exceeded. Only the highest value (peaks) of the wave height during these periods of exceedance is recorded as 'the' storm wave height H_{ss}, and all observations below the threshold are discarded. See Figure A.2 for an illustration. In more advanced PoT techniques a further selection of storms is made, for instance if the peaks of two storms are less then 24 hours apart they are counted as a single storm. Usually a simple approach as sketched here is sufficient however.

The number of storms per year, N_s follows directly from the remaining dataset. For instance, if a total dataset of 20 years has been processed and after the PoT analysis 100 storms remain, then $N_s = 100/20 = 5$ storms per year.

The number of data points in the resulting dataset of storm observations will be a function of the selected threshold level. The higher this threshold was set, the fewer storm observations will be left (so the fewer data points remain for a statistical analysis, so the uncertainty in the prediction will go up). The lower the threshold is set, the more data points remain, but if the threshold is set too low the resulting data points will not really resemble extreme storms anymore (in the limit, if we set a threshold of 0 m, we are back at our original dataset of 'non-storm' observations). In practice it is the experience of the engineer who performs the analysis to select a reasonable compromise. Usually several threshold levels need to be selected in a trial-and-error process until a satisfactory result is obtained. A good rule of thumb for a first approach is to select the threshold level such that approximately $N_s = 10$ storms per year of data remain.

The peak-over-threshold method is not the only available method for this type of analysis. An alternative would be to scan the dataset for the highest wave observation in each year and provide a list of annual maxima. This method gives per definition $N_s = 1$. The rest of the procedure for the annual maxima method would be exactly the same as for the PoT method, but just with less data. For this reason this method is not further treated here.

If only summarized data such as frequency tables of observations are available instead of continuous timeseries data, no PoT analysis can be made. In that case an estimate of the extreme wave conditions can also be obtained from the frequency tables, albeit with a lower accuracy. See section A.4.3 for more information.

A.3.3. ESTIMATING RETURN PERIOD FROM OBSERVATIONS

Now that we have a good dataset of storm observations, we can proceed to study the statistics of this dataset. For instance, we can sort the data and assign each data point a rank number i from lowest to highest, $i = 1,2,3,...N$. We can then calculate the exceedance and non-exceedance probabilities of our data points as follows:

$$\begin{aligned} P_i &= \frac{i}{N+1} \\ Q_i &= 1 - P_i \end{aligned} \tag{A.8}$$

These probabilities are sometimes called 'plot positions', for reasons that will become clear later. The above formulas for the plot positions are not the only ones found in literature, for instance GODA (2000) gives factors α and β as in:

$$P_i = \frac{i-\alpha}{N+\beta}$$

In these lectures we are not using these advanced methods and we will stick to the simple plot positions given by equation A.8.

Finally, the sample mean $\overline{H}$ and the sample standard deviation s_x can also provide useful statistical information.

These simple statistics will form the basis for the final extreme value analysis as outlined below.

A.4. EXTRAPOLATION METHODS

A.4.1. FIT PROCEDURE

We started out with the question *'What is the storm wave height H_{ss} that corresponds to a given return period R?'*, and we have now proceeded up to the point where we have a dataset of joint observations of storm wave heights $H_{ss.i}$ and the corresponding exceedance probabilities Q_i per storm. What remains is to translate the target return period R to a target exceedance probability Q and find the corresponding wave height H_{ss} either by interpolation or extrapolation of the dataset. Commonly, the dataset will be relatively short and the target exceedance probability will be relatively low, so we are looking at extrapolation rather than interpolation.

The translation from R to Q can be done using the relationships of equation A.7 established earlier. The extrapolation method is treated here.

As described in the introduction, we will extrapolate our data by fitting it to any of the following four *extreme value distributions*:

Exponential distribution

$$P = 1 - exp\left(-\left(\frac{H_{ss}-\gamma}{\beta}\right)\right)$$

Weibull distribution

$$P = 1 - exp\left(-\left(\frac{H_{ss}-\gamma}{\beta}\right)^{\alpha}\right)$$

Gumbel distribution

$$P = exp\left[-exp\left(-\left(\frac{H_{ss}-\gamma}{\beta}\right)\right)\right]$$

Generalized Pareto distribution

$$P = 1 - \left(1 + \alpha \frac{H_{ss} - \gamma}{\beta}\right)^{-1/\alpha}$$

Note that these four distributions are formally defined by their non-exceedance probabilities P. In engineering practice it is easier to work with the exceedance probability $Q = 1 - P$. The shape of these functions $Q(H)$ and the inverse functions $H(Q)$ are given in section A.7.

All candidate distributions are a function of H_{ss} and at least two parameters β and γ. For some distributions a third parameter α is needed. If we plot $H_{ss,i}$ against Q_i we can fit the candidate distributions and find the unknown parameters. If we consider a candidate distribution with only two parameters (β and γ) we can do that by means of a simple linear regression. If our candidate distribution has a third parameter α we need to select (guess) a value for α, fit for β and γ and try various other values of α until the fit does not improve anymore. (Note: we could also attempt more advanced fitting techniques to fit all three parameters at once, but we will not treat that further here.) Since all candidate distributions are non-linear, we will have to transform the $(H_{ss,i}, Q_i)$ pairs into $(H_{ss,i}, X_i)$ pairs where X is some non-linear function of Q. We can then fit $H_{ss} = A + BX$ by linear regression and find β and γ by inverse transformation of A and B. The appropriate transformations are given in section A.7 for each of the four candidate distributions. Once the transformation has been carried out, the linear regression can be carried out easily in e.g. Excel or other kinds of calculation software.

An alternative method to fit the parameters is by the Method of Moments. This method states that for every distribution with known parameters, the *moments* of the distribution can be expressed as a function of those parameters. The first moment is the mean of the distribution, the second moment is the variance (which is the standard deviation squared). For instance for the Exponential distribution we have:

$$\begin{aligned} &\text{First moment } \mu = \gamma + \beta \\ &\text{Second moment } \sigma^2 = \beta^2 \end{aligned} \tag{A.9}$$

We can estimate the first and second moment, respectively, by the sample mean $\overline{H}$ and the sample variance s_H^2 that we calculated earlier. This gives us two equations with two unknowns, which can be solved for β and γ:

$$\begin{aligned} \beta &= s_H \\ \gamma &= \overline{H} - \beta \end{aligned} \tag{A.10}$$

The formulas for the first and second moments for the other extreme value distribution types are given in section A.7.

Finally, after the values of α, β, γ have been estimated we can calculate our desired design wave height H_{ss} as the value that corresponds to our target exceedance probability Q using the inverse distribution function, for instance for the exponential distribution:

$$H_s s = \gamma - \beta \ln Q$$

A.4.2. UNCERTAINTY IN THE PREDICTION

The uncertainty involved in the prediction of our design wave height can be large. In fact, we are not only predicting a design value $H_{ss,R}$ but also a margin or error around it. This information can be used for instance in a probabilistic design approach. Intuitively it will be clear that this margin of error is larger if we extrapolate 'further ahead', i.e. if our return period becomes much larger than the number of years in our measured dataset, and if the quality of our original data becomes poorer (for instance if we have fewer data points or more scatter in the data).

Several methods exist to estimate this uncertainty, of which we will treat two. The first method is based on empirical research by GODA (2000). Assume we have found a design value $H_{ss,R}$ corresponding to a return period R years. Goda suggests that the uncertainty in the prediction can be modeled as a normal distribution with mean $\mu = H_{ss,R}$ and standard deviation σ_H, and presents an empirical formula to calculate σ_H in the case of Weibull or Gumbel distributions:

distribution	a_1	a_2	c
Gumbel	0.64	9.0	0
Weibull, $\alpha = 0.75$	1.65	11.4	0
Weibull, $\alpha = 1.0$	1.92	11.4	0.3
Weibull, $\alpha = 1.4$	2.05	11.4	0.4
Weibull, $\alpha = 2.0$	2.24	11.4	0.5

Figure A.3: Coefficients for GODA (2000) method of uncertainty estimate

$$\begin{aligned} \sigma_H &= \sigma_x \cdot \sigma_z \\ \sigma_z &= \sqrt{\frac{1 + a\left(y_R - c\right)^2}{N}} \\ a &= a_1 exp\left(a_2 \cdot N^{-1.3}\right) \end{aligned} \tag{A.11}$$

where σ_x is the standard deviation of the original $H_{ss,i}$ data values, N is the number of storms in the dataset, α is the shape factor of the Weibull distribution and the coefficients a_1, a_2 and c can be read from the table A.3. The parameter y corresponds to the reduced variable X_W or X_G used in the linear regression with $Q_s = 1/(R \cdot N_s)$ instead of Q, so:

For Weibull distributions:

$$y_R = \ln\left((N_s \cdot R)^{1/\alpha}\right) \tag{A.12}$$

For Gumbel distributions:

$$y_R = -\ln\left(-\ln\left(1 - \frac{1}{N_s \cdot R}\right)\right) \tag{A.13}$$

The Goda method is only defined for the Weibull and Gumbel distributions. For other distributions advanced methods like bootstrapping can be used. In bootstrapping, a new dataset with the same size of the original dataset is created by selecting N random samples from the original dataset, with replacement. This process, called resampling, will create a new dataset that consists of mainly the same samples, but some will be missing and some occur twice or more. This new dataset can be fitted to the extreme value distribution again, resulting in a slightly different prediction. If this is done many times (say 1000 times or more) in a Monte Carlo fashion, a distribution of predictions can be obtained. The mean and standard deviation of a certain prediction (for instance $H_{ss,R}$) can then be found from this data.

A.4.3. WHAT TO DO IF ONLY SUMMARIZED DATA IS AVAILABLE?

If only summarized information of wave observations is available, for instance in a frequency table, a Peak over Threshold analysis cannot be made. It is still possible to estimate the design wave height, but the accuracy will be smaller.

Summarized data will usually be in the form of frequencies (or numbers of observations) in *wave bins*, such as 'there were 102 observations of wave heights between $H_s = 0.5$ and 1.0 m, 1216 observations between $H_s = 1.0$ and 1.5 m', and so on. We can still perform an extreme value analysis if we treat these bins as 'storm' observations and make the following changes to our analysis:

- We must assign a representative value of H_{ss} to each bin, which could be the lowest, highest or the average value depending on the choice of the engineer.
- We must modify the plotting position for each representative value to:

$$\begin{aligned} P_i &= \frac{n}{N+1} \\ Q_i &= 1 - P_i \end{aligned} \tag{A.14}$$

where n is the *cumulative* number of observations in bin i *or smaller*

- We do not have a direct count of the number of storms N_s anymore, but we can estimate it as:

$$N_s = \frac{365 \cdot 24}{t_s}$$

  where t_s is an estimate of the duration of a single "storm" in our dataset. Common estimates are in the order $t_s = 3$ to 12 hours.
- The method of moments is not defined for binned data, so we must use the linear regression method

A.5. CONCLUDING REMARKS

A.5.1. WHAT CAN POSSIBLY GO WRONG?

Although the procedure described in this chapter appears very rational and straightforward, there are still many points along the way where you will have to make decisions based on engineering judgment. For most cases, there is no single correct answer and there is no norm or standard that you can fall back on. Examples of this type of decisions could be:

Selection of return period:

- Limit state (ULS / SLS)
- Actual choice of R

Dataset:

- Location of the dataset, size of the area for which observations are available, source of information, type of observations (e.g. satellite or bouys etc)
- Filter data for homogeneity (direction, period etc)
- Presentation of data (frequency table or timeseries)

Analysis:

- Method of analysis (summarized data or PoT)
- Selection of threshold value (PoT) or storm duration t_s (summarized data)
- Choice of extreme value distribution type (Gumbel, Weibull, exponential, GPD or even otherwise
- Choice of plotting position (definition of P and Q)
- Fit method of distribution parameters (linear regression, method of moments or otherwise)
- Estimate of the uncertainty in prediction

Transformation:

- Bottom schematisation
- Choice of model
- Model settings (grid size, time step)
- Calibration parameters if any (bottom roughness, breaker height)
- Combination wind and waves
- Output location
- Wave propagation inside harbour basin

And then some more:

- Joint probability wave height - water level
- Joint probability wave height - wave period
- How do you handle the uncertainty in your prediction?

It is important to make the right decisions, but it is even more important to report them well so other people can judge the validity of your analysis. The list above may help you as a checklist to see if your reporting is complete.

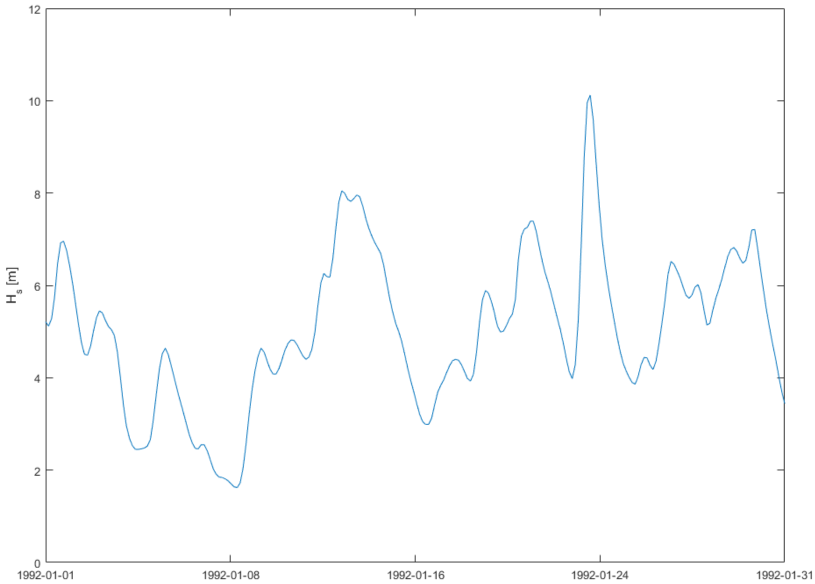


Figure A.4: First month of observations in example dataset

A.6. CALCULATION EXAMPLE

In the following pages we will elaborate an example through all steps in the analysis. The underlying dataset is taken from `www.waveclimate.com`, which is a commercial site of oceanographic data owned by the company BMT-Argoss. A fee must be paid to obtain this data, but a demo dataset is available free of charge. This dataset is used in this document. It corresponds to a point in the Atlantic Ocean just south of Iceland ($63^o 00' N, 20^o 00' W$), so the expected design waves are very large.

The dataset consists of 23 years of wave model data with an output in terms of wave height (H_s), wave period (T_z, T_m, T_p), wind speed, wind direction and wave direction, at intervals of 3 hours. The raw data file can be downloaded as a comma separated ascii file (.csv) from `www.waveclimate.com` or from the Blackboard site of CIE5308.

A.6.1. SELECTING A RETURN PERIOD

In our example we will select an arbitrary design return period of $R_D = 200$ years, which corresponds e.g. to a failure probability $p_{f,L}$ of just over 20% for a lifetime of $L = 50$ years.

A.6.2. ASSEMBLING A DATASET OF OBSERVATIONS

The raw data file `demo26826_series_20161130095542.csv` can be opened in Microsoft Excel or any scripting language like `Matlab` or `Python`. The first 23 lines are header lines, the next 67208 lines contain the actual data. Columns 1-4 contain the time and data of each observation, column 7 contains the wave height H_s, column 8 contains the wave direction, column 10 contains the period T_m and so on, as can be easily seen when the file is opened. The first observation is from 1 January 1992 ($0:00$ hours), the last observation is from 31 December 2014 ($21:00$ hours), giving a total of 23 years of data.

The file can now be read and used for further processing. As an example we have plotted the wave height during the first month (January 1992), see figure A.4.

A.6.3. FILTERING THE DATASET

For our example we have made a quick scan of the dataset, plotting the observed wave heights against wave period and wave direction (see figure A.5). We can observe that:

- There is a clear dominance of wave conditions coming from the East and South-West. There are two peaks, a smaller peak around $90^o N$ and a larger one around $240^o N$. We are interested in the highest

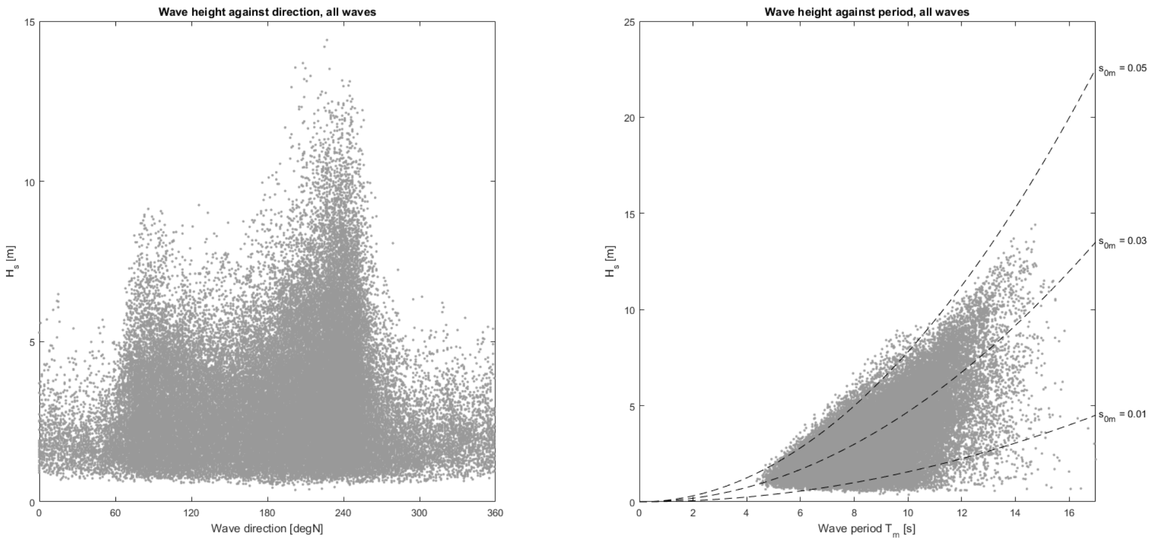


Figure A.5: Quick scan of homogeneity of example dataset

peak, so we will filter out the conditions corresonding to the smaller peak. The boundary between the peaks appears to be at roughly $150^o N$, so we will filter our dataset and keep only those data points that have waves coming from directions between $150^o N$ and $360^p N$. In other situations it may be that there are two or more dominant wave directions. In those cases two separate extreme value analyses must be made, and we will have to re-combine the resulting failure probabilities in some way. There are no clear guidelines on how to handle this type of situation – a choice is left to the professionalism of the designer.

- Swell waves would typically show up in the lower right hand side of a (H, T) plot (long periods, low waves). In the example dataset there are some swell waves, but especially for the higher wave conditions this is not the case so we do not expect that this will cause a bias in the estimate of the extreme wave conditions (in fact we will filter out the lower wave conditions anyway in the Peak over Threshold analysis). There does not appear to be a reason to separate out the swell waves in the present example.

This type of analysis is very crude, and it is stressed that a more sophisticated analysis may have to be made for actual design purposes. The analysis will suffice however for our present example.

A.6.4. PEAK-OVER-THRESHOLD METHOD

From the quick scan of the dataset we see that the larger wave conditions are in the order of $H_s = 10$ m or higher. Let's select a first threshold level of 7.5 m, the sensitivity for this choice will be investigated later.

Scanning the dataset step by step we see that the first time the threshold level of 7.5 m is exceeded occurs on 13 January 1992 (06 : 00 hours) when $H_s = 7.80$ m. The wave heights then stays above the threshold until 14 January 1992 (09 : 00 hours) when the wave height drops to $H_s = 7.45$ m. The maximum wave height between these two moments occurs at 13 January 1992 (09 : 00 hours) when the wave height reaches $H_s = 8.05$ m (corresponding to a direction of 212^o and $T_m = 11.3$ s). This is our first storm. The direction is within our target range of $150^o N - 360^o N$ so we will keep this storm in our analysis. Repeating this process all the way to the last line of data gives the results as shown in table A.1. We find a total of 280 storms this way, so the number of storms per year is $N_s = 280/23 = 12.2$.

If we would have set a different threshold level, we would have found a different number of storms (the lower the threshold, the more storms). The result is given in table A.2. A good rule of thumb is to aim for approximately $N_s = 10$ storms per year, so the original threshold level of 7.5 m was good and we will use these results in the remainder of the example.

A.6.5. EXTREME VALUE DISTRIBUTION FIT - LINEAR REGRESSION

We can now proceed to fit our extreme value distributions (Exponential, Weibull, Gumbel and Generalized Pareto (GPD)), which means that we want to estimate the values of the distribution parameters α, β and γ.

Table A.1: Results from PoT method, example dataset (threshold 7.5 m, directions $150^o N - 360^o N$)

Date	$H_s[m]$	$T_m[s]$	$Dir[^oN]$
13 January 1992	8.05	11.3	212
23 January 1992	10.12	12.3	181
02 February 1992	9.48	12.3	233
07 February 1992	8.96	11.4	220
18 February 1992	10.02	13.3	245
21 February 1992	10.11	12.9	234
24 February 1992	9.06	10.8	244
25 February 1992	9.26	12.0	203
⋮	⋮	⋮	⋮
06 December 2014	8.84	12.7	247
09 December 2014	12.22	14.7	234
17 December 2014	10.60	12.1	250

Table A.2: PoT method, sensitivity of threshold level (directions $150^o N - 360^o N$)

Threshold [m]	Number of storms [-]	$N_s[-]$
5	746	32.4
7.5	280	12.2
10	84	3.7
12.5	7	0.3

We will denote the 'true' values (that we do not know) as α etc and our estimated values as $\hat{\alpha}$, $\hat{\beta}$ and $\hat{\gamma}$.

The first step is to rank the data from table A.1 from the smallest to the highest storm and calculate the (non-)exceedance probabilities P_i and Q_i according to equation A.8. In this example we are using linear regression to find the parameters of the distributions, so we can proceed to calculate the linearized X-axis variables X_E, X_G, X_W and X_P for the exponential, Gumbel, Weibull and GPD distributions respectively. The formulas for these transformations are given in section A.7. The calculation results are given in table A.3.

For the Weibull and GPD distributions the values of X_W and X_P depend on the choice of the parameter $\hat{\alpha}$, which has to be estimated. For the time being, only the values that gave the best fit after a trial-and-error process are presented here. This procedure is explained later.

The next step is to plot $H_{ss,i}$ against X_i and draw the regression lines $H_{ss} = A + B \cdot X$, see figure A.6. The values for A and B are given in the plots as well as in table A.3. The values of the distribution parameters can now be found from these regression parameters, and because of the way we defined the linearization X_E etc, we get $\hat{\beta} = B$ and $\hat{\gamma} = A$ in all cases.

Now that we know $\hat{\alpha}$, $\hat{\beta}$ and $\hat{\gamma}$ for all distributions, we can calculate a value for H_{ss} for every value of Q using the inverse distribution formula (e.g. for the exponential distribution $H_{ss} = \gamma - \beta \ln Q$, see section A.7). If we do this for all values of Q_i in our dataset we get a predicted value of $H_{ss,pred,i}$ that we can compare to the original data point $H_{ss,i}$. We can then define a metric for the goodness-of-fit of the regression, for instance the Root Mean Square Error (RMSE):

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(H_{ss,i} - H_{ss,pred,i}\right)^2}$$

The values of the RMSE for the fit of all four extreme value distributions is given in table A.3. For the Weibull and GPD distributions, this metric can also be used to optimize the value of $\hat{\alpha}$. The initial guess of $\hat{\alpha}$ can be adjusted, and the regression can be repeated, until the lowest value for RMSE is obtained. This procedure has been followed here, and the calculation results for the best fit (lowest RMSE) have been presented.

A.6.6. EXTREME VALUE DISTRIBUTION FIT - METHOD OF MOMENTS

We can also use the method of moments to estimate the distribution parameters. In that case we need to calculate the mean $\overline{H}$ and standard deviation s_H from the H_{ss} values in table A.1. We find $\overline{H} = 9.23$ m and s_H

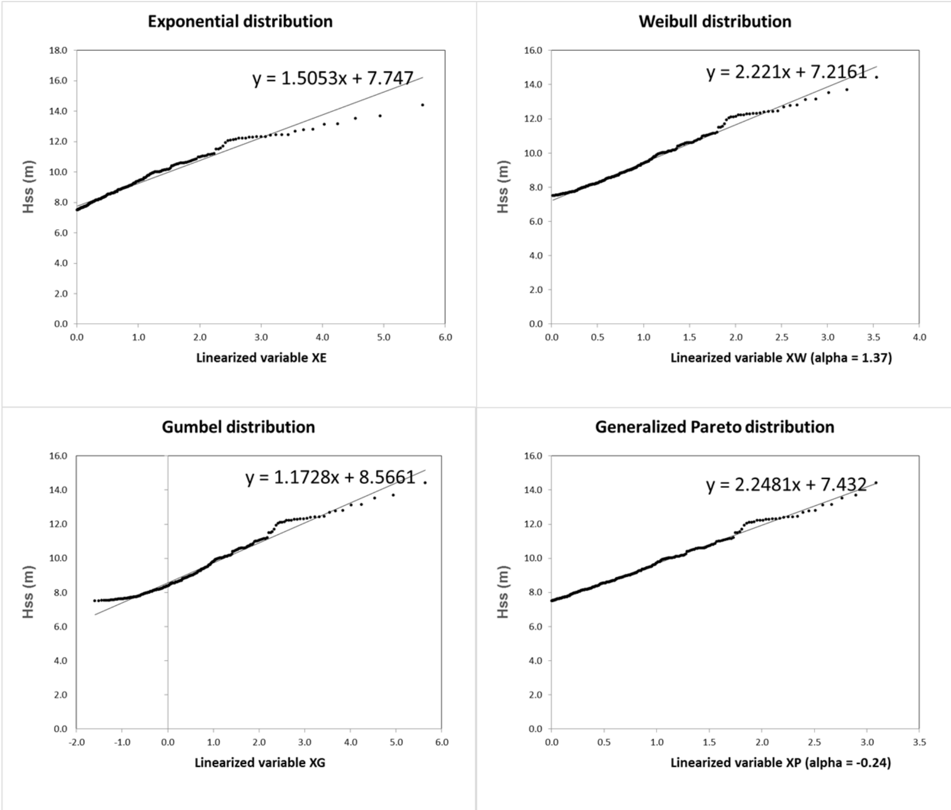


Figure A.6: Result from linear regression, example dataset (data from PoT analysis, sector $150^o N - 360^o N$, threshold $7.5m$)

Table A.3: Extreme value distribution fit (ranked data from PoT analysis, threshold 7.5 m, directions $150^o N - 360^o N$)

Data				Expon.	Gumbel	Weibull $\alpha = 1.37$	GPD $\alpha = -0.24$
$H_{ss,i}$	i	P_i	Q_i	$X_{E,i}$	$X_{G,i}$	$X_{W,i}$	$X_{P,i}$
7.51	1	0.004	0.996	0.0036	–	0.0163	0.0036
7.51	2	0.007	0.993	0.0071	-1.5984	0.0271	0.0071
7.51	3	0.011	0.989	0.0107	-1.5129	0.0365	0.0107
7.52	4	0.014	0.986	0.0143	-1.4474	0.0451	0.0143
7.52	5	0.018	0.982	0.0180	-1.3935	0.0532	0.0179
7.53	6	0.021	0.979	0.0216	-1.3472	0.0608	0.0215
7.53	7	0.025	0.975	0.0252	-1.3063	0.0682	0.0252
7.53	8	0.028	0.972	0.0289	-1.2695	0.0752	0.0288
7.55	9	0.032	0.968	0.0326	-1.2358	0.0821	0.0324
7.55	10	0.036	0.964	0.0362	-1.2047	0.0888	0.0361
…	…	…	…	…	…	…	…
13.53	278	0.989	0.011	4.5397	4.5344	3.0171	2.7651
13.69	279	0.993	0.007	4.9452	4.9416	3.2115	2.8951
14.41	280	0.996	0.004	5.6384	5.6366	3.5342	3.0900
Linear regression coefficients:			A	7.75	8.57	7.22	7.43
			B	1.51	1.17	2.22	2.25
			RMSE	0.265	0.218	0.143	0.092

Table A.4: Estimated parameters extreme value distributions
(data from PoT analysis, sector $150^o N - 360^o N$, threshold $7.5m$, $N_s = 12.2$)

	Linear regression				Method of moments			
	$\hat{\alpha}$	$\hat{\beta}$	$\hat{\gamma}$	RMSE	$\hat{\alpha}$	$\hat{\beta}$	$\hat{\gamma}$	RMSE
Expon.	–	1.51	7.75	0.265	–	1.47	7.76	0.267
Gumbel	–	1.17	8.57	0.218	–	1.15	8.57	0.220
Weibull	1.37	2.22	7.22	0.143	1.37	2.18	7.24	0.146
GPD	-0.24	2.25	7.43	0.092	-0.24	2.22	7.44	0.093

Table A.5: Design storm wave height predictions (data from PoT analysis, sector $150^o N - 360^o N$, threshold $7.5m$, $N_s = 12.2$)

R [years]	1	10	20	50	100	**200**	500	1000
Q [-]	$8.2 \cdot 10^{-2}$	$8.2 \cdot 10^{-3}$	$4.1 \cdot 10^{-3}$	$1.6 \cdot 10^{-3}$	$8.2 \cdot 10^{-4}$	$\mathbf{4.1 \cdot 10^{-4}}$	$1.6 \cdot 10^{-4}$	$8.2 \cdot 10^{-5}$
Expon.	11.5	15.0	16.0	17.4	18.4	**19.5**	20.9	21.9
Gumbel	11.4	14.2	16.1	16.1	16.9	**17.7**	18.8	19.6
Weibull	11.6	14.2	14.9	15.8	16.5	**17.2**	18.0	18.6
GPD	11.7	13.8	14.3	14.8	15.1	**15.4**	15.6	15.8

= 1.47 m.

The expressions to calculate $\hat{\beta}$ and $\hat{\gamma}$ are different for each of the four extreme value distributions, see section A.7. For the exponential distribution, for example, we have $\hat{\beta} = s_H = 1.47$ m and $\hat{\gamma} = \overline{H} - \hat{\beta} = 7.76$ m.

For the Weibull and GPD distributions, the result is dependent on the initial guess of $\hat{\alpha}$, as before. The procedure to find the best fit value is the same as for the linear regression method, i.e. guess $\hat{\alpha}$, find $\hat{\beta}$ and $\hat{\gamma}$, calculate $H_{ss,pred,i}$ and RMSE, and update $\hat{\alpha}$ until the lowest RMSE is obtained.

A.6.7. EXTREME VALUE DISTRIBUTIONS - FINAL RESULTS

The results of the parameter estimation, for both the linear regression method and the method of moments, are given in table A.4. For the Weibull and GPD distributions, these are based on the $\hat{\alpha}$ values that gave the lowest RMSE.

With these results we can calculate the design wave height for any given target return period R. Note that so far we have not used the 'design' concepts of R or N_s in our extreme value distribution fit, we have only looked at the statistical parameters like exceedance probability Q. Now is the time to translate these results back to design parameters. For any given value of R the corresponding exceedance probability can be calculated using equation A.7:

$$Q = \frac{1}{R \cdot N_s}$$

In our example, for $R = 200$ years and $N_s = 12.2$, our design wave is the wave that corresponds to an exceedance probability $Q = 1/(200 \cdot 12.2) = 4.1 \cdot 10^{-4}$. We can calculate the corresponding wave height using inverse of the extreme value distributions and the fitted parameters. For the exponential distribution we find for instance $H_{ss,200y} = \hat{\gamma} - \hat{\beta} \ln Q = 7.75 - 1.51 \ln 4.1 \cdot 10^{-4} = 19.5$ m. For all other distributions and return periods the calculation results are given in table A.5. Figure A.7 depicts the results graphically, along with the original data points. These points are the $(H_{ss,i}, Q_i)$ points of table A.3 where the Q_i coordinate has been transformed to R_i using equation A.7 again:

$$R_i = \frac{1}{Q_i \cdot N_s}$$

It is clear from both the RMSE values in table A.4 and the visual representation of figure A.7 that the Weibull and GPD distributions fit our data best (in this example). The final answer to the question *What is the 200 year wave height?* would therefore be $H_s = 15.4m$ or $H_s = 17.2m$, depending on which of the two distributions is considered most reliable. The GPD results have a better statistical fit, the Weibull results are more conservative which could be appropriate considering the uncertainties of the whole procedure. This choice is up to the professional judgment of the designer.

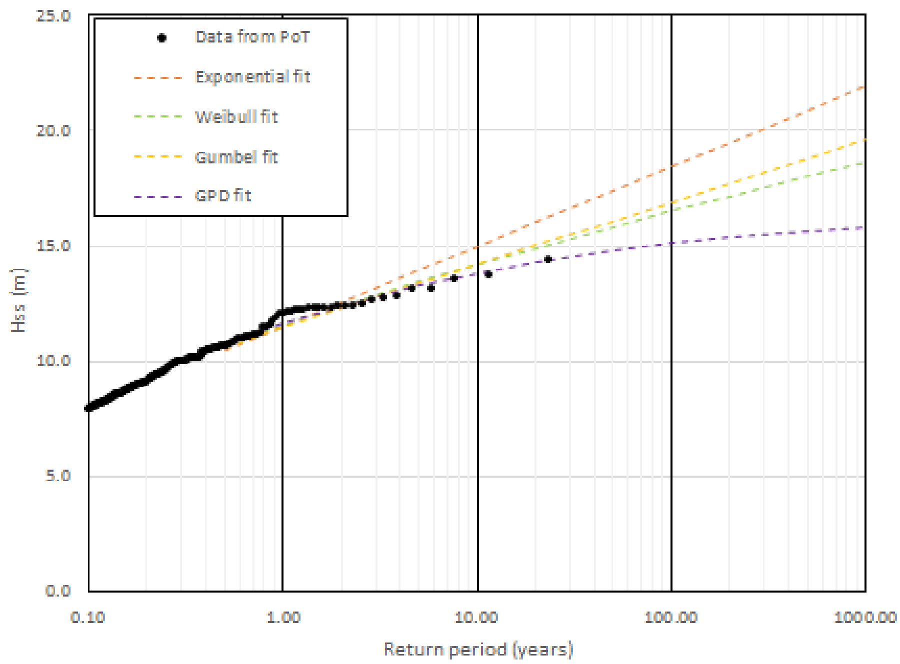


Figure A.7: Result from extreme value analysis, example dataset (data from PoT analysis, sector $150^o N - 360^o N$, threshold $7.5m$, $N_s =$ 12.2)

A.6.8. WHAT TO DO IF ONLY SUMMARIZED DATA IS AVAILABLE?

In the above we were in the position that we had the full timeseries data available, so we could make a Peak-over-Threshold analysis, but this is not always the case. Table A.8 shows exactly the same data as the original dataset, but now summarized into a frequency table of wave height vs month of the year, in bins. Could we have made an estimate of the 200 year wave height as well, if we would have had only this table at our disposal?

The answer is yes, but with a lower accuracy and with more uncertainties. The first drawback of this table is that we cannot analyse whether the waves come from a homogeneous population in terms of wave period or direction, so we are forces to take the entire dataset without any filters applied. Second, we now have only 29 wave height bins, so 29 data points in our analysis, so it makes little sense to apply a threshold since that would reduce the number of data points even further.

If we step over these issues, we can make an estimate of the design wave by applying the modified procedure described in section A.4.3. First we rank the wave heights (which they already are) and we consider the average value of the bin to be representative for the entire bin, in other words we represent the bin '0.0 - 0.5 m' by $H_s = 0.25$ m. We can then calculate the adapted plotting positions from equation A.14. Here, we are not interested in the distribution of the wave heights over the months, so we only use the total number of observations per bin. Finally, we can calculate the linearized X-axis variables as before and we can draw the linear regression lines for each distribution. The results are given in table A.6

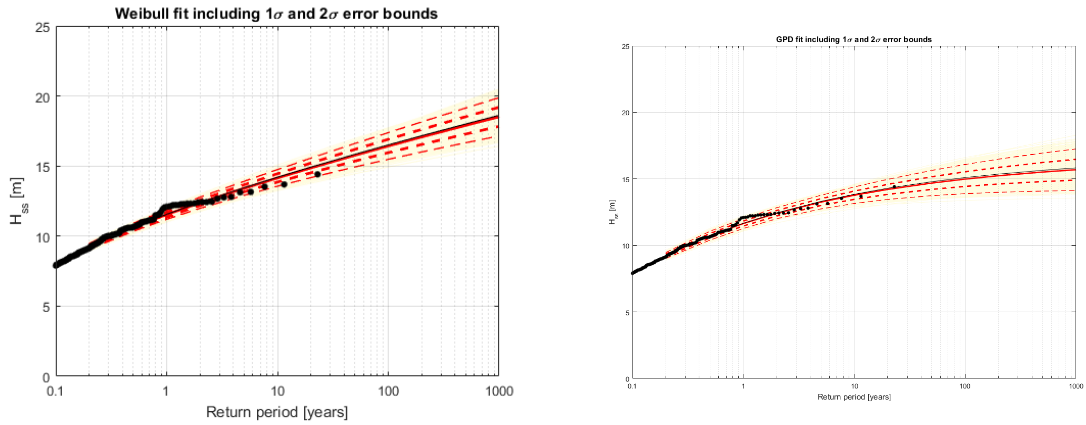
Let's take the exponential distribution as our example again. We find $\hat{\gamma} = A = 2.03$ m and $\hat{\beta} = B = 1.29$ m. We can now calculate the target 200 year wave height, if we know the number of storms per year N_s. In this method, N_s is not well defined, but we can estimate it as $N_s = 365 \cdot 24/t_s$ where t_s is an estimate of a representative duration of the 'storms' in our dataset. As a first guess, let's take $t_s = 3$ hours, so $N_s = 365{\cdot}24/3 = 2920$. Our target exceedance probability Q then becomes $Q = 1/(N_s{\cdot}R) = 1/(2920{\cdot}200) = 1.71{\cdot}10^{-6}$. For the exponential distribution, our estimate of the design wave height follows as $H_{ss,200y} = \gamma - \beta \ln Q = 2.03 - 1.29 \ln(1.71 \cdot 10^{-6}) = 19.2$ m. For the other distributions, the 200 year design wave height is given in table A.7 as function of the estimate of t_s. A comparison to the results from our PoT analysis shows that the results for $t_s = 3$ hours or 6 hours are the closest match in this example, but of course in reality we cannot make this comparison because we have not made a PoT analysis. This introduces an extra element of uncertainty in the prediction, which is why estimating a design wave height on the basis of a frequency table only is not to be preferred.

Table A.6: Extreme value distribution fit (ranked data from summarized observations)

Data					Expon.	Gumbel	Weibull $\alpha = 1.60$	GPD $\alpha = -0.14$
$H_{ss,i}$	$Obs.$	n	P_i	Q_i	$X_{E,i}$	$X_{G,i}$	$X_{W,i}$	$X_{P,i}$
0.25	6	6	0.00009	0.99991	0.0001	–	0.0029	0.0001
0.75	2480	2486	0.03699	0.96301	0.0377	-1.1931	0.1289	0.0376
1.25	8440	10926	0.16257	0.83743	0.1774	-0.5970	0.3393	0.1752
1.75	9572	20498	0.30499	0.69501	0.3638	-0.1718	0.5316	0.3547
2.25	9117	29615	0.44064	0.55936	0.5810	0.1990	0.7122	0.5580
2.75	7790	37405	0.55655	0.44345	0.8132	0.5344	0.8787	0.7686
3.25	6807	44212	0.65783	0.34217	1.0724	0.8703	1.0447	0.9958
3.75	5516	49728	0.73990	0.26010	1.3467	1.1999	1.2045	1.2274
4.25	4419	54147	0.80565	0.19435	1.6381	1.5320	1.3613	1.4638
4.75	3261	57408	0.85417	0.14583	1.9253	1.8475	1.5060	1.6877
…	…	…	…	…	…	…	…	…
13.25	5	67202	0.99990	0.00010	9.1697	9.1696	3.9946	5.1643
13.75	4	67206	0.99996	0.00004	10.0170	10.0169	4.2214	5.3856
14.25	2	67208	0.99999	0.00001	11.1156	11.1156	4.5051	5.6361
Linear regression coefficients:				A	2.03	2.61	0.01	0.70
				B	1.29	1.21	3.33	2.42
				RMSE	0.857	0.630	0.282	0.140

Table A.7: Results for $H_{ss,D}$ using the summarized data method

$R_D = 200 years$						
$t_s[h]$	$N_s[-]$	$Q[-]$	Expon.	Gumbel	Weibull	GPD
3	2920	$1.71 \cdot 10^{-6}$	19.2	18.7	16.8	15.3
6	1460	$3.42 \cdot 10^{-6}$	18.3	17.8	16.2	15.0
9	973	$5.14 \cdot 10^{-6}$	17.8	17.3	15.9	14.8
12	730	$6.85 \cdot 10^{-6}$	17.4	17.0	15.7	14.7
PoT			19.5	17.7	17.2	15.4



(a) Weibull distribution (b) Generalized Pareto distribution

Figure A.8: Example of results obtained from a bootstrapping analysis

A.6.9. SIMPLE WORKABILITY ANALYSIS

We have seen that summarized data like in table A.8 is not th ebest option for determining the design wave height. But such data can be very useful if you want to study the *operational* wave climate at a project site instead of the *extremes*. For instance, such a table could give valuable information regarding the workability of construction equipment (see also chapter 12). Suppose for example that we are using a construction method in which we cannot work for wave conditions higher than 4 m. (Typical limits for construction equipment are much lower, but a threshold of 4 m could for instance refer to unsafe working conditions because of overtopping on an unfinished breakwater).

We can see from this table that for example in January there are 2722 observations lower than 4 m, out of a total of 5704, so our workability is 2722/5704 = 47.7%. This means that, on average, we can expect to work 47.7% of the time, and we can use that number in our estimate of the project duration. We can also see that the workability in the summer months is much better than in January, so we can use this information to plan our project execution phase.

Of course this workability will vary from one year to the next, but this information is not available in our table, so we can only estimate the average workability now. If we would be interested in the variation between years we would have to find a better source of wave data, for example the complete underlying time series on which our table is based. The quality of our workabililty estimate depends on the extent in which we are able to find such information.

A.6.10. UNCERTAINTY IN THE PREDICTION

Now that we have calculated the target wave conditions for $R = 200$ years by various methods, we can go on to estimate the uncertainty in our prediction. The Goda method can only be applied for the Gumbel and Weibull distributions. In our example we have an original dataset with a total of $N = 280$ data points, N_s = 12.2 storms per year and a standard deviation of $\sigma_x = 1.47$ m as established earlier.

In our example we found a best-fit Weibull distribution for $\hat{\alpha} = 1.4$, so for $R = 200$ years we find $y_R = 5.69$ m from equation A.12. From table A.3 we find Goda's coefficients $a_1 = 2.05$, $a_2 = 11.4$ and $c = 0.4$. Calculating the prediction uncertainty (equation A.11) gives $a = 2.065$, $\sigma_z = 0.458$ m and $\sigma_H = \sigma_x \cdot \sigma_z = 0.67$ m. Compared to a mean prediction for the 200 year wave of $H_{ss} = 17.2$ m, this is approximately a 4% error.

For the Gumbel distribution we would find $y_R = 7.80$ m and $a_1 = 0.64$, $a_2 = 9.0$ and $c = 0$. This gives $a = 0.64$, $\sigma_z = 0.377$ m and $\sigma_H = \sigma_x \cdot \sigma_z = 0.56$ m. The mean prediction for the 200 year wave was $H_{ss} = 17.7$ m for the Gumbel distribution, this is approximately a 3% error.

If we apply the bootstrap method (in a Monte Carlo for 1,000 simulations, using Matlab) we get the results as given in A.8. The uncertainty bounds for the Weibull distribution are of the same order of magnitude, but slightly smaller, than the uncertainty found using the Goda method. For $R = 200$ years, we find approximately $\sigma_H = 0.55$ m.

Table A.8: Example of a simple workability assessment

Wave height	Observations per month												All year
[m]	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
0.0 - 0.5	0	0	0	0	0	6	0	0	0	0	0	0	6
0.5 - 1.0	10	10	42	67	452	492	602	489	113	126	57	20	2480
1.0 - 1.5	116	130	335	476	1201	1357	1577	1391	685	633	307	232	8440
1.5 - 2.0	305	272	421	817	1048	1185	1433	1338	871	899	504	479	9572
2.0 - 2.5	480	366	590	896	960	930	912	970	967	822	620	604	9117
2.5 - 3.0	547	371	621	851	710	673	496	710	788	787	621	615	7790
3.0 - 3.5	647	517	681	703	498	427	333	391	625	657	677	651	6807
3.5 - 4.0	617	572	561	539	335	216	208	204	479	531	700	554	5516
4.0 - 4.5	690	581	492	382	226	122	59	110	320	427	513	497	4419
4.5 - 5.0	532	475	415	290	102	67	37	47	224	230	392	450	3261
5.0 - 5.5	420	390	336	174	77	27	24	26	156	163	330	359	2482
5.5 - 6.0	320	340	279	110	50	11	18	14	113	139	247	287	1928
6.0 - 6.5	263	305	223	54	26	7	1	6	63	95	174	289	1506
6.5 - 7.0	203	231	179	43	15		2	4	46	70	103	170	1066
7.0 - 7.5	146	162	133	35	4		2	2	34	59	82	141	800
7.5 - 8.0	110	136	104	26				2	19	17	44	97	555
8.0 - 8.5	77	107	84	16					10	19	54	72	439
8.5 - 9.0	63	75	50	16					5	11	25	62	307
9.0 - 9.5	45	50	39	8					1	3	16	39	201
9.5 - 10.0	30	45	34	4					1	7	9	27	157
10.0 - 10.5	28	23	26	5						1	18	22	123
10.5 - 11.0	24	17	17	4						1	8	17	88
11.0 - 11.5	13	11	10	3						2	7	9	55
11.5 - 12.0	4	9	13	1						1	4	6	38
12.0 - 12.5	6	5	10							4	4	3	32
12.5 - 13.0	4		5								1	2	12
13.0 - 13.5	1		3								1		5
13.5 - 14.0	1		1								2		4
14.0 - 14.5	2												2
Workability for threshold of H_s = 4.0 m													
Obs. ≤ 4 m	2722	2238	3251	4349	5204	5286	5561	5493	4528	4455	3486	3155	49728
Total obs.	5704	5200	5704	5520	5704	5520	5704	5704	5520	5704	5520	5704	67208
Workab. [%]	47.7	43.0	57.0	78.8	91.2	95.8	97.5	96.3	82.0	78.1	63.2	55.3	74.0

A.6.11. CORRELATION BETWEEN WAVE HEIGHT AND WAVE PERIOD

We have now estabished the wave height corresponding to our target $R = 200$ years conditions, but for some calculations we will need information about the associated wave period as well. A first estimate can be obtained by finding a correlation of the type $T_m = a \cdot \sqrt{H_s}$. We can find easily from the PoT results in table A.1 that in our case the best fit value for $a = 3.9$ (by calculating $T_m / \sqrt{H_s}$ for each data point and taking the average).

If we accept the Weibull distribution and $H_{ss,200y} = 17.2$ m as our final answer, then the corresponding wave period would be $T_m = 3.9\sqrt{17.2} = 16.2$ s.

Figure A.9 shows the data points graphically along with the line $T_m = 3.9\sqrt{H_s}$. It is clear that the scatter is quite large (although it appears to reduce for higher wave heights), so the uncertainty in the estimate of the wave period is large. It is left up the professionalism of the designer to do something with this information, for instance by adding a safety coefficient (towards shorter or longer wave periods, depending on the design calculation that is being made), doing a sensitivity analysis or performing a probabilistic calculation.

In this examples we have used T_m as a metric for the wave period, but of course the same procedure applies for other metrics such as $T_{m-1,0}$ or T_p. This choice depends again on the type of calculation that is being made.

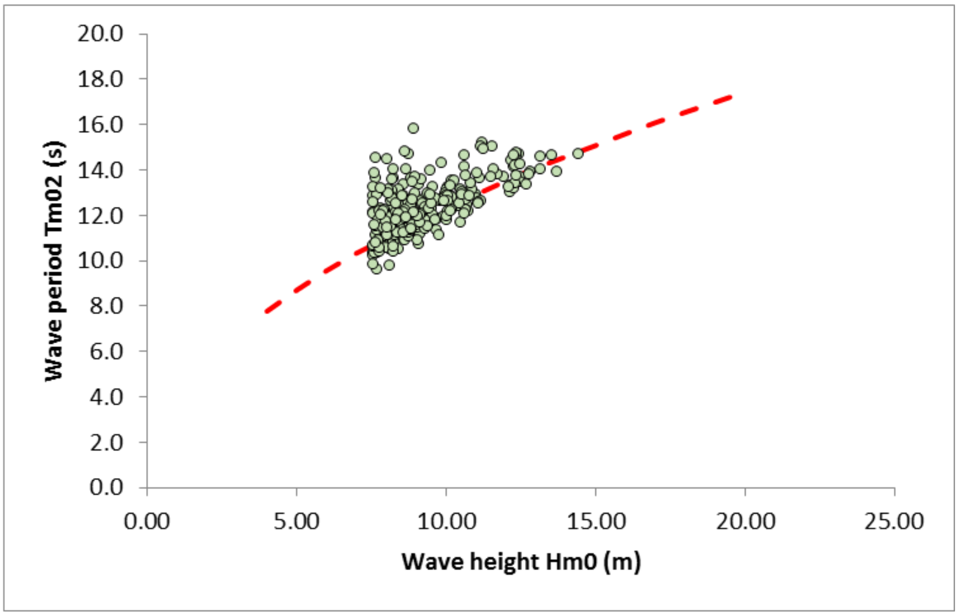


Figure A.9: Correlation between wave height and wave period, example dataset (data from PoT analysis, sector $150^o N - 360^o N$, threshold $7.5m$, $N_s = 12.2$)

A.7. MATHEMATICAL FACTSHEETS

Statistical parameters

$$\begin{aligned} P &= \text{Cumulative distribution function} = CDF = F_H(H) \\ &= \text{non-exceedence probability} \\ &= Pr\left\{\text{Wave height } H_{ss} \text{ is not exceeded } \textbf{per storm event}\right\} \end{aligned}$$

$$\begin{aligned} Q &= 1 - P \\ &= \text{exceedence probability} \\ &= Pr\left\{\text{Wave height } H_{ss} \text{ is exceeded } \textbf{per storm event}\right\} \end{aligned}$$

Design parameters

$$\begin{aligned} \lambda &= Q \cdot N_s \\ &= \text{Expected number of occurrences of a storm with a wave height of at least } H_{ss} \textbf{ per year} \\ &\approx Pr\left\{\text{Wave height } H_{ss} \text{ is exceeded at least once } \textbf{per year}\right\} \\ &= \text{Not strictly a probability in the statistical sense} \\ N_s &= \text{Number of storms per year} \\ R &= \text{Return period} = 1/\lambda \end{aligned}$$

Transformation from statistical parameters to design parameters and back

$$\begin{aligned} R &= \frac{1}{Q \cdot N_s} \\ Q &= \frac{1}{R \cdot N_s} \end{aligned}$$

Parameter estimation

$$\begin{aligned} \alpha, \beta, \gamma &= \text{true parameter values} \\ \hat{\alpha}, \hat{\beta}, \hat{\gamma} &= \text{estimated parameter values from data} \end{aligned}$$

- Linear regression: fitting slope (B) and intercept (A) of regression line through linearized data points using e.g. a least-squares method and estimate parameters as functions of A and B
- Method of moments: calculating sample mean $\overline{H}$ and sample variance s_H^2 of data points and estimate parameters as functions of $\overline{H}$ and s_H. Note the difference between the variance s_H^2 and the standard deviation s_H.

$$\begin{aligned} \overline{H} &= \frac{1}{N} \sum_{i=1}^{N} H_{ss,i} \\ s_H^2 &= \frac{1}{N-1} \sum_{i=1}^{N} \left(H_{ss_i} - \overline{H}\right)^2 \end{aligned}$$

EXPONENTIAL DISTRIBUTION

Non-exceedence

$$P = 1 - exp\left(-\left(\frac{H_{ss} - \gamma}{\beta}\right)\right)$$

Exceedence

$$Q = exp\left(-\left(\frac{H_{ss} - \gamma}{\beta}\right)\right)$$

Inverse

$$H_{ss} = \gamma - \beta \ln Q$$

Linear regression

$$H_{ss,i} = A + B \cdot X_{E,i}$$
$$X_{E,i} = -\ln Q_i$$

$$\hat{\beta} = B$$
$$\hat{\gamma} = A$$

Method of moments

$$\hat{\beta} = s_H$$
$$\hat{\gamma} = \overline{H} - \hat{\beta}$$

WEIBULL DISTRIBUTION

Non-exceedence

$$P = 1 - exp\left(-\left(\frac{H_{ss} - \gamma}{\beta}\right)^{\alpha}\right)$$

Exceedence

$$Q = exp\left(-\left(\frac{H_{ss} - \gamma}{\beta}\right)^{\alpha}\right)$$

Inverse

$$H_{ss} = \gamma + \beta\,[-\ln(Q)]^{1/\alpha}$$

Linear regression (for given choice of $\hat{\alpha}$)

$$H_{ss,i} = A + B \cdot X_{W,i}$$
$$X_{W,i} = [\ln(1/Q_i)]^{1/\hat{\alpha}}$$

$$\hat{\beta} = B$$
$$\hat{\gamma} = A$$

Method of moments (for given choice of $\hat{\alpha}$)

$$\hat{\beta} = \frac{s_H}{\sqrt{\Gamma\left(1 + \frac{2}{\hat{\alpha}}\right) - \left[\Gamma\left(1 + \frac{1}{\hat{\alpha}}\right)\right]^2}}$$
$$\hat{\gamma} = \overline{H} - \hat{\beta} \cdot \Gamma\left(1 + \frac{1}{\hat{\alpha}}\right)$$
$\Gamma(x)$ = Gamma function

GUMBEL DISTRIBUTION

Non-exceedence

$$P = exp\left[-exp\left(-\left(\frac{H_{ss}-\gamma}{\beta}\right)\right)\right]$$

Exceedence

$$Q = 1 - exp\left[-exp\left(-\left(\frac{H_{ss}-\gamma}{\beta}\right)\right)\right]$$

Inverse

$$H_{ss} = \gamma - \beta \ln\left(-\ln\left(1-Q\right)\right)$$

Linear regression

$$H_{ss,i} = A + B \cdot X_{G,i}$$
$$X_{G,i} = -\ln\left(-\ln\left(1-Q_i\right)\right)$$
$$\hat{\beta} = B$$
$$\hat{\gamma} = A$$

Method of moments

$$\hat{\beta} = s_H \cdot \frac{\sqrt{6}}{\pi}$$
$$\hat{\gamma} = \overline{H} - \epsilon\hat{\beta}$$
ϵ = Euler–Mascheroni constant = 0.5772...

GENERALIZED PARETO DISTRIBUTION

Non-exceedence

$$P = 1 - \left(1 + \alpha\frac{H_{ss}-\gamma}{\beta}\right)^{-1/\alpha}$$

Exceedence

$$Q = \left(1 + \alpha\frac{H_{ss}-\gamma}{\beta}\right)^{-1/\alpha}$$

Inverse

$$H_{ss} = \gamma + \beta\left(\frac{Q^{-\alpha}-1}{\alpha}\right)$$

Linear regression (for given choice of $\hat{\alpha}$)

$$H_{ss,i} = A + B \cdot X_{P,i}$$
$$X_{P,i} = \frac{Q_i^{-\hat{\alpha}}-1}{\hat{\alpha}}$$
$$\hat{\beta} = B$$
$$\hat{\gamma} = A$$

Method of moments (for given choice of $\hat{\alpha}$)

$$\hat{\beta} = s_H\sqrt{(1-\hat{\alpha})^2(1-2\hat{\alpha})}$$
$$\hat{\gamma} = \overline{H} - \frac{\hat{\beta}}{1-\hat{\alpha}}$$

B

Approaches to design reliability

B.1. Introduction

When we are designing a breakwater we will obviously want it to be safe, in other words we want to be reasonably sure that it will withstand the anticipated loads during a certain lifetime. But how do you analyze that? How do you express structure reliability in a meaningful, quantitative way that can be used for design of a breakwater? And then, how safe is safe enough?

As it happens, the design practice of breakwaters is mainly empirical and there is no standardized method to study these topics. In this chapter we will introduce a handful of possible methods and comment on them using a worked example.

B.2. Background literature

The answers to this type of questions are usually treated in national or international Standards or Building Codes. These standards however do not apply to coastal structures such as breakwaters, so the breakwater designers will have to make a lot of the reliability-related choices themselves and should be aware of the associated issues, perhaps more than in any other field of (civil) engineering.

The Rock Manual, which is sometimes seen as a *de facto* informal design standard for breakwaters (despite its lack of official status) does not mention reliability at all. More relevant guidance can be found in the Coastal Engineering Manual (chapter VI) which deals with reliability issues, or in the PIANC publication *Development of a partial coefficient system for the design of rubble mound breakwaters* (PIANC-MARCOM 12-F) . In fact, these two publications refer to essentially the same method. This guidance does not have any formal status either, and as we will see below, may also have a limited range of applicability.

In Europe, the relevant Building Code would have been the Eurocode (EN 1990), but as observed before this code is not (yet) applicable to coastal structures. Nevertheless, many concepts from the general part of the Eurocode can be used to provide a theoretical framework for the study of breakwater design reliability.

Finally, for The Netherlands, we can get some guidance from the CUR publication "Kansen in de civiele Techniek" (CUR 190) which corresponds to large extent to the lecture notes CIE4130 "Probabilistic Design" of Delft University. In this present document we have sought to be consistent with this publication wherever possible.

B.3. Definitions

Before we start it is useful to state a few definitions. These definitions will follow the concepts from the literature listed above as much as possible. At some points we will have to deviate from these concepts because of the typical application for coastal structures. Also, for some parameters the definitions or notation will vary across the literature cited above, in which case we will use the definition or notation most commonly applied in conventional breakwater design practice.

- We will consider our breakwater to be a *system* that can be in a certain *state* of stability. That state can be described by two parameters, *load* and *resistance*, that we will denote S and R respectively. The theoretical state in which $S = R$ precisely is called the *limit state*. If the limit state is exceeded, in other words if $S > R$, the system is said to have failed.

- It is often convenient to express a *limit state function* $Z = S - R$, so failure is defined as a state where $Z < 0$. If we are able to express the probability distributions of S and R in a statistically meaningful way, we can calculate the *probability of failure* $p_f = Pr\{Z < 0\}$.
- The *reliability* of the system is then defined as the probability of survival, which is the opposite of the probability of failure $p_s = 1 - p_f$. This can also be expressed as a *reliability index* $\beta = \Phi^{-1}(p_s) = -\Phi^{-1}(p_f)$ where Φ denotes the inverse cumulative standard normal distribution.
- We could make a design by selecting a certain dimension (such as armour block size, crest height etc) related to the resistance R and then demonstrate that the resulting reliability using these definitions is larger than a given pre-set standard. Such as method is referred to as a *probabilistic* design method,. It can be any of the following three general types, depending on how the actual probability calculations are made:

  - Level III approach: simulating the behavior of the system including the uncertainty of all relevant parameters with their actual distributions, possible non-linear behavior and mutual correlations in a Monte Carlo simulation or numerical integration.
  - Level II approach: approaching the true behavior of the system by simplifying the uncertainties to normal distributions and linear relationships, and solving for the resulting probability of failure analytically, e.g. in a FORM analysis.
  - Level I approach: defining *characteristic* values S_k and R_k for the load and resistance (usually some 'safe' value selected from their distribution such as the 95% percentile), and combining these with a *partial factor* γ. The *design value* for load then becomes $S_d = S_k \cdot \gamma_S$ and the design value for the resistance becomes $R_d = R_k/\gamma_R$. The value of the partial factors is typically a function of the desired target reliability and has been calibrated on the result of many Level II or Level III calculations, so the designer can be sure that the reliability of the system meets the target reliability if $S_d < R_d$ without having to make the actual Level II or Level III calculations themselves. The results of a Level I analysis consist of the definition of the characteristic values and the value of the partial factors. These are published in tables and reports and commonly form the basis of (inter)national Structural Design Codes such as the Eurocode system. Level I methods are sometimes also referred to as semi-probabilistic methods, as opposed to 'full probabilistic' i.e. Level II or III methods.

- Any method that is not probabilistic in this sense is called a *deterministic* method. Note that a deterministic method can still be based on probabilistic concepts like a target reliability. The difference is that these concepts are only used to calculate a part of the limit state (usually the load, or even a simplification of the load). So, since we are then not analyzing the entire limit state function we cannot calculate the failure probability. So in a deterministic method we cannot 'prove' or show that our design meets the target reliability. That is the principal difference with a probabilistic method.
- A system can have more than one single limit state. The first distinction can be made by observing that a system can be broken down into subsystems that can each fail in many different ways called *failure modes* or *failure mechanisms*. For breakwater design such failure modes could be loss of armour stability, failure of the toe structure or excessive overtopping. An overview is given in Figure B.1, see also Chapter 14. In principle, these failure modes can be recombined into failure of the overall system by means of a *failure tree*.
- A second way to analyze various limit states for a system is by looking at the consequences of failure. It is common to distinguish between an *Ultimate Limit State (ULS)* which corresponds to complete loss of the original function of the system, and one or more *Serviceability Limit States (SLS)* which correspond to, usually temporary, loss of availability, limitation of use, inconvenience or hindrance. It will be clear that the target reliability of an SLS limit state can typically be lower than the target for a ULS limit state.
- Each limit state (ULS or SLS), for each failure mode, can be associated with a *design criterion* which defines the acceptable state of the system in a quantitative way. For example, for breakwater design the design criterion for the failure mode 'overtopping' could be a certain acceptable mean overtopping rate Q [l/sm], for rock stability it could be an acceptable level of damage S or N_{od}.
- In order to make calculations, the load and the resistance must be expressed in the same units. The load on a coastal structure is almost always expressed as the wave height (in meters), so we will express the resistance also in meters, as a *critical wave height* H_c, being the maximum wave height that a structure can withstand. Since most coastal structure stability formulas have a shape like $(H_s/\Delta D) = f_d(...)$ where f_d is some *design formula*, we can express the resistance as $R = H_c = (\Delta D) \cdot f_d$. The limit state function

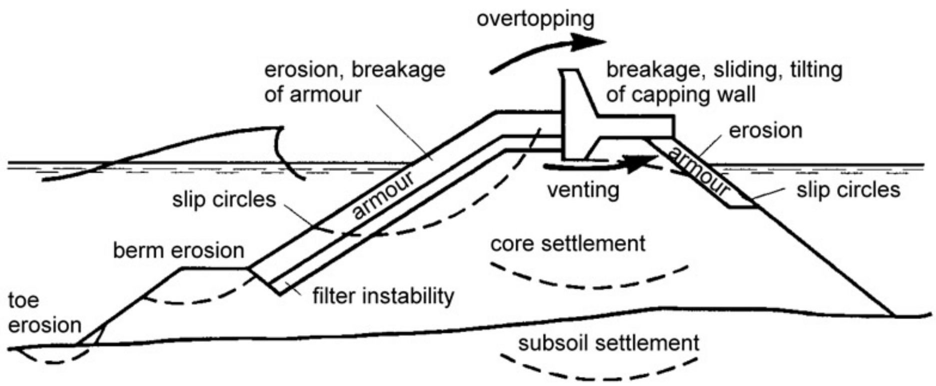


Figure B.1: Failure mechanisms for rubble-mound breakwaters (BURCHARTH 1992)

in a level I method can then be written as:

$$\frac{(\Delta D) \cdot f_d}{\gamma_R} - \gamma_H \cdot H_{ss} \leq 0$$

The design function f_d can have *coefficients* and (structural or geometrical) *parameters*. The parameters represent the physical properties of the construction material (such as rock diameter or density) or the structure (such as crest level or slope) and can be specified by the designer. Coefficients are part of the empirical model itself and are used to transform the parameters into the calculated resistance. They are usually calibrated on the basis of model tests and cannot be selected by the designer. Examples include the coefficients $c_{pl} = 6.2$ and $c_s = 1.0$ in the Van der Meer formula for rock stability. It should be noted that parameters and coefficients can both be uncertain and that both should be included in a reliability analysis.

- Finally, any structure will have a certain *lifetime* T_L, so we need to bring the dimension of time into play somehow. This can be done by defining probabilities in the context of a *reference period* that is short on the scale of the structure lifetime, but long enough to ensure that separate events are statistically independent and not correlated by e.g. seasonal effects. We will follow the standard convention to take a reference period of one year, in other words we will discuss *probabilities per year* unless mentioned otherwise. The probability of failure *during the reference period of one year* follows from the Poisson distribution, see also Appendix A:

$$p_{f,y} = 1 - e^{-\lambda} \tag{B.1}$$

where λ is the expected number of failures per year over a long period of time. In practical design situations it is more common to use its inverse, the *return period* $R = 1/\lambda$, instead. The probability of failure *during the lifetime of the structure* $p_{f,TL}$ can then be calculated as follows:

$$p_{f,TL} = 1 - (1 - p_{fy})^{T_L} = 1 - e^{-T_L/R} \tag{B.2}$$

B.3.1. DISCUSSION

Now that we have defined our framework, we can discuss the application of these concepts for breakwater design. The first simplification that we will make is that we will only look at limit state(s) for each failure mode separately, and we will not make an attempt to combine these into some kind of definition of the failure of the overall breakwater. In theory such a thing could be done, for example by defining the function of the breakwater as 'providing protection of the harbour interior to excessive wave agitation'. We could then for instance accept a large probability of failure of a subsystem like the toe berm as long as we analyze how this would contribute to an increase in the probability of exceeding the wave conditions in the harbour. Such an analysis, although theoretically sound, is never made in breakwater design practice because the system is usually considered to be too complicated to do this in a meaningful way.

A second, related issue is the fact that it is not very common to define such a high-level Ultimate Limit State as 'loss of functional protection of the port' anyway. Commonly, Ultimate Limit States are defined as 'start of damage' and design criteria are selected accordingly (such as '$S = 2$'). This is a typical feature of breakwater design which is not in line with most Building Codes, or in fact with the new (2017) Flood Risk standards of The Netherlands, which all define ULS as e.g. collapse of a building or breaching of a flood defense. It also means that there is some undefined margin of 'implicit safety' (or *residual strength*) present in most breakwater designs as there is still a difference between exceedence of the design limit state and actual catastrophic failure of the breakwater. This depends of course on the type of breakwater and the armour material, for instance for rock slopes this margin can be rather large where for armour systems that exhibit sudden 'brittle' failure like some Concrete Armour Units, this margin is much smaller. Because of this dependence on the armour system this concept of residual strength cannot be generalized and quantified, so it is not further discussed in the calculation examples of this chapter. It is important however to keep in mind because it is an important ingredient in the design specifications of many breakwater projects. It is sometimes also used as a shortcut for system considerations, as in 'we can select a less stringent design criterion (and accept more damage) for the toe berm because failure does not immediately lead to loss of functionality of the breakwater'. It should be clear that such kind of reasoning based on engineering judgment is not fully valid from a strict theoretical viewpoint on reliability-based design, although it is very common in design practice and usually universally accepted.

Another source of implicit safety is the practice to *round up* the calculated rock sizes or concrete element volumes to the nearest available standard class or size. This rounding can be substantial (e.g. if a median mass of, say, $M_{50} = 2500$ kg is calculated this can be rounded to the nearest larger standard EN13383 armour class, which is $3-6$ T with a median mass of $M_{50} = 4500$ kg, or 80% larger. In terms of d_{n50} this is still roughly a 20% increase. Again, this 'safety margin' is very strongly case-dependent (another design with for instance a calculated median mass of $M_{50} = 4400$ kg would also have been rounded to $3-6$ T) and besides, a design can always be optimized by using for instance non-standard gradings. For these reasons, this source of implicit safety is not further discussed here either.

Finally, it is important to realize that the theoretical framework of reliability theory only applies to uncertainties that are already present in the available data and that can be analyzed in a rational and mathematical way, using known or assumed distributions of relevant parameters, perhaps augmented by an estimate of foreseeable future trends like climate change. These are not the only sources of uncertainty related to the future functionality of the structure. For instance, future changes in the use of the structure can be implemented (for instance, continued trends in shipping require the port to be operational for different ships that need lower wave conditions inside the port). Or unforeseen circumstances can occur that increase the anticipated load on the structure at some point in the future, like the construction (or removal) of coastal structures in the vicinity. Or maybe a margin is required to anticipate potential execution inaccuracies, or maintenance and inspection issues. All these issues may require the definition of an additional safety margin called a *robustness factor*, over and above the factors that follow from a reliability analysis. It is usually not in the powers of the designer to anticipate all of these circumstances, so selection of an appropriate robustness factor, if any, should be done in a joint effort between all stakeholders. No general guidance for robustness factors can be given and this topic is not further discussed in these lecture notes.

B.3.2. SOURCES OF UNCERTAINTY

Before we start our discussion of the most common design method for rubble mound breakwaters, we will list the principal sources of uncertainty involved in the design considerations. These are:

- Uncertainty in the design load. The most important load parameter in breakwater design is the storm wave height. The occurrence of storms, and the magnitude of the wave field if one occurs, is a stochastic process and therefore the design value of the wave height during the lifetime of the structure is uncertain.
- Uncertainty in the prediction of the wave climate. We can model the wave climate with an appropriate extreme value distribution like Weibull or Gumbel (see Appendix A), but such a process is based on extrapolation of observed storm data and is therefore somewhat inaccurate itself. This leads to a bandwidth of predicted extreme values, which is a second source of uncertainty.
- Uncertainties in the load caused by unknown or variable correlations between wave height and water level, wave height and wave period, or wave height and wave direction. In some cases it could be appropriate to study the joint probability of for instance extreme wave heights and water levels, but such an analysis is beyond the scope of these lecture notes.
- Uncertainty in the resistance, because the response of a structure under a given wave load can only be predicted within a certain accuracy, consisting of three parts:

- First, the design formulas that we use to calculate for example the required concrete element size are based on empirical evidence, usually from laboratory test data, which inhibit a certain amount of scatter around their predictions. This uncertainty can be modeled by defining the coefficients of the design formula as stochastic variables with a mean and a standard deviation. Sometimes these values have been published by the researcher who developed the design formula, but this is not always the case.
- Second, the range of validity of these prediction methods is always limited to the range that was actually included in the underlying laboratory tests, and this range can sometimes be small. Extrapolation of empirical formulas outside this range cannot always be avoided and is an important extra source of uncertainty
- Third, there may be uncertainties in the properties of the construction materials such as mass density of the armour rocks, or uncertainties pertaining to construction inaccuracies such as the slope angle or the crest level

The second part of the uncertainty in prediction of the resistance, the limited validity of the design formulas, cannot be treated in the formal context of a reliability analysis. The usual way of mitigating this type of uncertainty in a real-life design project is to conduct additional laboratory tests (physical model tests) for the purpose of validating the design under the specific project circumstances. In preliminary design stages, where additional model testing is not feasible for budget or time constraints, this can remain a very important source of uncertainty that requires sound professional judgment from the part of the designer.

For the purpose of our discussion of design methods, we will compare the various methods on the degree in which they treat these sources of uncertainty, summarized into four groups: the wave climate itself (i.e. the uncertainty in design wave height), the bandwidth around the predicted wave climate, uncertainty related to the scatter in the empirical design formulas and the remaining uncertainties (grouped as 'other').

B.4. DETERMINATION OF TARGET RELIABILITY

In both the probabilistic and deterministic design approaches, a design process always starts with the specification of the target reliability, in other words with a statement about "how safe the structure should be". The difference between the two approaches is that in a probabilistic approach it is formally *proven* that the target reliability is achieved, while in a deterministic method the target reliability is only used to calculate the design load conditions and the proof that the design meets the specification is left implicit. Usually, target reliabilities need to be specified for both ULS and SLS conditions.

This section deals with the question how this target reliability can be determined. One way, which works particularly well for determining target reliability for ULS, is to define an acceptable probability of failure during the lifetime of the structure ($p_{f_{TL}}$) and calculate the required return period from there by inverse application of equations B.1 and B.2:

$$p_{fy} = 1 - (1 - p_{f_{TL}})^{1/T_L}$$

and

$$R = \frac{1}{-\ln(1 - p_{f,TL})}$$

When R is relatively large (say > 100 years) this can be simplified by the approximation:

$$R = \frac{1}{p_{fy}}$$

These relationships can also be read from the graph given in Figure B.2

Although this may seem like shifting the problem to another time scale, it is usually easier to estimate or discuss failure probabilities at this scale since it relates directly to economic considerations like loss of investment, downtime of the port et cetera. The corresponding lifetime failure probabilities are typically quite large, in the order 5-20% or higher, so for typical lifetimes of 20-50 years the failure probability will be roughly in the range 1/100 to 1/1000 per year, or equivalently a design return period of 100-1000 years. This is in contrast to e.g. residential buildings or flood defenses which typically have much stricter standards. This is understandable since for coastal structures like breakwaters the risk of loss of life in case of failure is very low, with only moderate economic damages and small environmental impact. This is of course only true in

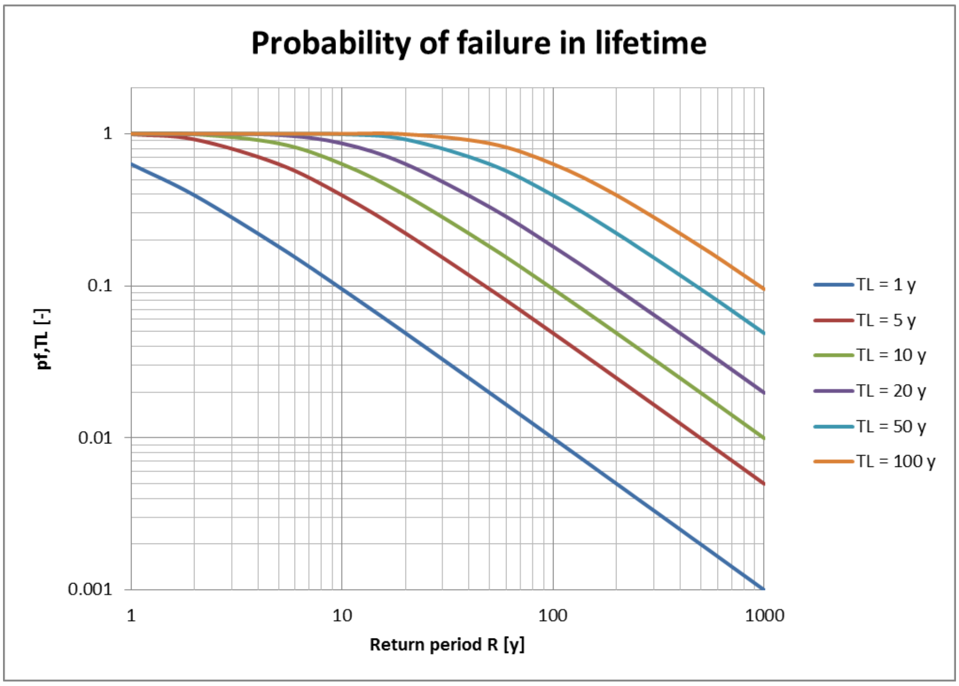


Figure B.2: Relationship between probability of failure, lifetime and return period

general terms, a specific analysis must be made for every project. When the breakwaters are protecting vital port infrastructure, or if the breakwaters are integrated into a coastal defense system against flooding, other considerations can be made.

Also, some national codes have defined 'Importance classes' for structures based on socio-economic or environmental impact of failure, such as the building codes of Spain or Italy. These classes can be applied to coastal structures as well, generally confirming the use of rather high lifetime failure probabilities and subsequently low design return periods. A good overview is given in the recent PIANC publication "Criteria for the selection of breakwater types and their related optimum safety levels" (PIANC/Marcom 196, 2016), which can serve as a first starting point for further deliberations.

For SLS conditions, it might be easier to think in terms of return periods directly. For instance, consider an SLS condition that is specified in terms of non-availability of the access road on top of a breakwater because of wave overtopping. Most of the time we do not want that to happen, but 'every now and then' closing the access road could be acceptable. We can then consider how often we want to accept this, in other words specify how often we want 'every now and then' to be. Every year? Once every 5 years? Twice per year? Such reasoning will give as our return period directly. Of course, a formal consideration using failure probabilities, like in the case of ULS, can be made as well.

Another option is to carry out an economic optimization. In the example of the access road, we may be able calculate the economic damage that occurs as a result of an overtopping event, estimate how often this will happen during the lifetime of the structure (using the Poisson distribution) and calculate the total cost of failure (if necessary, using a Nett Present Value technique). We can then change the design, for instance by raising the crest level and make this calculation again. The initial costs of the second design will be higher (because we are building a higher breakwater) but its failure costs will be lower. In this way an economic optimum can be reached. This is further detailed in chapter 14. This type of procedure has been applied in PIANC/Marcom 196 for a number of breakwater types and failure mechanisms. Usually such an analysis confirms that it is not economically optimal to design for very large return periods, but of course for any a situation the outcome may be different. For more information, see PIANC/Marcom 196 (2016).

B.5. DESIGN APPROACHES

Now that we have established our theoretical framework, we can discuss the various approaches that can be used to design a structure for a given target reliability. Some of these approaches are deterministic, some are (semi-)probabilistic. We will discuss these separately.

B.5.1. CONVENTIONAL APPROACH

The conventional approach (in other parts of these lecture notes also called 'classical' approach) is, as the name suggests, the most commonly applied method. It is deterministic.

In its basic form, all uncertainty is assigned to the load side of the limit state. The probability of failure of the structure is then assumed to be equal to the probability that this load is exceeded. An example of deterministic reasoning is "We want a target failure probability of so-and-so much during the lifetime of our structure, therefore our return period should be some value, say 100 years, *and therefore we should design for the 100 years wave conditions*." The corresponding wave conditions are then determined (using the methods described in Appendix A) and used as input for the design formula f_d. Whatever dimensions follow from the design formula (armour size, crest height et cetera) is accepted as 'the' design.

All other uncertainties (resistance, prediction of the load, or otherwise) are not included. There are some variants on the conventional method in which these additional uncertainties are incorporated in one way or another, for instance by:

- Selecting 'safe' values for the coefficients of the design formula. Examples of such choices are to add one standard deviation to the coefficients of the overtopping formula, or to use the 5% exceedance value of the coefficients in the Van der Meer formula for armour rock stability. These choices will make the design safer, no doubt, but the problem is that it is not clear by how much. These choices are usually *ad hoc* decisions by the designer, sometimes based on recommendations from literature, sometimes based on experience or expert judgment. In any case the total effect of these choices in terms of improving the probability of failure is not quantifiable, which still makes this a deterministic method.
- Selecting an overall safety factor or 'robustness factor' after the design has been made. There could be very valid reasons to include such a factor, as described earlier, but the main problem is the same as with the choice of 'safe' coefficient values: the overall effect on the reliability of the structure is not quantifiable.

B.5.2. LEVEL I APPROACH: PARTIAL SAFETY FACTORS

Probabilistic design methods could be used to overcome these issues, and of the three possible levels, a level I approach might be preferred because of its simplicity and compatibility to building codes from neighboring engineering fields such as the EuroCodes.

PIANC has developed and published such an approach, see PIANC/Marcom 12F (1992). This method can only be applied to rubble mound breakwaters. It defines characteristic values and partial safety factors for both load and resistance, as described below.

Load The characteristic value for the load is defined as the wave height with a return period that is equal to the lifetime of the structure. There is no reasoning behind this, this is simply a matter of definition. So if our lifetime is 25 years, the representative wave height is the 25 year wave. This is denoted as H_{ss}^{tL}. The corresponding partial safety factor is given as:

$$\gamma_H = \frac{H_{ss}^{tpf}}{H_{ss}^{tL}} + {\sigma'}_{F_{Hs}}^{\left(1+\left(\frac{H_{ss}^{3tL}}{H_{ss}^{tL}}-1\right)k_\beta p_f\right)} + \frac{0.05}{\sqrt{p_f N}} \tag{B.3}$$

in which:

H_{ss}^{tL}	= Wave height with a return period that is equal to the lifetime of the structure
H_{ss}^{tpf}	= Wave height with a return period that corresponds to the target failure probability of the structure. (Note that this is the wave height that we would have used in the conventional, deterministic method)
H_{ss}^{3tL}	= Wave height with a return period that is equal to 3 times the lifetime of the structure
p_f	= Target failure probability during the lifetime of the structure (the same as $p_{f,TL}$ in equation B.2)
${\sigma'}_{F_{Hs}}$	= Factor representing the quality of the data used to determine the design wave height
N	= The number of data points used to determine the design wave height
k_β	= Fit coefficient

This formula consists of three terms, we will discuss the second and third term first. They both represent the uncertainty in the extreme value analysis used to determine the design wave height, where the second term focuses more on the quality of the data and the third term on the quantity. The second term is a function of the type of data used in the extreme value analysis, and recommended values are given in Table B.1.

The first term is a factor that brings the design wave height back to the scale of the target reliability. Remember that the safety factor γ_H is to be applied over the representative value, which was defined as H_{ss}^{TL}, so in the design wave height is equal to $H_D = \gamma_H H_{ss}^{TL}$. If there would be no uncertainty in the prediction of the wave climate, the second and the third term would be zero, so the first term would have the effect of bringing the design wave height to $H_D = H_{ss}^{tpf}$. This would be the same wave height as in the conventional, deterministic method. We conclude that the first term is an artifact of the choice to define the representative value as H_{ss}^{TL}. Be aware of this choice, a commonly made mistake is to calculate γ_H and then apply it over H_{ss}^{tpf} instead of H_{ss}^{TL}, which would result in a large overestimation of the design wave height.

Resistance Since the load is expressed as a wave height, the resistance is taken as the critical wave height $(\Delta D) f_d$ as we defined earlier, where the design formula f_d depends on the failure mechanism that we are studying. The uncertainty in the resistance is then caused by the uncertainty in the coefficients of the design formula. These coefficients are commonly fitted onto observations from laboratory measurements and will have a certain scatter, represented by a mean value and a standard deviation. In the PIANC method, the representative value of the resistance is the value that would follow if we would use the *mean value* of the coefficients of the design formula. The partial safety factor is given as:

$$\gamma_z = 1 - \left(k_\alpha \ln p_f\right) \tag{B.4}$$

where k_α is a fit coefficient and p_f is the target failure probability as defined before. The values of the fit coefficients k_α and k_β have been calibrated by PIANC against simulations with Level II and Level III methods, so that application of these partial safety factors will result in the 'correct' structure reliability. The recommended values are a function of the failure mechanism and the design formula, see Table B.2.

Parameters	Method of determination	Typical value for σ'
Wave height		
Significant wave height offshore	Accelerometer buoy, pressure cell, vertical radar	0.05 – 0.1
	Horizontal radar	0.15
	Hindcast, numerical model	0.1 – 0.2
	Hindcast, SMB method	0.15-0.2
	Visual observation (Global wave statistics	0.2
H_{ss} nearshore determined from offshore H_{ss} taking into account typical nearshore effects (refraction, shoaling, breaking)	Numerical models	0.1-0.2
	Manual calculation	0.15-0.35
Other wave parameters		
Mean wave period offshore on condition of fixed H_{ss}	Accelerometer buoy	0.02-0.05
	Estimates from amplitude spectra	0.15
	Hindcast, numerical model	
Duration of sea state with H_{ss} exceeding a specific level	Direct measurements	0.1-0.2
	Hindcast, numerical model	0.02
Spectral peak frequency offshore	Measurements	0.05-0.1
	Hindcast, numerical models	0.05 0.15
Spectral peakedness offshore	Measurements and hindcast with numerical models	0.1-0.2
		0.4
Mean direction of wave propagation offshore	Pitch-roll buoy	
	Measurement of horizontal velocity components and waterlevel or pressure	5°
	Hindcast, numerical model	10°
		15-30°
Water level		
Astro tides	prediction from constants	0.001-0.07
storm surges	numerical models	0.1-0.25

Table B.1: Recommended values for $\sigma'_{F_{Hs}}$ (PIANC 1992)

Discussion The PIANC method is not very often applied in practice. Part of the reason is that the calibration of the k_α and k_β values has only been carried out for a relatively small number of cases, mostly in deep water. This leads to uncertainty in the application of the method, especially in shallow water.

A second reason could be more philosophical. The basic premise behind a level I approach is that it is easier and faster than a level II/III method (and more conservative, too). The designer can then use a level

Formula, type of construction	k_α	k_β
Hudson, rock	0.036	151
Van der Meer, rock, plunging waves	0.027	38
Van der Meer, rock, surging waves	0.031	38
Van der Meer, Tetrapods	0.026	38
Van der Meer, Cubes	0.026	38
Van der Meer, Accropodes	0.015	33
Van der Meer, rock, low crested	0.035	42
Van der meer, rock, berm	0.087	100

Table B.2: Recommended values for k_α and k_β (PIANC 1992)

I method to get a first indicative result, and can resort to level II/III methods if a more accurate answer is needed. For Rubble Mound Breakwater design this premise is questionable. It is relatively easy to set up a full probabilistic calculation, so a direct and accurate estimate of the structure reliability can be obtained directly, circumventing any issues related to calibration of the partial safety factors.

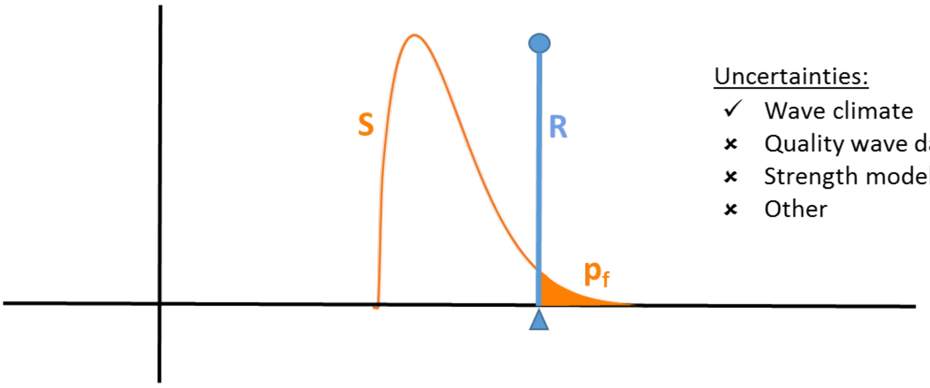
B.5.3. LEVEL II AND III: FULL PROBABILISTIC APPROACHES

Finally, level II (FORM analysis) or Level III methods (Monte Carlo) can be applied as well. There is no prescribed method to do this, but commercially available software packages like `VaP` or `Prob2B` can be used. It is also not very difficult to develop a Monte Carlo script in languages like `Matlab` or `Python`, so purpose-made solutions can be developed for any given design as well.

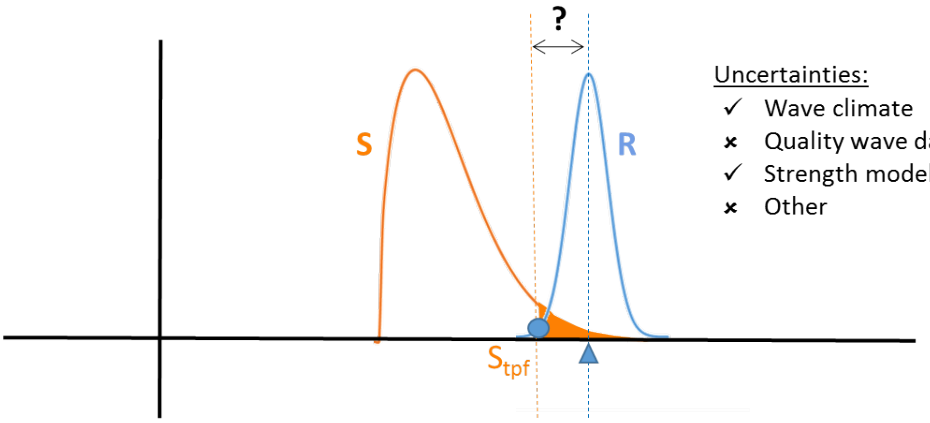
Application of a Level II or Level III method requires quantitative data related to the uncertainty in the wave prediction and the design formula. For the wave prediction this information can be obtained (or estimated) using the methods given in Appendix A. For the design formula, the quality of the fit onto the underlying model data must be analyzed, leading to an estimate of the standard deviation of the model coefficients. For some commonly applied design formulas (such as the Van der Meer formula for rock stability, or the overtopping formula) these values are known, for other formulas they may need to be estimated by studying the original literature in which the formula was first published.

B.6. CALCULATION EXAMPLE

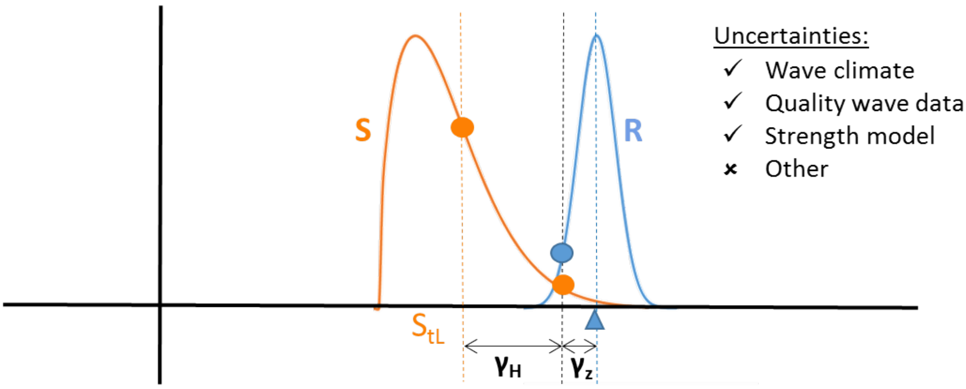
A calculation example will be provided separately, see Blackboard. This example will be included here in future versions of these lecture notes.



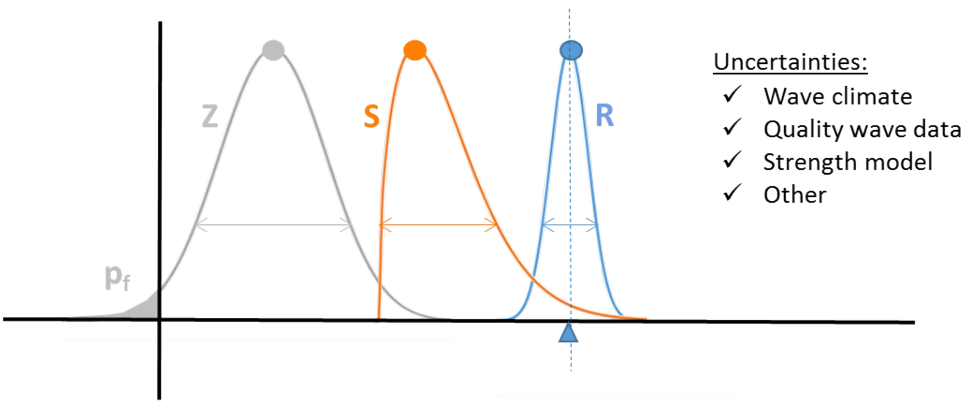
(a) Conventional approach (with mean values of design formula coefficients)



(b) Conventional method (with conservative values of design formula coefficients)



(c) PIANC method of partial safety coefficients (PIANC method)



(d) Level II or Level III full probabilistic methods

Figure B.3: Illustration of various approaches to design reliability

C

QUARRY OPERATIONS

C.1. RECONNAISSANCE

Only few quarries in the world are specialised in producing armourstone. The market for most of the non-dedicated quarries is to produce aggregates for road ballast, concrete industry, but also for the chemical industry. A fine fragmentation is required. It is achieved by special drilling and blasting techniques. Classification is done by sieving. A second large group of quarries produces nice rectangular blocks, so called dimension stone quarries. They produce mainly stone for architectural and furniture use (façade cladding, kitchen blades, etc). Usually these blocks are made by sawing operations or drilling large numbers of small holes. Fragmentation is avoided.

For armourstone large blocks are required. In normal aggregate quarries a small number of large blocks is produced as by-product. Sometimes these blocks are sold as armourstone. In the dimension stone quarries many stones are rejected because of cosmetic reasons, they are simply not beautiful enough. Often these rejected stones can be used very well as armourstone. Some companies are specialising in processing these rejected dimension stones for use in hydraulic engineering.

But for many projects no appropriate quarry can be found in the neighbourhood, and then a dedicated quarry has to be opened specially for the project. So the starting point is a reconnaissance for a suitable quarry. This is rather specialised work for an engineering geologist. This appendix gives only an introduction. For details one is referred to chapter 3 of the Rock Manual [2007].

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

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

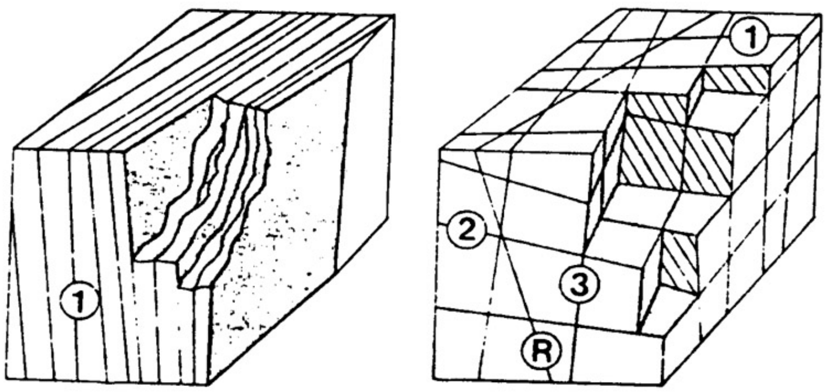


Figure C.1: Types of joints

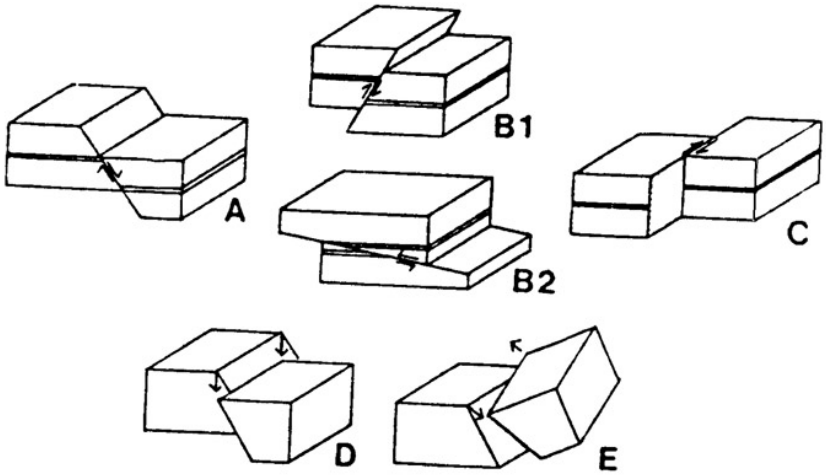


Figure C.2: Types of fault

JOINTS

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

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

FAULT

A fracture or fracture zone along which there has been recognizable displacement from a few centimetres to a few kilometres in scale. The walls are often striated and polished (slickensided) resulting from the shear-displacement. (see figure C.1)

Frequently rock on both sides of a fault is shattered and altered or weathered, resulting in fillings such as breccia and gouge. Fault width may vary from millimetres to hundreds of meters.

DISCONTINUITIES

The general term for any mechanical discontinuity in a rock mass having zero or low tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistocity-planes, weakness zones and faults.

The ten parameters selected to describe discontinuities and rock masses are as follows:

- **Orientation:** Attitude of discontinuity in space (see Figure C.3)
- **Spacing:** Perpendicular distance between adjacent discontinuities (see and Figure C.4).
- **Persistence:** Discontinuity trace length as observed in an exposure.
- **Roughness:** Inherent surface roughness relative to the mean plane of a discontinuity.
- **Wall strength:** Equivalent compression strength of the adjacent rock walls of a discontinuity. Maybe lower than rock block strength due to weathering or alteration of the walls.
- **Aperture:** Perpendicular distance between rock-walls of a discontinuity in the intervening space is air or water filled.
- **Filling:** Material that separates the adjacent rock-walls of a discontinuity and that is usually weaker than the parent-rock.
- **Seepage:** Water-flow and free-moisture visible in individual discontinuities or in the rock mass as a whole.
- **Number of sets:** The number of joint sets comprising the intersecting joint system the rock mass may be divided by individual discontinuities.
- **Block-size:** (see Figure C.5)

Rock-block dimensions result from the mutual orientation of intersecting joint sets and from the spacing of the individual sets. Individual discontinuities may further influence the block and the shape. Block-size

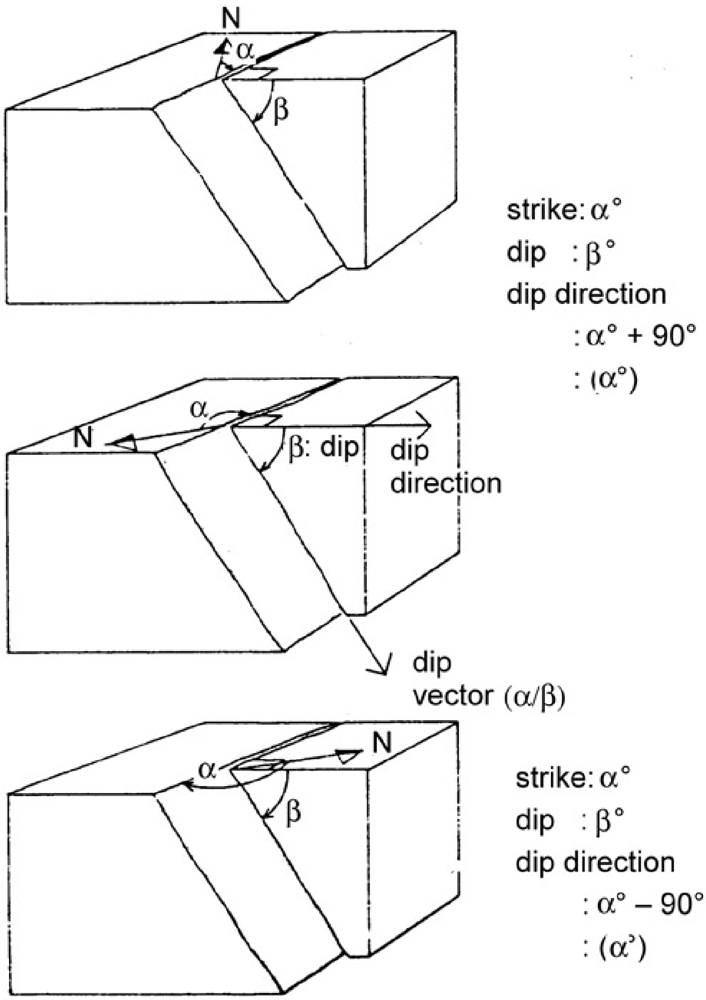


Figure C.3: Attitude of discontinuity

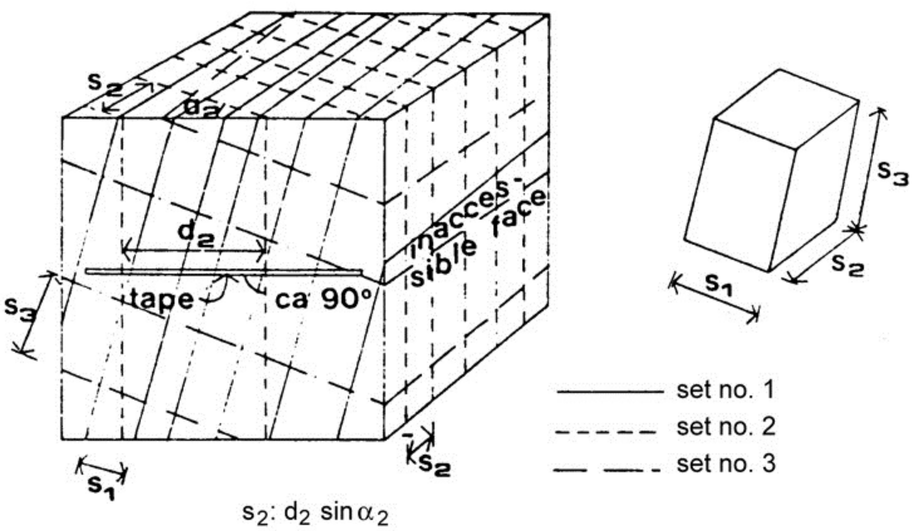


Figure C.4: Distance between adjacent discontinuities

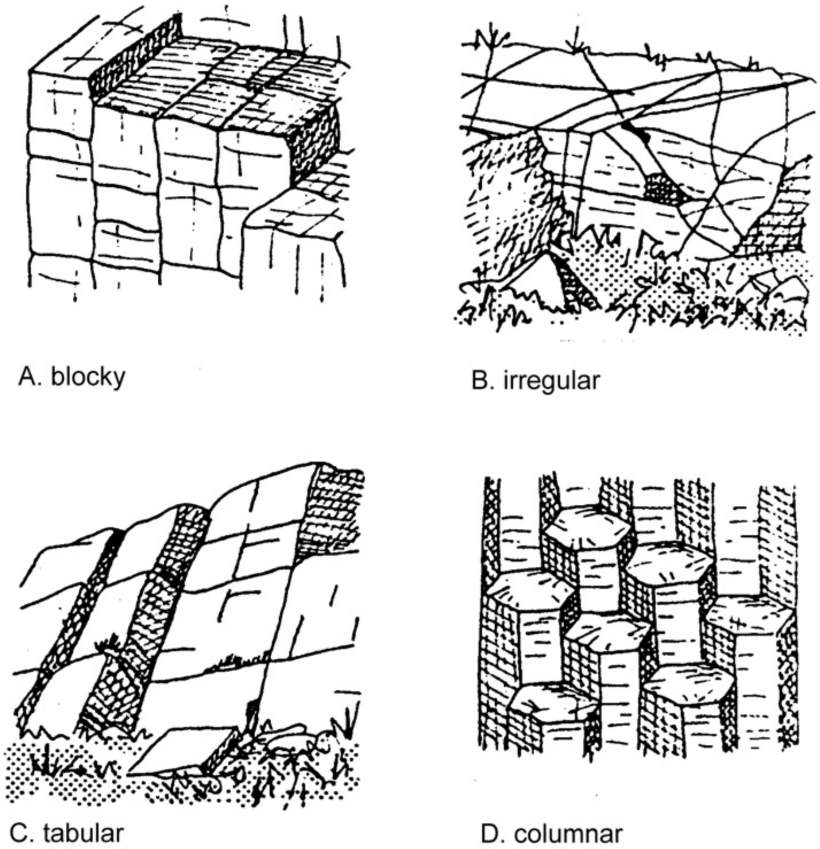


Figure C.5: Block-size

can be described either by means of the average dimension of typical blocks (block-size index l_b) or by the total number of joints intersecting a unit volume of the rock mass (Volumetric Joint Count J_v) (see Table C.1)

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

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

Table C.1: Block size

Values of $J_v > 60$ would represent crushed rock, typical of a clay-free crushed zone. On the basis of this information an experienced geologist is able to provide an expected fragmentation curve. See Figure **??** as example.

Apart from these data, information must be obtained on the density, the mechanical strength, the abrasive resistance and the chemical durability (in sea water!). The European Standard EN13383 gives detailed descriptions of all required tests.

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

- easy accessibility
- volume of the formation must be enough to serve the whole job
- blasting must be possible without excessive damage to human life or the environment in general
- concessions must be made available

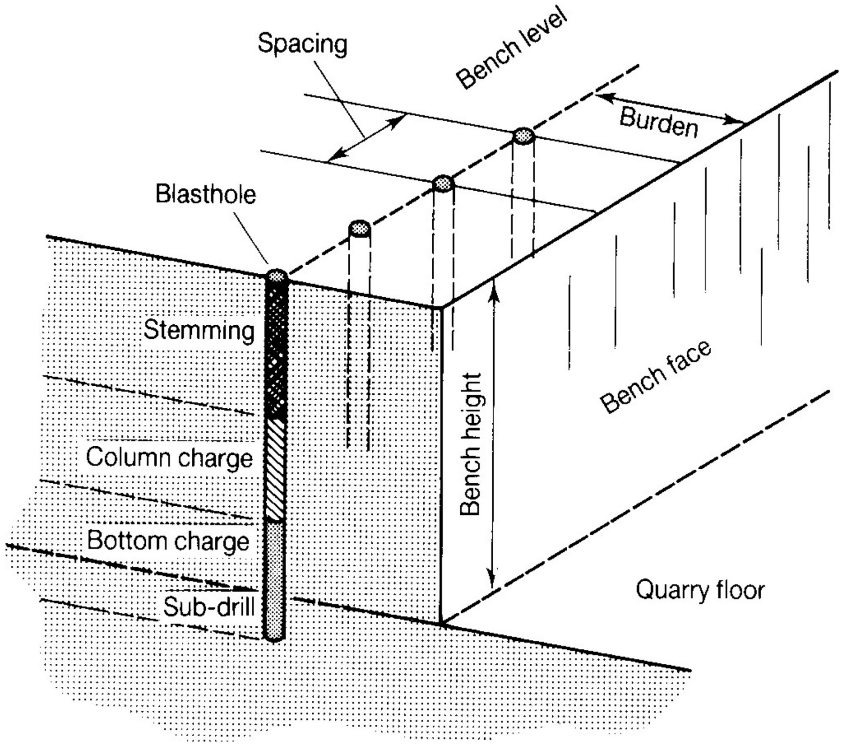


Figure C.6: Blasting lay-out

- in the near vicinity of the quarry sufficient space should be available to open work yards, depots, etc.

C.2. BLASTING

Often the engineer/designer has to rely on contractors or quarry operators regarding information on possible maximum quarry yields or the sizes of the largest stones obtainable from available quarries. These estimates are very often biased by the size of equipment the contractors or quarry operators are using or the actual requirement for stone sizes in previous projects. It seems to be commonly accepted that quarries only yield up to 6 to 8 tonne stones. Dedicated armourstone production is not common and therefore there are not many contractors who have much experience in this field. Guidelines for blasting for armourstones are insufficient and only a few contractors have much experience in drilling and blasting for breakwater construction. This is, however, gradually improving. Contractors are gaining experience in obtaining stone classes to the requested specifications and an increasing number of contractors are now familiar with the quarry yield prediction curves and rely on them in their tenders.

For normal aggregate blasting the bottom charge is only slightly stronger than the column charge (see Figure A2-6). However, for producing large blocks, the column charge is usually replaced by an air deck (i.e. the blast hole is empty). Typical sizes are a bench height of 12 m, a sub-drill of 1 to 2 m, a bottom charge height of 4 m and a stemming of also 4 m. A typical burden size is 4 m and a spacing of also 4 m. The blast hole diameter is in the order of 10 cm.

Furthermore, increased knowledge through quarry yield prediction and in the production of armourstone from various quarries has allowed the specification of large (10-20 tonnes) and extra-large (20-35 tonnes) stones, typically to improve the stability of the berm. The percentage of large stones produced in the quarry can be as low as 2-5% of the total quarried volume to be used as the largest stone class. Large hydraulic excavators and front loaders (75 to 110 tonnes) that can handle these large to extra-large stones have become readily available. These large machines may raise the cost of the projects by a modest 1-2%. Recent projects in Iceland and Norway have utilised large to extra-large stones to the advantage of the stability and strength of the berm structures. A relatively low percentage of these largest stone classes can be of great advantage for the integrity of most breakwaters. This is not only valid for high to moderate wave conditions but also applies to lower wave load conditions where quarries with relatively low yield size distribution are used. For the same design wave condition and stability of the berm, the additional cost of the larger hydraulic excavator is compensated for by smaller berm width. Table C.2 shows the results of a few quarry investigations where large and extra-large blocks have been required, (SMÁRASON 2005). In all cases the actual quarry yield has been pretty close to the prediction.

Project	>0.1t %	>0.5 t %	>1.0 t %	>2.0 t %	>5.0 t %	>10 t %	>20 t %	>50 t %
Sirevåg	75	60	51	42	30	23	17	10
Hammerfest	60	42	33	25	16	11	6	4
Karwar	47	32	25	20	15	8	4	1
Port Elizabeth	58	44	35	27	16	10	4	1
Hornafjördur	74	58	47	34	26	21	15	7
Breidárlón	68	50	43	36	27	20	12	5
Vopnafjördur	73	60	52	44	31	23	17	10
Húsavík	55	40	33	26	20	15	10	3

Table C.2: The predicted optimum yields for quarries for some recent breakwater projects in Iceland, Norway, India and South Africa

C.2.1. PREDICTING THE QUARRY YIELD

A quarry blast will usually yield a very wide range of rock sizes. In some cases it is necessary to predict this distribution (called the quarry *yield curve*) in advance, for instance when the design of the breakwater needs to be optimized by using as much from the quarry blast as possible.

In general terms, the yield curve follows the Rosin-Rammler equation:

$$y = 1 - \exp\left\{-\left(\frac{d}{d_{63}}\right)^{n_{RRD}}\right\} \tag{C.1}$$

where:

y	cumulative weight in % finer than d
d	particle size (block size)
d_{63}	characteristic particle size (63 % smaller than d_{63})
n_{RRD}	index of uniformity

The d_{63} is not a very practical measure to use, so the Rosin-Rammler curve is usually expressed in terms of d_{50}:

$$y = 1 - \exp\left\{-0.693\left(\frac{d}{d_{50}}\right)^{n_{RRD}}\right\} \tag{C.2}$$

or its inverse:

$$d_y = \left\{\frac{-\ln(1-y)}{0.693}\right\}^{1/n_{RRD}} \tag{C.3}$$

Sometimes it is more practical to express the quarry yield in terms of rock mass instead of diameter. The Rosin-Rammler curve then becomes:

$$y = 1 - \exp\left\{-0.693\left(\frac{d}{M_{50}}\right)^{n_{RRM}}\right\} \tag{C.4}$$

or its inverse:

$$M_y = \left\{\frac{-\ln(1-y)}{0.693}\right\}^{1/n_{RRM}} \tag{C.5}$$

Since $M \sim D^3$ is follows that $n_{RRD} = 3 \cdot n_{RRM}$.

The prediction of the quarry yield now comes down to predicting M_{50} and n_{RRM} after the blast. These two parameters are a function of:

- The geology of the *in situ*, i.e. unblasted, rock. The more cracks, discontinuities etc are in the rock, and the closer these are spaced, the smaller the resulting blocks after the blast will be. This is a given natural parameter that cannot be controlled (other then by the choice of quarry location or bench face)

- The blasting procedure itself, such as the spacing of the charges and the amount of explosives used. This can be controlled by the quarry operator.

The Rock Manual (chapter 3) gives an elaborate overview of methods to describe these two parameters. In these lecture notes we will use a strongly simplified method to get a first order indication of the expected quarry yield (taken from The Rock Manual section 3.9.4.4).

We will characterize the geology of the *ins itu* rock by the Principle Mean Spacing (PMS) of the crack pattern, which is a measure for the average distance between the cracks. We will assume that the value of this parameter has been given (e.g. as a result from a quarry visit). Generally speaking, the higher the PMS, the more suitbale the rock is for producing large armour rock.

For the blasting procedure, we will distinguish between two methods. One is the *aggregate blasting* method, which uses a relatively close spacing and large quantity of explosives, leading to an emphasis on smaller rocks. This procedure is usually followed if the quarry is mainly producing small stones or gravel for use as aggregates in asphalt or concrete production. The opposite, *armourstone blasting*, uses smaller quantities of explosives and aims at producing more larger stones (and fewer smaller ones). Aggregate blasting is a common practice in many quarries as their principal business is to deliver material to the concrete industry. Armourstone blasting may sometimes be required specifically for breakwater construction projects. Aggregate blasting leads to a wider range of stone sizes characterized by $n_{RRD} = 0.9$ ($n_{RRD} = 0.30$). Armourstone blasting leads to a smaller range and $n_{RRD} = 0.7$ ($n_{RRM} = 0.23$).

The situ rock PMS can be correlated to a characteristic volume V_{80} of the blocks in the situ rock, where 80% by mass is smaller than V_{80}. The ratio between the V_{80} of the situ rock before the blast and the V_{80} of the blasted rock is then a function of the blasting method (aggregate or armour blasting), according to the following table (ROCK MANUAL table 3.26, WANG ET AL 1991)). In general, for armourstone blasting $V_{80,\text{blasted}} \approx V_{80,\text{situ}}/6$ and for aggregate blasting $V_{80,\text{blasted}} \approx V_{80,\text{situ}}/20$.

PMS	$V_{80,\text{situ}}$	$V_{80,\text{blasted}}$ [m^3]	
		Armourstone blast	Aggregate blast
[m]	[m^3]	$n_{RRM} = 0.23$	$n_{RRM} = 0.30$
0.5	1.1	0.2	0.1
0.6	1.9	0.3	0.1
0.7	3.1	0.5	0.2
0.8	4.6	0.8	0.2
0.9	6.5	1.1	0.3
1.0	9.0	1.5	0.5
1.1	11.9	2.0	0.6
1.2	15.5	2.6	0.8

Table C.3: Simplified prediction of quarry yield parameters

The Rosin-Rammler curve for the predicted quarry yield can then be plotted using equation C.4 or C.5, observing that $M_{80} = \rho_s V_{80}$ and

$$\frac{M_{80}}{M_{50}} = \left(\frac{-\ln(1-0.8)}{-\ln(1-0.5)}\right)^{1/n_{RRM}} = (2.32)^{1/n_{RRM}}$$

C.2.2. EXAMPLE OF DESIGN OPTIMIZATION FOR QUARRY YIELD

In the following, we will work out an example showing how a design can be optimized against the predicted quarry yield. Suppose we have made an initial design for an Icelandic-type breakwater as shown in Figure C.7. The design uses four different rock classes and for each class the volumes per running metre are known, see Table C.4.

We can then draw a *demand curve* for our design as shown in Figure C.8(a), showing that 100% of the rock material that we need is heavier than 0.3 T, (100-51 =) 49% of the material that we need is heavier then 1 T and so on. Note that the curve is drawn 'upside down' from the normal convention, i.e. the y-axis shows the percentage *heavier* instead of the percentge *passing*. This makes these plots somewhat easier to interpret.

Suppose further that we have a quarry that can be characterized by $PMS = 0.8$ and $\rho_s = 2.7$ t/m^3. If we use an armourstone blasting method we predict an as-blasted $n_{RRM} = 0.23$ and $V_80 = 0.8$ m^3, so $M_{80} = \rho_s V_{80} = 2.1$

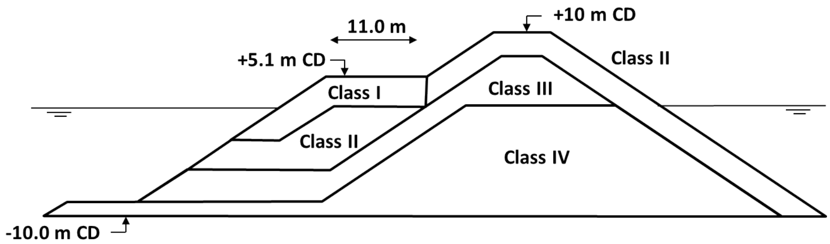


Figure C.7: Example optimization: initial design

		m^3/m^1	% of total
Class I	8 - 12 T	62.5	7
Class II	4 - 8 T	214.0	24
Class III	1 - 4 T	167.8	19
Class IV	0.3 - 1 T	453.4	51
Total		897.7	

Table C.4: Rock volumes initial design

T and $M_{50} = M_{80}/(2.32)^{1/0.23} = 0.054$ T (54 kg). The corresponding Rosin-Rammler curve is drawn in Figure C.8(b) (only for the part M > 0.01 T).

From the predicted quarry yield plot we can see that the percentage of very large stones (> 10 T) in our blast is in the order of 10%, which roughly corresponds to the demand of our initial design. So the quarry will be able to deliver the stones that we need for our design. The downside, however, becomes obvious when we realize that our design only uses stones > 0.3 T, and that according to the yield curve only approximately 35% is heavier than this. This means that the remaining 100-35 = 65% *undersize* of the quarry blast is not used and is thus wasted. Clearly there is some room for optimization here. Another potential optimization becomes clear when we superimpose our demand curve (now scaled for the 65% undersize, so the 0.3 T point lies at y = 35%). Not just the rocks smaller than 0.3 T are wasted, but also all the volume in between the lines. We can optimize our design if we use some more heavy rocks (and also heavier than the 12 T that we had used so far) and fewer rocks in the intermediate range.

With this information we can attempt a re-design, optimizing the rock demand. The optimized design is given in Figure C.9, and the corresponding volumes in Table C.5. With respect to the initial design, we have made the following changes:

- We have changed the Class I armour type from 8-12 T to 10-20 T. As a consequence the berm can also be a little bit shorter (see chapter on berm breakwater design).
- We have lowered the percentage of rock in the intermediate range by replacing some of the Class II rocks on the rear side by smaller classes.
- We have introduced a Quarry Run, using the material below 0.3 T. We only have small filter layer in 0.3-1 T left. For the lower boundary of the quarry run we have now set an arbitray value of 0.01 T (which is approximately 200 mm). The consequences of modifying this lower limit can be explored in another round of optimizations, not reported here.

The quarry yield curve of course does not change as a result of our re-design. We are now using rocks larger than 0.01 T, for which 60% of our blast was heavier, so our percentage undersize has gone down to 40%. The effect on the resulting demand curve is shown in Figure C.8(d). Clearly we are using the quarry yield to a much closer degree, resulting in a more economical design. This becomes particularly clear if we draw the entire quarry yield curve (not just the part > 0.01 T) and indicate the losses by shading, see Figure C.10. Note that we have used a mixed x-axis on this figure, showing diameters (in mm) for the smaller ranges and mass (in T) for the larger ranges.

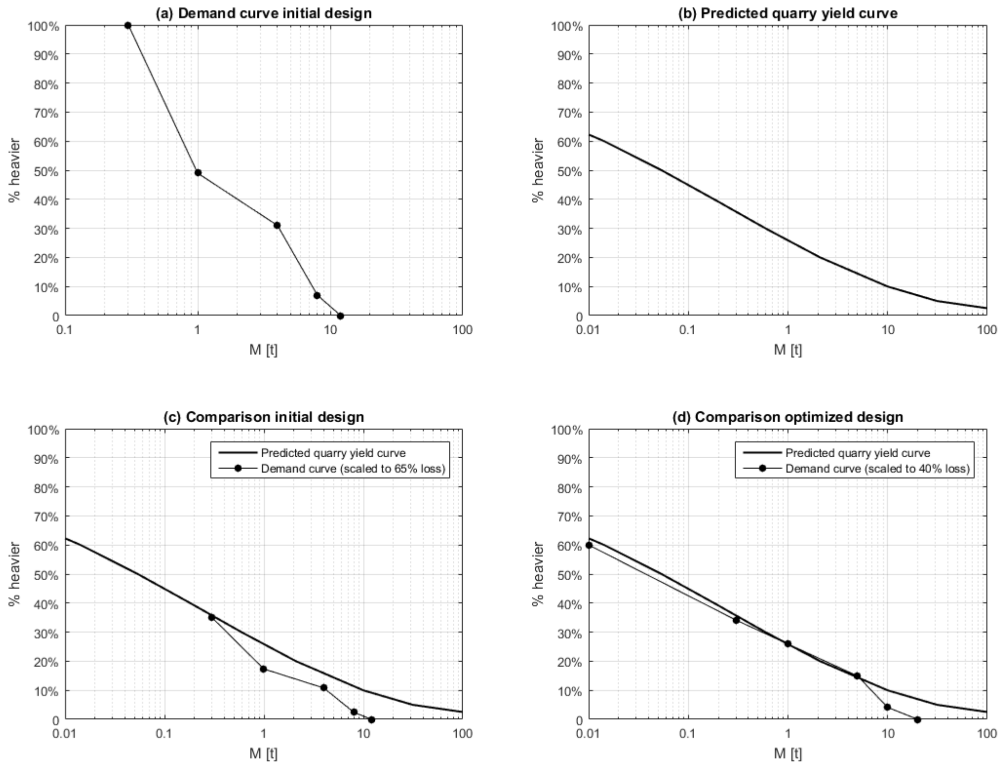


Figure C.8: Example optimization

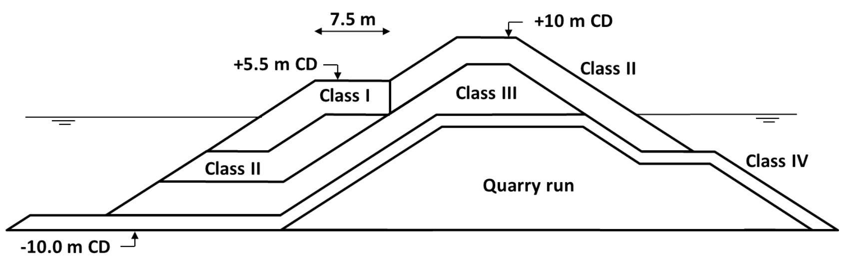


Figure C.9: Example optimization: optimized design

		m^3/m^1	% of total
Class I	10 - 20 T	59.4	7
Class II	4 - 10 T	154.9	18
Class III	1 - 4 T	158.9	18
Class IV	0.3 - 1 T	119.2	14
Quarry run	200-500 mm	371.9	43
Total		864.3	

Table C.5: Rock volumes optimized design

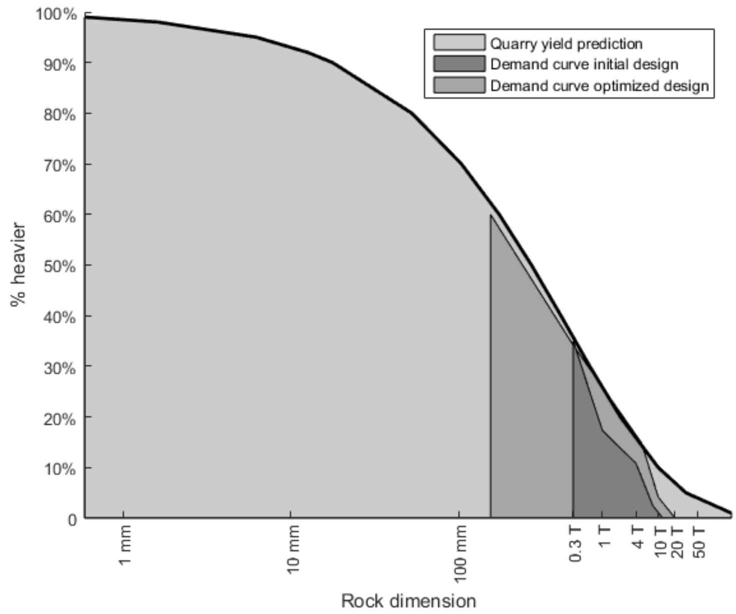


Figure C.10: Example effect of optimization

C.3. OPERATION OF THE QUARRY

The mining operation itself should be done in a systematic way, following a pre-designed mining plan. During the blasting, bench floors are created. The sequence of blasting depends on the overall pit slope (soil mechanical stability!). The width on each bench floor should be sufficient to create working space for classification, loading and transport (Figure C.11 and Figure C.12).

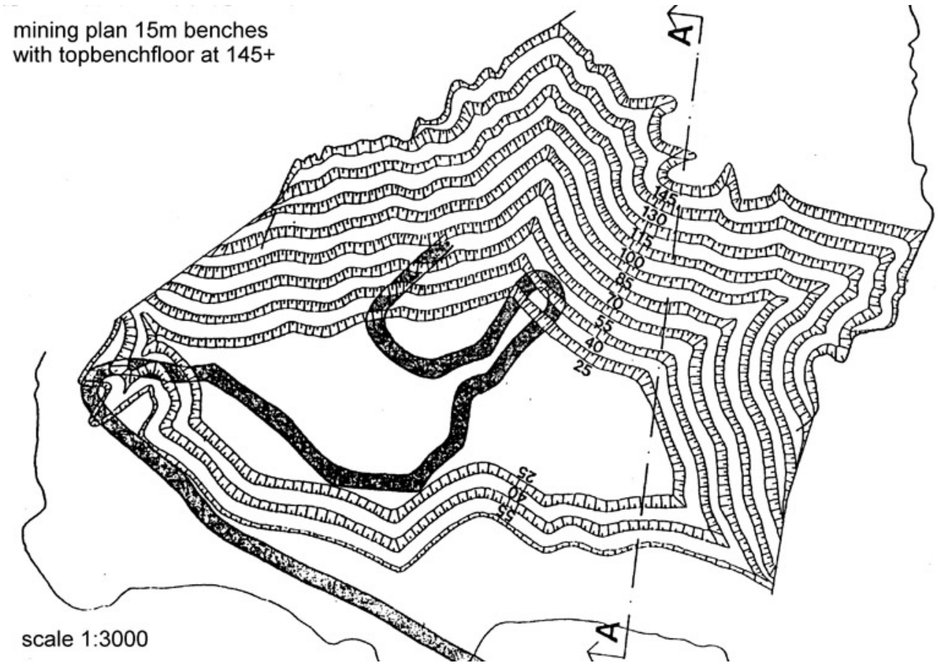


Figure C.11: Mining plan

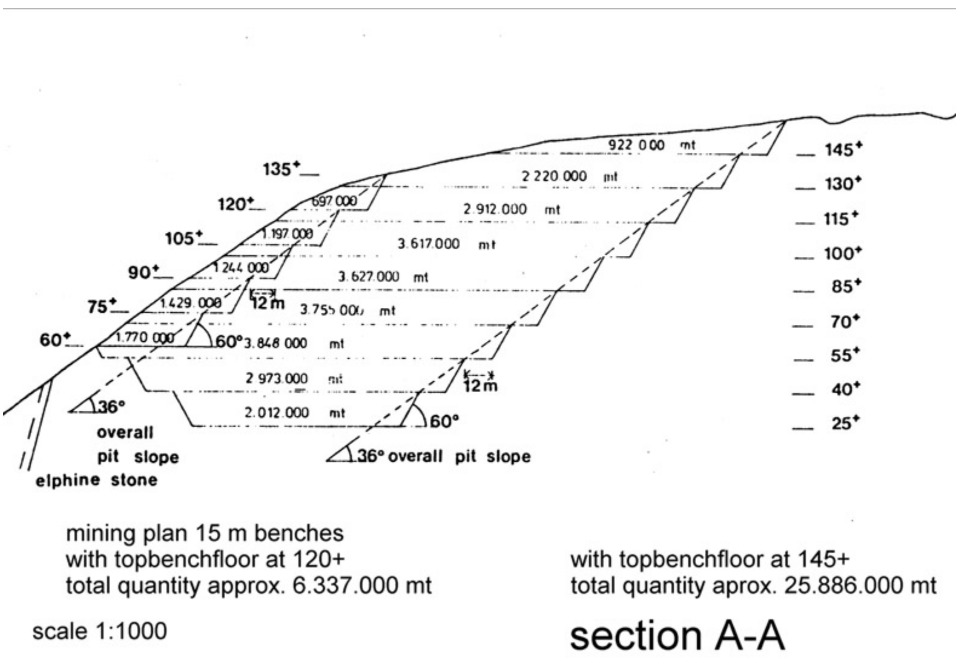


Figure C.12: Mining plan, cross section

D

CONCRETE ARMOUR UNITS

D.1. SHAPE

Compared with quarry stone, concrete armour units have the advantage that shape; the designer or contractor can choose size and density at liberty. As indicated in Chapter 7, a large number of different shapes has been developed by consultants, researchers and contractors. A rather complete overview is given by PIANC MarCom 36 [2005]; the online version of this catalogue is regularly updated.

The simplest shapes are the Cube and the Parallelepiped. The advantage of these shapes is that moulds are simple and that handling and storage is also quite simple. Some modifications of the shape are used to improve the behaviour of the blocks under wave attack. Amongst others, one may discern sleeves in the side plains, ears at the corners or holes in the centre. It has been common practice to use special shapes as well, providing interlocking properties that lead to an enhanced stability. Most units in this category are slender units like Tetrapod and Dolos. The slender shape creates a much better interlocking than present in quarry stone and the simple blocks. Also the porosity of these units is slightly larger than the porosity of the more traditional shapes, which probably adds to the stability as well. This has resulted in (Hudson) stability numbers K_D that are ranging from 8 upwards. (see Chapter 7). Drawings of these units are given in Figure D.1 to Figure D.5, along with the relation between size and volume. Because of the slender shapes, the characteristic dimensions may be much larger than the nominal diameter d_n, defined as the cubic root of the volume.

$$d_n = \sqrt[3]{\frac{M}{\rho_s}} \tag{D.1}$$

The latest developments turn away from the very slender units because of structural problems encountered, and discussed in Chapter 7. These latest units are Accropode-II, Core-loc, and Xbloc, developed respectively by Sogreah (France), the Waterways Experiment Station (USA) and by Delta Marine Consultants (Netherlands).

Dimensions of these blocks are also given in the Figures. The Core-loc has originally been developed as a repair unit for damaged Dolos breakwaters, but it is also applied for new constructions.

The original Accropode was the first element that could be placed in a single layer, and is therefore very economic. The unit proved to be very successful. However, because of the two flat side of the block, careful placing is required. And therefore placing costs are quite high. Both Sogreah as well as Delta Marine Consultants were looking for a unit which did not have that disadvantage. More or less simultaneously both the Accropode II and the Xbloc were presented. Because these elements do not have a flat side, placing is easier and therefore placing goes much faster.

D.2. SIZE

Although the size of the concrete armour units seems unlimited, there is a limitation in practice. Since the units are in principle not reinforced, to avoid corrosion problems, the structural integrity depends largely on the tensile strength of the concrete. Increasing the linear size of the units leads to an increase of mass and forces proportional to the third power of the size. ($V \propto D^3$).

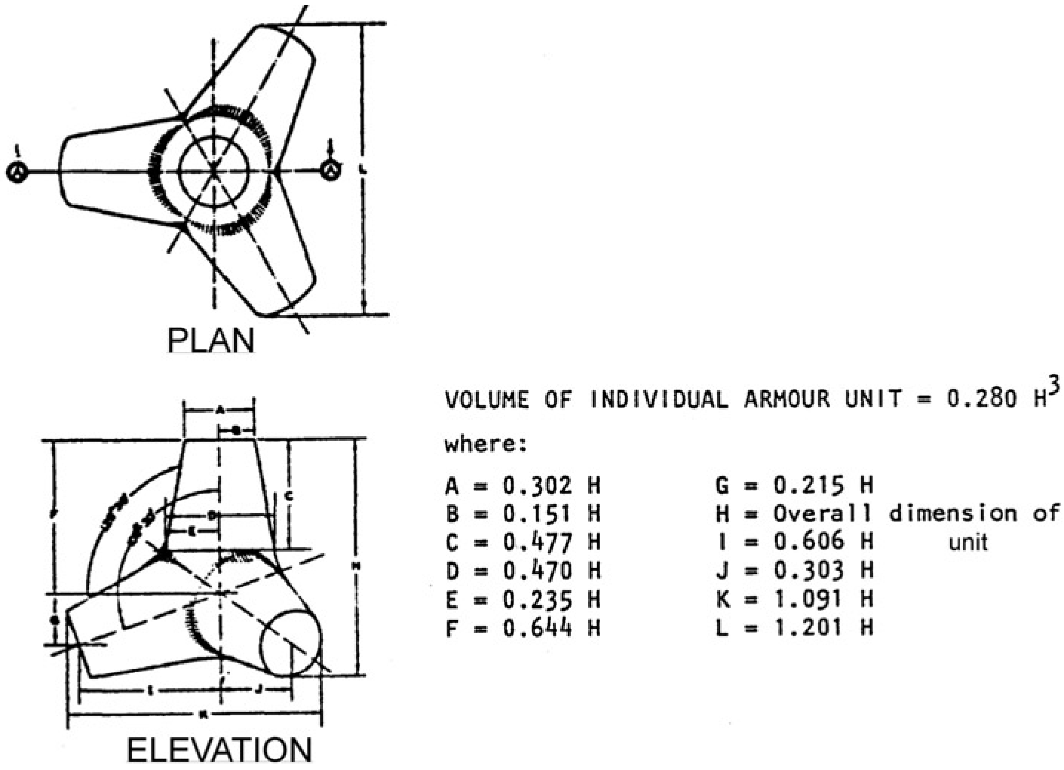


Figure D.1: Tetrapod

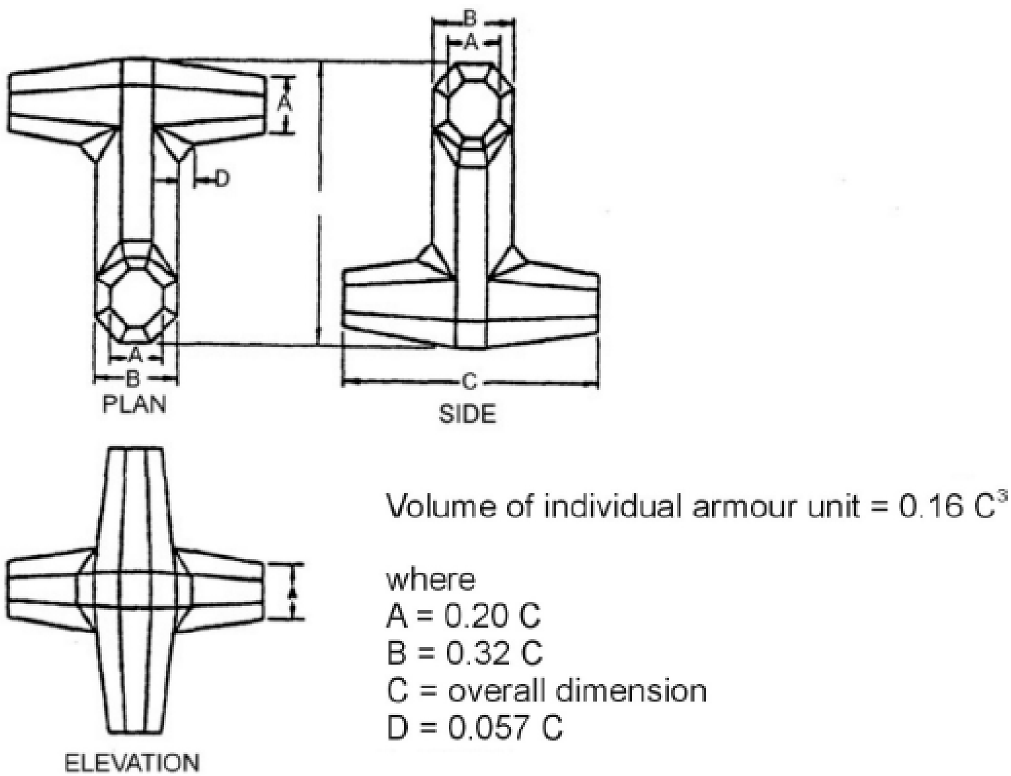


Figure D.2: Dolos

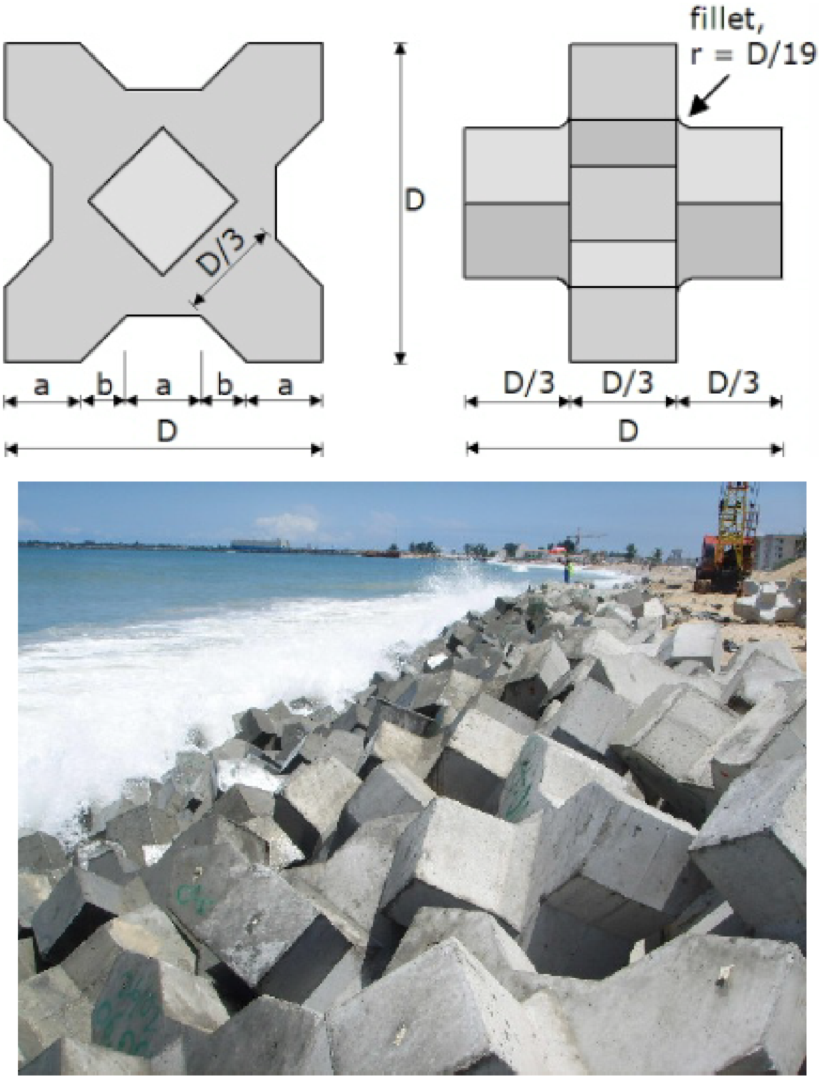


Figure D.3: Xbloc

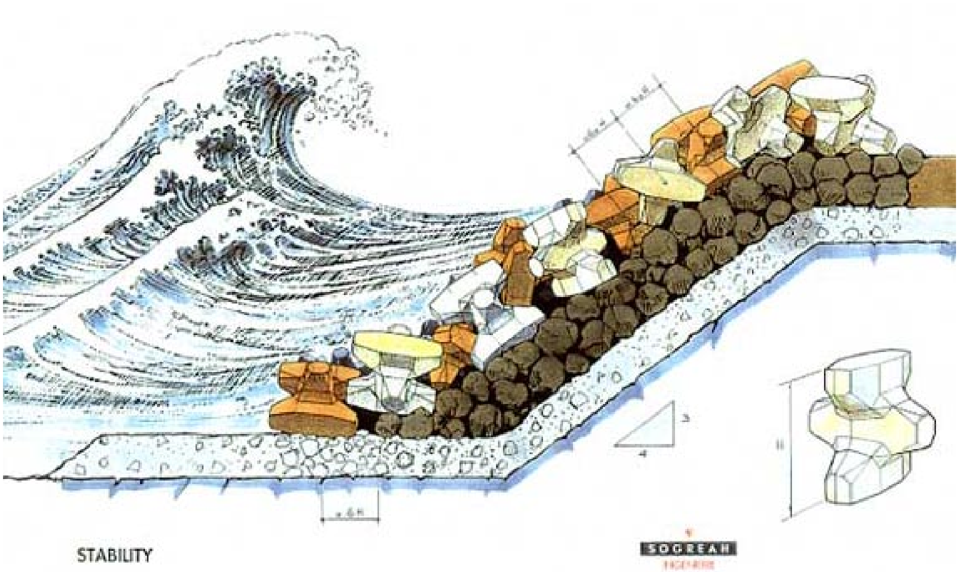


Figure D.4: Accropode

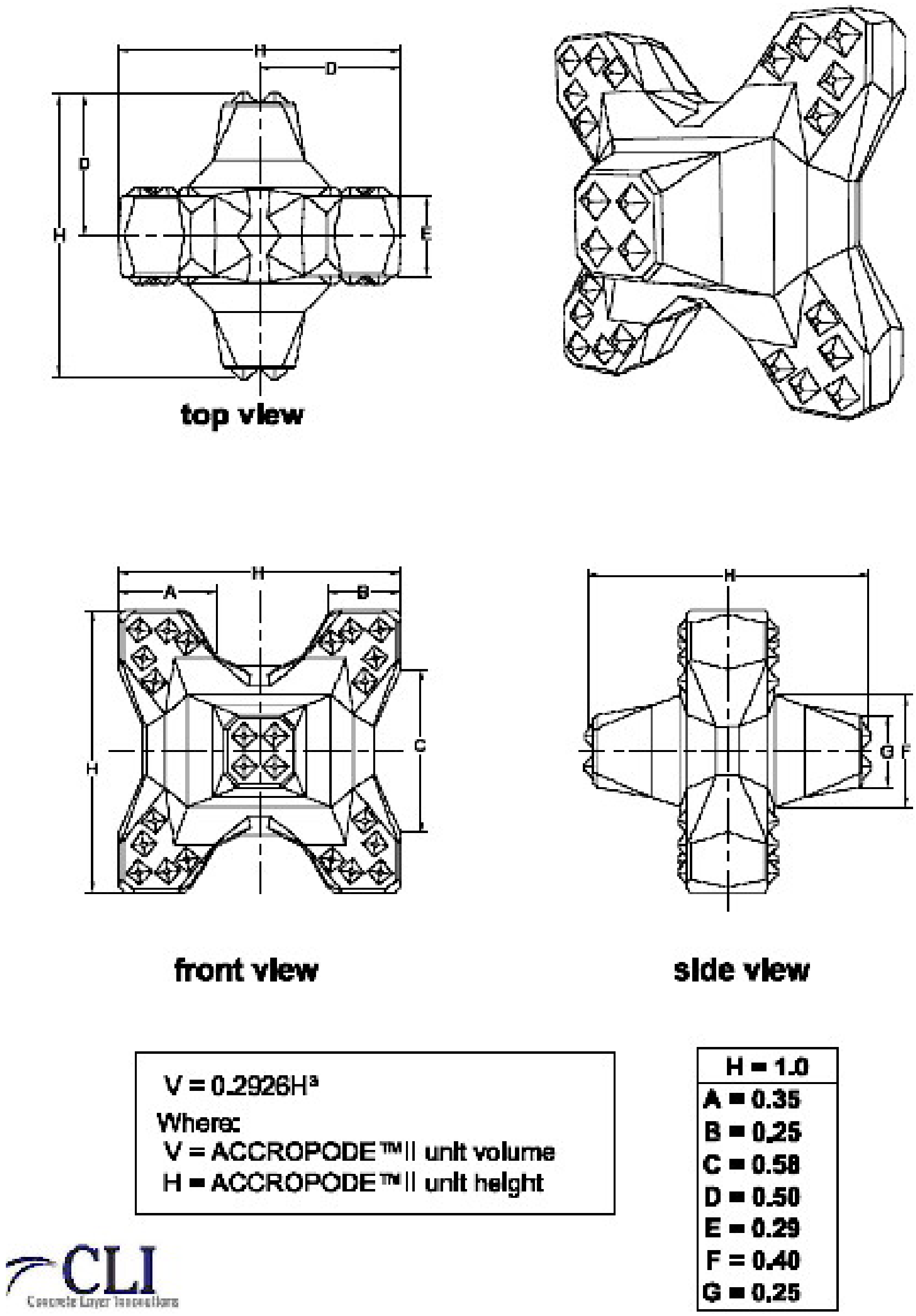


Figure D.5: Accropode II

Figure D.6: Xbloc formwork

The cross-sectional area that provides the structural strength increases, however with the square of the size only. This means that the tension in any cross section increases basically linear with the dimension of the unit. Since the strength is constant, a larger block becomes more and more vulnerable to structural damage when the actual tension exceeds the available tensile strength. This is a failure mechanism that was certainly overlooked in the (small-scale) model investigations aimed at the hydraulic stability of the units.

D.3. DENSITY

The non-reinforced concrete as used in armour units will have a density ρ_r of 2200 to 2400 kg/m^3 if no special measures are taken. Care shall be taken to achieve a high density so as to avoid penetration of chloride and chemical damage. Since the volume of the units is rather large, proper attention shall be paid to the granulometry of the aggregates forming the skeleton (sand, gravel). Higher densities can be achieved by selecting heavier aggregates like basalt, slacks or ore (Magnadense). In that case, the durability of the end product shall be ascertained by an adequate test programme. In this respect, the use of sulphate resisting cement is always recommended.

D.4. FABRICATION

For the fabrication of armour units, a special concrete mixing plant is almost a requirement to achieve a good and constant quality.

For economic reasons, the contractor wants a mixture that enables him to achieve a quick turnaround time for the moulds. This reduces the number of moulds required. Since the moulds of the special shaped blocks are costly, there is a pressure to use a quick hardening mixture, so that the mould can be removed within say 24 hours after casting.

The shape of these concrete units requires rather complicated formwork. Attention has to be paid to forms which allow a fast turnaround time of the process. Details on formwork design can be provided by the licensees of the units (CLI and DMC).

Because of the large mass of the units, attention is required for the heat generated during the hardening process. This is concentrated in the heart of the units. Specifically when the mould is removed, the surface of the units can cool rapidly, which leads to large tensions in the fresh concrete. In many cases these temperature gradients initiate cracks. It can easily be demonstrated that such cracks have a large influence on the eventual strength of the unit, and thus on the breakage of units during handling or during exposure to high (wave) loads. The problem of temperature gradients plays a very dominant role in places with a strong wind and a low humidity (cooling of the surface) and in regions with a large range between daily maximum and minimum temperatures. The latter occurs in tropical areas and in areas with a desert climate.

This problem can be tackled by the following measures:

- Reduction of the cement content

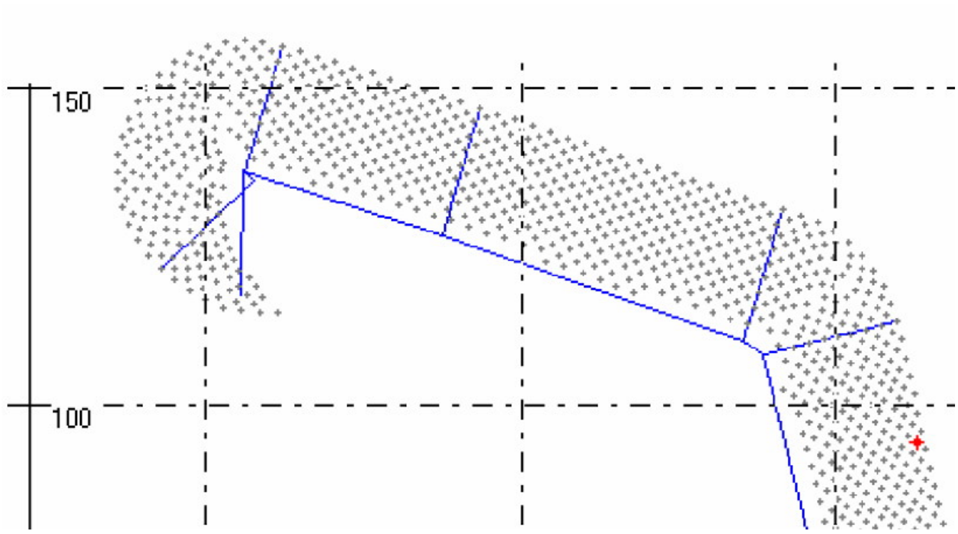


Figure D.7: Xbloc placing grid for Port Oriel (TEN OEVER [2006])

- Use of low heat cement
- Use of slower hardening cement
- Insulation of the units after removal of the moulds
- Spraying curing compound after removal of the moulds.

D.5. PLACEMENT

Concrete units are lifted using slings or clamps. The use of steel hooks is not recommended because they will initiate spalling due to corrosion.

Most concrete blocks are placed at random. Special placement is difficult when working in deeper water and under exposed conditions where no diver assistance is available on a regular basis. Another disadvantage of special placing is the difficulty of repairing damage.

Single layer units require a good support for the toe. In case there is some movement at the toe, the layer will settle and will become quite open around the water line. This might initiate damage. Toe stability can be guaranteed by digging in the toe in the original bottom (perfect, but costly), by placing a support toe in front of the breakwater (not always possible because of the limited water depth) or by a special construction. For example at the revetment in Scarborough in the UK a support has been made by drilling holes in the bedrock in which concrete piles as support were placed. As an alternative, DMC has developed a toe block which can be used as toe for Xbloc slopes.

Special computer programs have been developed to calculate the most optimal placing grid for the concrete units (TEN OEVER, 2006). Especially for roundheads placing is complicated. At roundheads it is not possible to continue with the systematic placing with identical x-distances and y-distances at all levels. Open areas will appear. Software is able to optimise the placement (in fact minimising the gaps), but still this is a problem. Because the waterline is the most endangered zone, it is wise to try to design the placing grid in such a way that the most open areas are not at the water line.

Placement should also be done with care, i.e. without damaging he structures. Slender units have the risk of breakage during placement and during operation. The more sturdy single layer units are less sensitive to breakage, but also in this case one should handle them with care. However, a small amount of damage does not directly cause failure of the breakwater. Broken elements have a somewhat more ductile behaviour than non-broken elements, which means that the breakwater will have damage, but will not completely collapse. For details is referred to the work of DE ROVER [2007].

E

GODA'S PRINCIPLES FOR BREAKWATER DESIGN

From: the 1992 short course for the ICCE '92: "Design and Reliability of Coastal Structures", published by Instituto di Idraulica, Universita di Bologna, Italy.

THE DESIGN OF UPRIGHT BREAKWATERS

Yoshimi Goda
Department of Civil Engineering Yokohama National University, Japan

ABSTRACT
The historical development of upright breakwaters in Japan is briefly reviewed as an introduction. Various wave pressure formulas for vertical walls are discussed, and then the design formulas currently employed in Japan are presented with an example of calculation. Several design factors are also discussed.

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E.1. INTRODUCTION

An upright breakwater is defined here as a structure having an upright section rested upon a foundation It is often called a vertical breakwater or composite breakwater. The former is sometimes referred to a structure directly built on the rock foundation without layers of rubble stones. The latter on the other hand means a breakwater functioning as a sloping-type structure when the tide level is low but as a vertical-wall structure when the tide level is high. Because the terminology may vary from person to person, the definition above is given here in order to avoid further confusion.

Upright breakwaters are of quite old structural type. Old ports in the Roman Empire or ports in even older periods had been provided with breakwaters with upright structures. The upright breakwaters of recent construction have the origin in the 19th century. Italian ports have many upright breakwaters as discussed in the following lecture by Dr. L. Franco. British ports also have a tradition of upright breakwater construction as exemplified in Dover Port. The British tradition can be observed in old breakwaters of Indian ports such as Karachi, Bombay, and Madras. Japanese ports owes this tradition of upright breakwaters to British ports, because the modern breakwater construction began at Yokohama Port in 1890 under supervision of British army engineer, retired Major General H. S. Palmer. Since then Japan has built a large number of upright breakwaters along her long coastline extending over 34,000 km. The total length of upright breakwaters in Japan would exceed several hundred kilometers, as the total extension of breakwaters is more than 1,000 km.

The present note is intended to introduce the engineering practice of upright breakwater design to coastal and harbor engineers in the world, based on the experience of Japanese engineers.

E.2. HISTORICAL DEVELOPMENT OF UPRIGHT BREAKWATERS IN JAPAN

E.2.1. EXAMPLES OF UPRIGHT BREAKWATERS IN MODERN HISTORY OF JAPANESE PORTS

Figure 1 illustrates typical cross sections of upright breakwaters in Japan in time sequences, which is taken from Goda [1985]. The east breakwater of Yokohama Port in Fig. 1 (a) utilized the local material of soft clayey stones for rubble foundation and minimized the use of concrete blocks in the upright section. The stone-filled middle section was replaced by concrete blocks during reconstruction after the storm damage in 1902. The wave condition in Yokohama was not severe with the design height of 3 m.

The structural type of upright breakwaters was adopted at a more exposed location of Otaru Port as shown in Fig. 1 (b) by 1. Hiroi in 1897, who was the chief engineer of regional government, later became a professor of the Tokyo Imperial University, and established the framework of Japanese harbor engineering. The first reinforced concrete caisson breakwater in Japan was built at Kobe in 1911, based on the successful construction of caisson-type quaywall at Rotterdam in 1905. Then Hiroi, immediately seeing the bright future of caisson breakwaters, employed the concept to an island breakwater of Otaru Port shown in Fig. 1 (c), where the design wave was 6 m high. He carried out various field measurements, including wave pressures on a vertical wall, for his finalization of breakwater design. Through these efforts, he came to propose the wave pressure formula for breakwater design, which is to be discussed in the next section.

Hiroi's breakwater caissons were filled with concrete for durability and stability. The work time for concrete placement was sometimes saved by the use of precast blocks as in the example of Onahama Port in Fig. 1 (d). Concrete filling of breakwater caisson had been a tradition before the end of World War II, but a pioneering construction of reinforced concrete caisson breakwater with sand filling was carried out in Yokohama Port during the period of 1928 to 1943: Fig. 1 (e) shows its cross section. After World War II the use of sand as the filler material of caisson cells gradually became a common practice in Japan.

The breakwater of Wakayama Port shown in Fig. 1 (f) was built upon a quite soft ground so that it was provided with a wide foundation for the purpose of counter-balancing the weight of upright section. The breakwater of Ofunato Port in Fig. 1 (g) was built to reduce the inflow of tsunami waves into the bay. The water depth of 35 m below the datum level was the deepest one at the time of construction in 1962, but the present record of the deepest breakwater in Japan is held at Kamaishi Port with the depth of 60 m. Some design features and wave pressures on this breakwater have been discussed by Tanimoto and Goda [199lb]. One of the widest breakwaters is that of Hosojima Port shown in Fig. 1 (h): the widest at present is found at Hedono Port in a remote island with the width 38m (see Tanimoto and Goda l991a). The breakwater of Onahama Port shown in Fig. 1 (i) is of recent design using Goda's wave pressure formulas to be discussed later.

E.2.2. SOME FEATURES OF JAPANESE UPRIGHT BREAKWATERS

As seen in these examples, Japanese breakwaters of upright type have a few common features. One is the relatively low crest elevation above the high water level. Presently, the recommendation for ordinary breakwaters is the crest height of 0.6 $H_{1/3}$ above the high water level for the design condition.

For the design storm condition, this elevation is certainly insufficient to prevent wave agitations by the

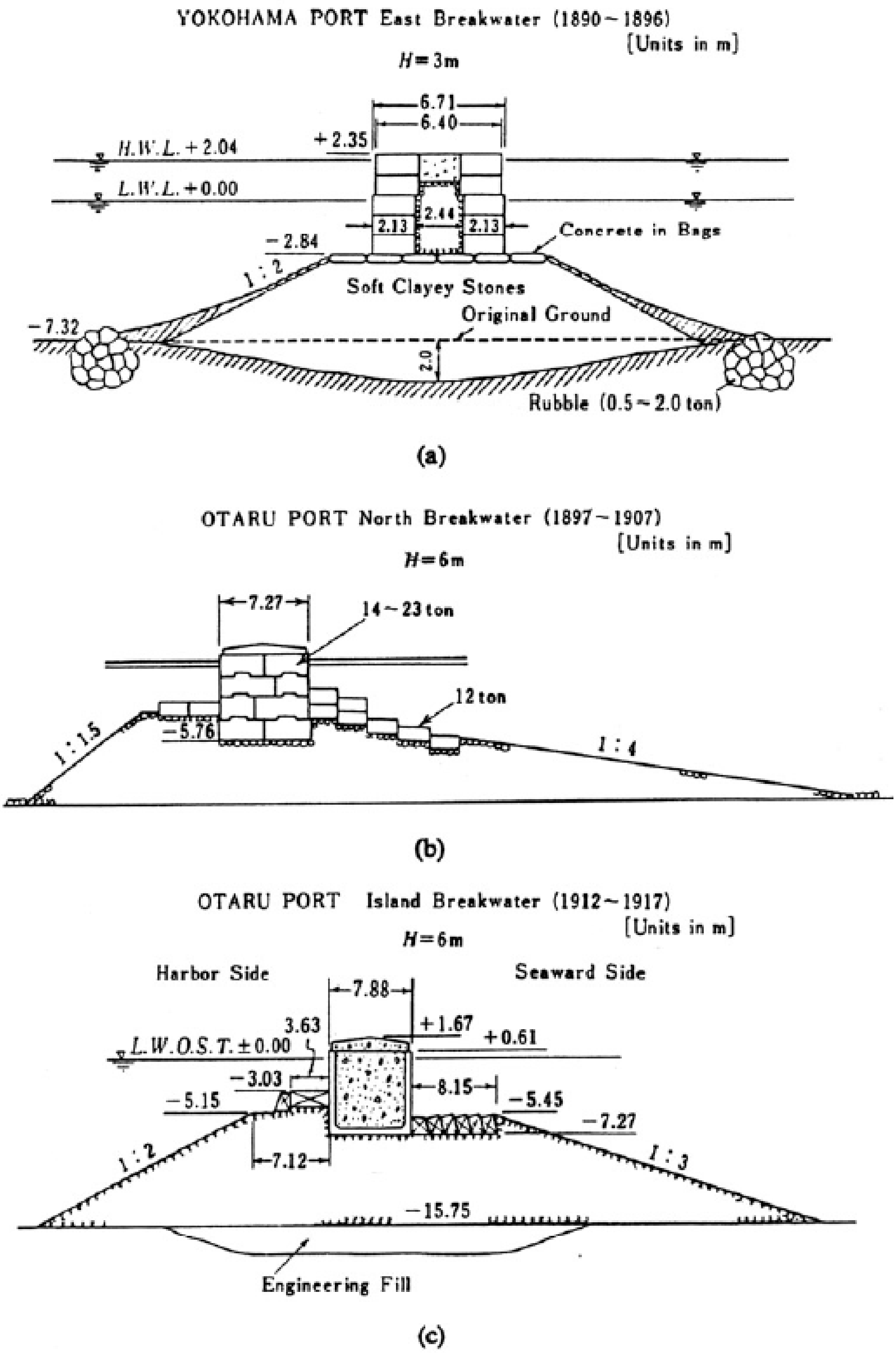

Fig. 1 (a-c) Historical development of upright breakwater in Japan after Goda [1985].

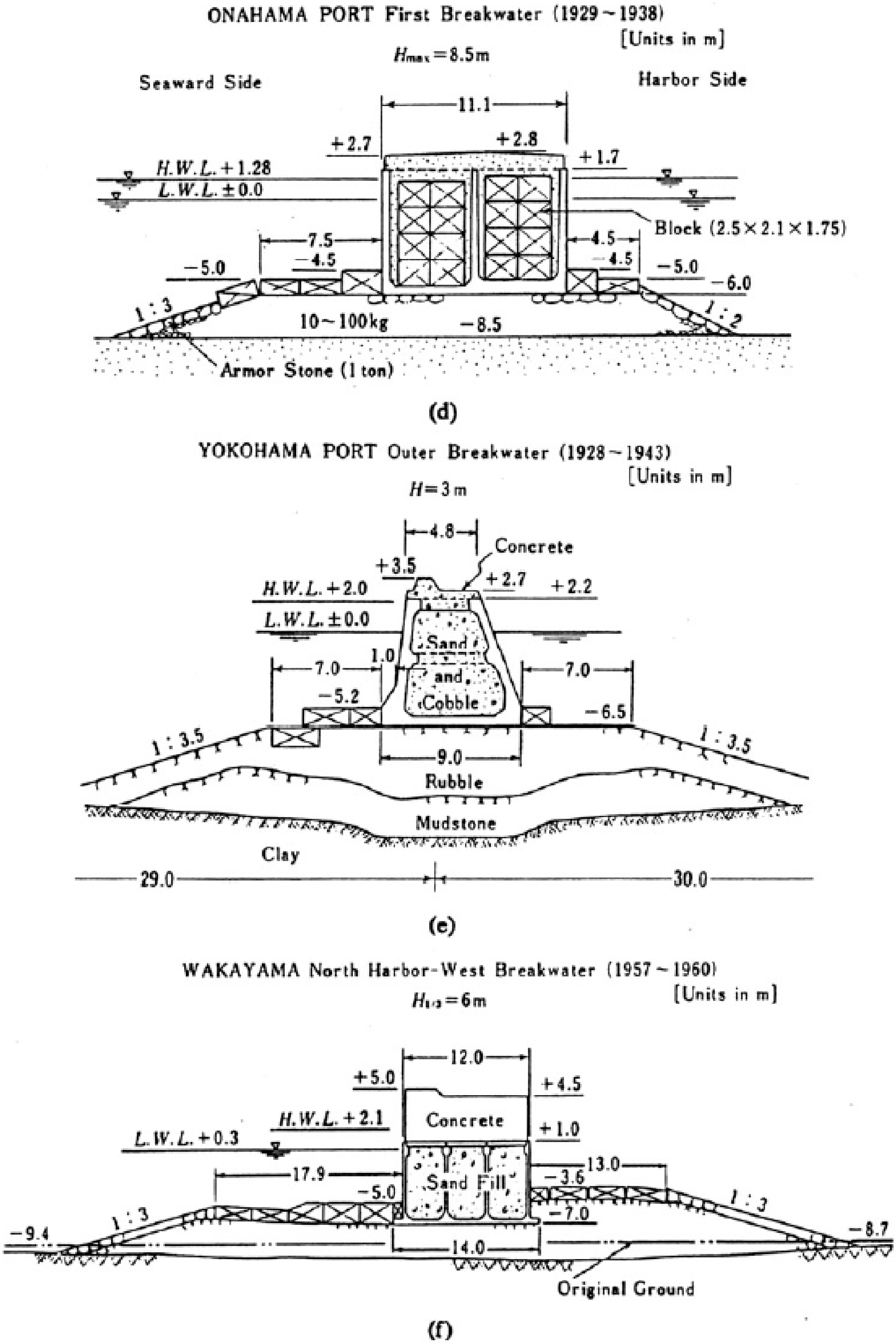

Fig. 1 (d-f) Historical development of upright breakwater in Japan (continued) after Goda [1985].

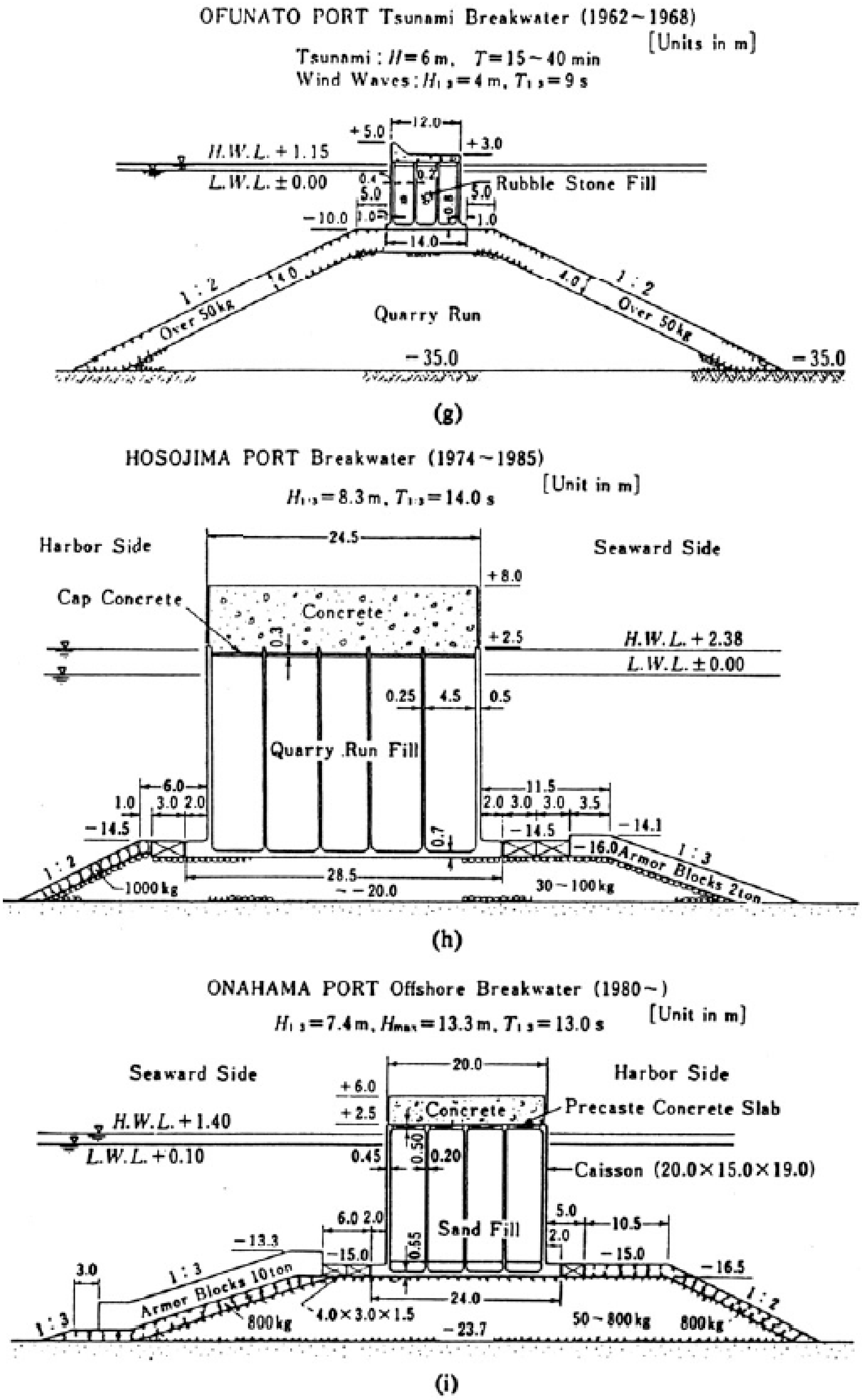

Fig. 1 (g-i) Historical development of upright breakwater in Japan (continued) after Goda [1985].

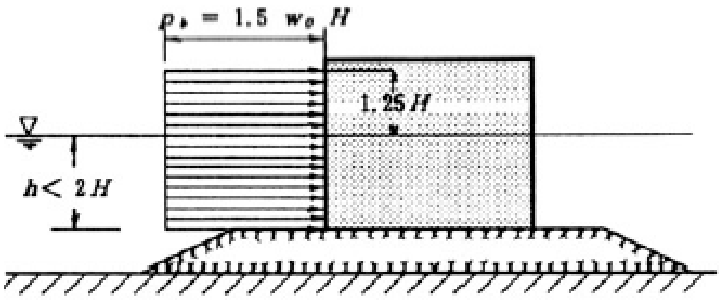

Fig. 2 Wave pressure distribution by Hiroi's formula.

overtopped waves. However, it is a way of thinking of harbor engineers in Japan that the design waves are accompanied by strong gale and storm winds in any case and safe mooring of large vessels within a limited area of harbor basin cannot be guaranteed even if wave agitations are reduced minimum. As the storm waves with the return period of one year or less are much lower than the design wave, the above crest elevation is thought to be sufficient for maintaining a harbor basin calm at the ordinary stormy conditions.

Another feature of Japanese upright breakwaters is a relatively wide berm of rubble foundation and provision of two to three rows of large foot (toe) protection blocks. There is no fixed rule for selection of the berm width and engineers always consult with the examples of existing breakwaters in the neighborhood or those at the location of similar wave conditions. It is somewhat proportional to the size of concrete caisson itself, but the final decision must await good judgment of the engineer in charge. The foot protection concrete blocks have the size ranging from 2 to 4 m in one direction and the height of 1.5 to 2 m, weighing 15 to 50 tf. Though these blocks used to be solid ones, recent blocks are provided with several vertical holes to reduce the uplift force and thus to increase the stability against wave action.

A new development in upright breakwaters of Japan is the employment of various modifications to the shape of concrete caissons, such as perforated walls, vertical slits, curved slits with circular arc members, dual cylindrical walls and others (see Tanimoto and Goda 1991a). These new caisson shapes have been developed to actively dissipate wave energy and thus to reduce wave reflection and wave pressures. A number of these breakwaters have been built and functioning as expected.

E.3. REVIEW OF WAVE PRESSURE FORMULAE FOR VERTICAL WALL

E.3.1. HIROI'S FORMULA

Prof. Hiroi published the wave pressure formula for breakwater design in 1919. It is a quite simple formula with the uniform pressure distribution of the following intensity:

$$p = 1.5 w_0 H \tag{E.1}$$

where w_0 denotes the specific weight of sea water and H the incident wave height. This pressure distribution extends to the elevation of $1.25H$ above the design water level or the crest of breakwater if the latter is lower, as shown in Fig. 2.

Prof. Hiroi explained the phenomenon of wave pressure exerted upon a vertical wall as the momentum force of impinging jet flow of breaking waves and gave the reasoning for its quantitative evaluation. However, he must have had some good judgment on the magnitude of wave pressure from his long experience of harbor construction and several efforts of pressure measurements in situ. He states that he obtained the records of wave pressure exceeding 50 $\mathrm{tf/m^2}$ by the pressure gauges set at a concrete wall in water of several meters deep. Nevertheless, he did not incorporate such high pressures into the formula of breakwater design, by saying that the high wave pressure must have lasted for only a short duration and are ineffective to cause appreciable damage to breakwaters.

Hiroi's wave pressure formula was intended for use in relatively shallow water where breaking waves are the governing factor. He also recommended to assume the wave height being 90% of water depth if no reliable information is available on the design wave condition. Hiroi's wave pressure formula was soon accepted by harbor engineers in Japan, and almost all breakwaters in Japan had been designed by this formula till the mid 1980s.

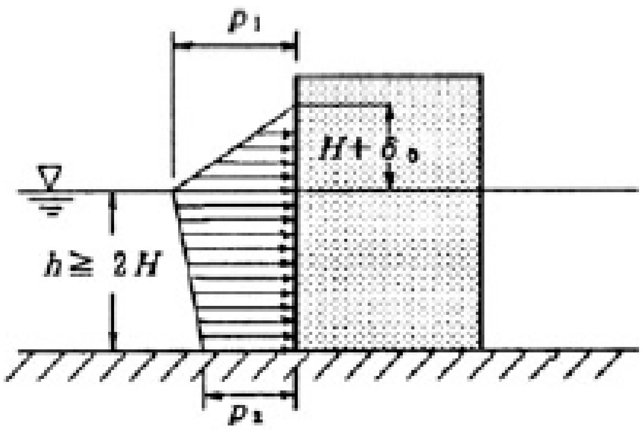

Fig. 3 Wave pressure distribution by Sainflou's formula.

The reliability of Hiroi's formula had been challenged thrice at least. The first challenge was the introduction of Sainflou's formula in 1928 for standing wave pressures. Differentiation of two formulas was made, by referring to the recommendation of PIANC in 1935, in such a way that Hiroi's formula was for the case of the water depth above the rubble foundation being less than twice the incident wave height, while Sainflou's formula was for the water depth equal to or greater than twice the wave height. The second challenge was raised when the concept of significant wave was introduced in early 1950s. Which one of H_{max}, $H_{1/10}$, or $H_{1/3}$ is to be used in Hiroi's formula was the question. A consensus was soon formed as the recommendation for the use of $H_{1/3}$ based on the examination of existing breakwater designs and wave conditions. The third challenge was made by Goda [1973] against the insensitivity of the estimated pressure intensity to the variations in wave period and other factors. Hiroi's formula could not meet this challenge and is not used presently for the design of major breakwaters.

Though the pressure formula by Hiroi was so simple, the total wave force thus estimated was quite reliable on the average. Thanks to this characteristic, Japanese breakwaters had rarely experienced catastrophic damage despite the very long extension around the country.

E.3.2. SAINFLOU'S FORMULA

As well known, Saiflou published a theory of trochoidal waves in front of a vertical wall in 1928 and presented a simplified formula for pressure estimation. The pressure distribution is sketched as in Fig. 3, and the pressure intensities and the quantity of water level rise δ_0 are given as:

$$p_1 = \frac{\left(p_2 + w_0 h\right)(H+\delta_0)}{h+H+\delta_0} \tag{E.2}$$

$$p_2 = w_0 H / \cosh kh$$

$$\delta_0 = \left(\pi H^2 / L\right) \coth kh$$

where L is the wavelength and k is the wavenumber of $2\pi/L$.

Sainflou [1928] presented the above formula for standing wave pressures of nonbreaking type and the formula has been so utilized. The formula was derived for the purpose of practical application from the standpoint of a civil engineer and it has served its objective quite well. Just like the case of Hiroi's formula, it was born when the concept of wave irregularity was unknown. There seems to exist no established rule for the choice of representative wave height to be used with Sainflou's formula. Some advocates the use of $H_{1/3}$, some favors $H_{1/10}$, and the other prefers the selection of $H_{1\%}$.

It was customarily in Japan to use $H_{1/3}$ with Sainflou's formula but in a modified form. Through examinations of several minor damage of breakwaters, it had been revealed that a simple application of Sainflou's formula had yielded underestimation of wave pressures under storm conditions. For the zone extending $\pm H/2$ around the design water level, the wave pressure by Sainflou's foumula was replaced with that by Hiroi's formula. The modified formula was sometimes called the partial breaking wave pressure formula in

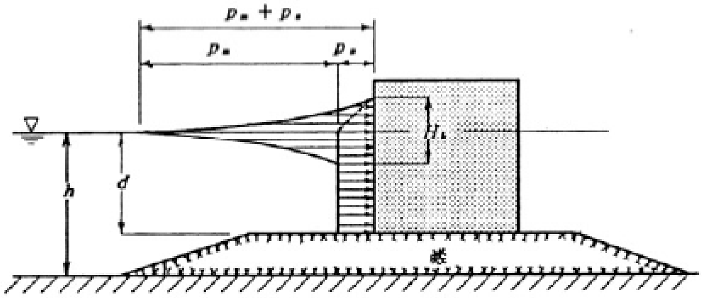

Fig. 4 Wave pressure distribution by Minikin's formula.

Japan, because it was aimed to introduce the effect of partial wave breaking in relatively deep water. The dual system of Hiroi's wave pressure formula for breaking waves and of modified Sainflou's formula for standing waves had been the recommended engineering practice of breakwater design in Japan for the period from around 1940 to the early 1980s.

E.3.3. MINIKIN'S FORMULA AND OTHERS

Although Hiroi's formula had been regarded as the most dependable formula for breaking wave pressures in Japan, it remained unknown in Europe and America. As the field measurement at Dieppe revealed the existence of very high pressures caused by impinging breaking waves and the phenomenon was confirmed by laboratory experiments by Bagnold [1939], harbor engineers in western countries began to worry about the impact breaking wave pressures. Then in 1950, Minikin proposed the following formula for breaking wave pressures, which consisted of the dynamic pressure p_n and the hydrostatic pressure p_s as sketched in Fig. 4:

Dynamic pressure:

$$p_n = p_{max}(1-2|z|/H)^2 \quad \text{for } |z| \le H/2 \tag{E.3}$$

$$p_{max} = 101 w_0 d(1+d/h)(H/L)$$

Hydrostatic pressure:

$$p_s = \begin{cases} 0.5w_0H(1-2z/H) & \text{for } 0 \le z < H/2 \\ 0.5w_0H & \text{for } z < 0 \end{cases} \tag{E.4}$$

Because it was the first descriptive formula for breaking wave pressures, it was immediately accredited as the design formula and listed in many textbook and engineering manuals. Even in present days, technical papers based on Minikin's formula are published in professional journals from time to time.

Minikin [1950] did not give any explanation how he derived the above formulation except for citing the experiments of Bagnold. In the light of present knowledge on the nature of impact breaking wave pressures, the formula has several contradictory characteristics. First, the maximum intensity of wave pressure increases as the wave steepness increases, but the laboratory data indicates that waves with long periodicity tends to generate well developed plunging breakers and produce the impact pressure of high intensity. In fact, Bagnold carried out his experiments using a solitary wave.

Second, Eq. 3 yields the highest p_{max} when d is equal to h or when no rubble foundation is present. It is harbor engineers' experience that a breakwater with a high rubble mound has a larger possibility of being hitten by strong breaking wave pressures than a breakwater with a low rubble mound.

Third, Minikin's formula yields excessively large wave force against which no rational upright breakwater could be designed. To the author's knowledge, no prototype breakwater has ever been constructed with the wave pressures estimated by Minikin's formula. Reanalysis of the stability of prototype breakwaters in Japan which experienced storm waves of high intensity, some undamaged and others having been displaced over a few meters, has shown that the safety factor against sliding widely varies in the range between 0.09 and 0.63 [Goda 1973b and 1974]. The safety factors of undamaged and displaced breakwaters were totally mixed together and no separation was possible. Thus the applicability of Minikin's formula on prototype breakwater design has been denied definitely.

There have been several proposals of wave pressure formulas for breakwater design. Among them, those by Nagai [1968, 1969] and Nagai and Otusbo [1968] are most exhaustive. Nagai classified the various patterns of wave pressures according to the wave conditions and the geometry of breakwater, and presented several sets of design formulas based on many laboratory data. However, his system of wave pressure formulas was quite complicated and these formulas gave different prediction of wave pressures at the boundaries between the zones of their applications. Another problem in the use of Nagai's method was the lack of specification for representative wave height for irregular waves. There was only a few cases of verification of the applicability of his method for breakwater design using the performance data of prototype breakwaters. Because of these reasons, the method is not used in Japan presently.

The Miche-Rundgren formula for standing wave pressure [CERC 1984] represents an effort to improve the accuracy of Sainflou's formula for engineering application. Certainly, the formula would give better agreement with the laboratory data than Sainflou's one. However, it has not been verified with any field data and its applicability for breakwater design is not confirmed yet.

E.4. DESIGN FORMULAE OF WAVE PRESSURES FOR UPRIGHT BREAKWATERS

E.4.1. PROPOSAL OF UNIVERSAL WAVE PRESSURE FORMULAE

It is a traditional approach in wave pressure calculation to treat the phenomena of the standing wave pressures and those by breaking waves separately. Casual observations of wave forms in front of a vertical wall could lead to a belief that breaking wave pressures are much more intensive than nonbreaking wave pressures and they should be calculated differently. The previous practice of wave pressure calculation with the dual formulas of Hiroi's and Sainflou's in Japan was based on such belief. The popularity of Minikin's formula prevailing in western countries seems to be owing to the concept of separation of breaking and nonbreaking wave pressures.

The difference between the magnitudes of breaking and nonbreaking wave pressures is a misleading one. The absolute magnitude of breaking wave pressures is certainly much larger than that of nonbreaking one. The height of waves which break in front of a vertical wall, however, is also greater than that of nonbreaking waves. The dimensionless pressure intensity, p/w_0H, therefore, increases only gradually with the increase of incident wave height beyond the wave breaking limit, as demonstrated in the extensive laboratory data by Goda [1972].

A practical inconvenience in breakwater design with the dual pressure formula system is evident when a breakwater is extended offshoreward over a long distance from the shoreline. While the site of construction is in shallow water, the wave pressures are evaluated with the breaking wave pressure formula. In the deeper portion, the breakwater would be subject to nonbreaking waves. Somewhere in between, the wave pressure formula must be switched from that of breaking to nonbreaking one. At the switching section, the estimated wave pressures jump from one level to another. With the Japanese system of the combined formulas of Hiroi's and modified Sainflou's, the jump was about 30%. To be exact with the pressure calculation, the width of upright section must be changed also. However, it is against the intuition of harbor engineers who believe in smooth variation of the design section. The location of switching section is also variable, dependent on the design wave height. If the design wave height is modified by a review of storm wave conditions after an experience of some damage on the breakwater, then an appreciable length of breakwater section would have to be redesigned and reconstructed.

The first proposal of universal wave pressure formula for upright breakwater was made by Ito et al. [1966] based on the sliding test of a model section of breakwaters under irregular wave actions. Then Goda [1973b, 1974] presented another set of formulas based on extensive laboratory data and being supported by verification with 21 cases of breakwater displacement and 13 cases of no damage under severe storm conditions. The proposed formulas were critically reviewed by the corps of engineers in charge of port and harbor construction in Japan, and they were finally adopted as the recommended formulas for upright breakwater design in Japan in 1980, instead of the previous dual formulas of Hiroi's and modified Sainflou's.

E.4.2. DESIGN WAVE

The upright breakwater should be designed against the greatest force of single wave expected during its service life. The greatest force would be exerted by the highest wave among a train of random waves corresponding to the design condition on the average. Thus the wave pressure formulas presented herein are to be used together with the highest wave to be discussed below.

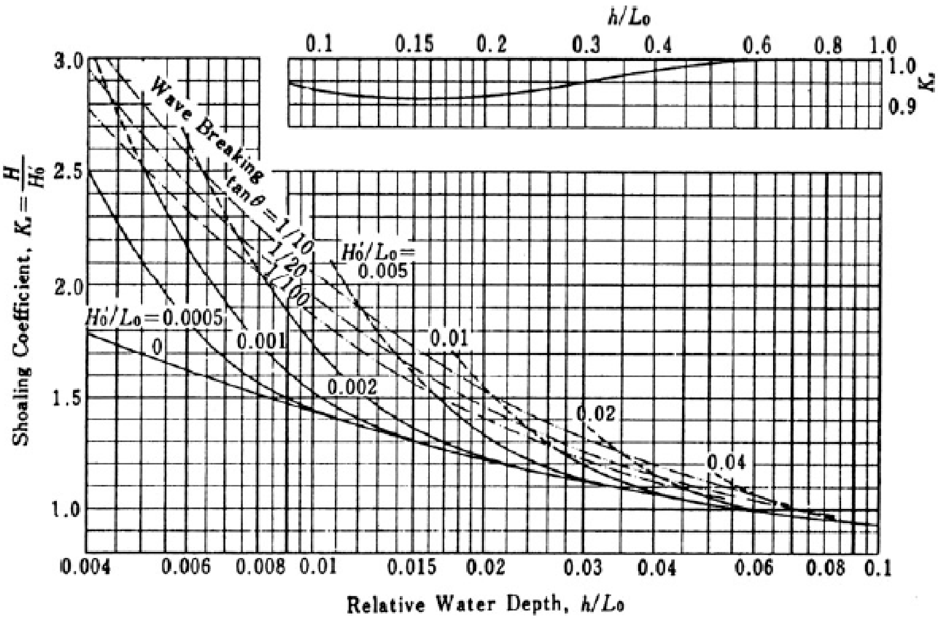

Fig. 5 Diagram of nonlinear wave shoaling coefficient K_s.

WAVE HEIGHT

$$H_{max} = \begin{cases} 1.8 H_{1/3} & \text{for } h/L0 \geq 0.2 \\ \min\{(\beta_0^* H'_0 + \beta_1^* h), \beta_{max}^* H'_0, 1.8 H_{1/3}\} & \text{for } h/L_0 < 0.2 \end{cases} \tag{E.5}$$

$$H_{1/3} = \begin{cases} K_s H'_0 & \text{for } h/L0 \geq 0.2 \\ \min\{(\beta_0 H'_0 + \beta_1 h), \beta_{max}^* H'_0, K_s H'_0\} & \text{for } h/L_0 < 0.2 \end{cases} \tag{E.6}$$

in which the symbol min$\{a, b, c\}$ stands for the minimum value among a, b and c, and H_0' denotes the equivalent deepwater significant height. The coefficients β_0 and others have empirically been formulated from the numerical calculation data of random wave breaking in shallow water as follows, after Goda [1975]:

$$\left.\begin{aligned} \beta_0 &= 0.028\left(H'_0/L_0\right)^{-0.38} \exp\left[20 \tan^{1.5}\theta\right] \\ \beta_1 &= 0.52 \exp\left[4.2 \tan\theta\right] \\ \beta_{max} &= \max\left\{0.92, 0.32\left(H'_0/L_0\right)^{-0.29} \exp\left[2.4 \tan\theta\right]\right\} \end{aligned}\right\} \tag{E.7}$$

$$\left.\begin{aligned} \beta_0^* &= 0.052\left(H'_0/L_0\right)^{-0.38} \exp\left[20 \tan^{1.5}\theta\right] \\ \beta_1^* &= 0.63 \exp\left[3.8 \tan\theta\right] \\ \beta_{max}^* &= \max\left\{1.65, 0.53\left(H'_0/L_0\right)^{-0.29} \exp\left[2.4 \tan\theta\right]\right\} \end{aligned}\right\} \tag{E.8}$$

in which the symbol max$\{a, b\}$ stands for the larger of a or b, and tanθ denotes the inclination of sea bottom.

The shoaling coefficient K_s is evaluated by taking the finite amplitude effect into consideration. Figure 5 has been prepared for this purpose based on the theory of Shuto [1974].

The selection of the fixed relation $H_{max} = 1.8 H_{1/3}$ outside the surf zone was based on three factors of reasoning. First, the fixed ratio was preferred to an introduction of duration dependent relation based on the Rayleigh distribution of wave heights, because such variability in the design wave height would cause some confusion in design procedures. Second, the examination of prototype breakwater performance under severe storm wave actions yielded reasonable results of safety factor against sliding by using the above fixed relation. Third, a possible deviation of the ratio $H_{max}/H_{1/3}$ from 1. 8 to 2.0, say, corresponds to an increase of 11% and it can be covered within the margin of safety factor which is customarily taken at 1.2. However, it is a recommendation and an engineer in charge of breakwater design can use other criterion by his own judgment.

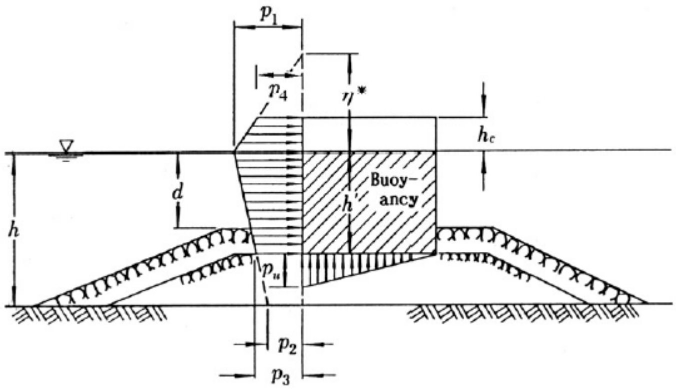

Fig. 6 Wave pressure distribution by Goda's formulas.

For evaluation of H_{max} by the second part of Eq. 7 or within the surf zone, the water depth at a distance $5H_{1/3}$ seaward of the breakwater should be employed. This adjustment of water depth has been introduced to simulate the nature of breaking wave force which becomes the greatest at some distance shoreward of the breaking point. For a breakwater to be built at the site of steep sea bottom, the location shift for wave height evaluation by the distance $5H_{1/3}$ produces an appreciable increase in the magnitude of wave force and the resultant widening of upright section.

WAVE PERIOD

The period of the highest wave is taken as the same with the significant wave period of design wave, i.e.,

$$T_{max} = T_{1/3} \tag{E.9}$$

The relation of Eq. 9 is valid as the ensemble mean of irregular waves. Though individual wave records exhibit quite large deviations from this relation, the use of Eq. 9 is recommended for breakwater design for the sake of simplicity.

ANGLE OF WAVE INCIDENCE TO BREAKWATER

Waves of oblique incidence to a breakwater exert the wave pressure smaller than that by waves of normal incidence, especially when waves are breaking. The incidence angle β is measured as that between the direction of wave approach and a line normal to the breakwater. It is recommended to rotate the wave direction by an amount of up to 15^o toward the line normal to the breakwater from the principal wave direction. The recommendation was originally given by Prof. Hiroi together with his wave pressure formula, in consideration of the uncertainty in the estimation of wave direction, which is essentially based on the 16 points-bearing of wind direction.

E.4.3. WAVE PRESSURE, BUOYANCY AND UPLIFT PRESSURE

ELEVATION TO WHICH THE THE WAVE PRESSURE IS EXERTED

The exact elevation of wave crest along a vertical wall is difficult to assess because it varies considerably from $1.0H$ to more than $2.0H$, depending on the wave steepness and the relative water depth. In order to provide a consistency in wave pressure calculation, however, it was set as in the following simple formula:

$$\eta^* = 0.75\left(1+\cos\beta\right)H_{max} \tag{E.10}$$

For waves of normal incidence, Eq. 10 gives the elevation of $\eta^* = 1.5H_{max}$.

WAVE PRESSURE EXERTED UPON THE FRONT FACE OF A VERTICAL WALL

The distribution of wave pressure on an upright section is sketched in Fig. 6. The wave pressure takes the largest intensity p_1, at the design water level and decreases linearly towards the elevation η^* and the sea bottom, at which the wave pressure intensity is designated as p_2.

The intensities of wave pressures are calculated by the following:

$$\left.\begin{aligned} p_1 &= 0.5\left(1+\cos\beta\right)\left(\alpha_1+\alpha_2+\cos^2\beta\right) w_0 H_{max} \\ p_2 &= p_1 / \cosh kh \\ p_3 &= \alpha_3 p_1 \end{aligned}\right\} \tag{E.11}$$

in which:

$$\left.\begin{aligned} \alpha_1 &= 0.6 + 0.5\,[2kh/\sinh 2kh]^2 \\ \alpha_2 &= \min\left\{[(h_b-d)/3h_b]\,(H_{max}/d)^2, 2d/H_{max}\right\} \\ \alpha_3 &= 1-\left(h'/h\right)[1-1/\cosh kh] \end{aligned}\right\} \tag{E.12}$$

The coefficient α_1 takes the minimum value 0.6 for deepwater waves and the maximum value 1.1 for waves in very shallow water. It represents the effect of wave period on wave pressure intensities. The coefficient α_2 is introduced to express an increase of wave pressure intensities by the presence of rubble mound foundation. Both coefficients α_1 and α_2 have empirically been formulated, based on the data of laboratory experiments on wave pressures. The coefficient α_3 is derived by the relation of linear pressure distribution. The above pressure intensities are assumed to remain the same even if wave overtopping takes place.

The effect of the incident wave angle on wave pressures is incorporated in η^* and p_1 with the factor of $0.5(1+\cos\beta)$ and a modification to the term of α_2 with the factor of $\cos 2\beta$.

BUOYANCY AND UPLIFT PRESSURE

The upright section is subject to the buoyancy corresponding to its displacement volume in still water below the design water level. The uplift pressure acts at the bottom of the upright section, and its distribution is assumed to have a triangular distribution with the toe pressure p_u given by Eq. 13.

$$p_u = 0.5\left(1+\cos\beta\right)\alpha_1\alpha_3 w_0 H_{max} \tag{E.13}$$

The toe pressure p_u is set smaller than the wave pressure p_3 at the lowest point of the front wall. This artifice has been introduced to improve the accuracy of the prediction of breakwater stability, because the verification with the data of prototype breakwater performance indicated some overestimation of wave force if p_u were taken the same with p_3.

When the crest elevation of breakwater h_c is lower than η^*, waves are regarded to overtop the breakwater. Both the buoyancy and the uplift pressure, however, are assumed to be unaffected by wave overtopping.

E.4.4. STABILITY ANALYSIS

The stability of an upright breakwater against wave action is examined for the three modes of failure: i.e., sliding, overturning, and collapse of foundation. For the first two modes, the calculation of safety factor is a common practice of examination. The safety factors against sliding and overturning are defined by the following:

$$\text{Against sliding: } S.F. = \mu\,(W-U)\,/\,P \tag{E.14}$$

$$\text{Against overturning: } S.F. = (Wt - M_U)\,/\,M_P \tag{E.15}$$

The notations in the above equations are defined as follows:

- M_P: moment of total wave pressure around the heel of upright section
- M_U: moment of total uplift pressure around the heel of upright section
- P: total thrust of wave pressure per unit extension of upright section
- t: horizontal distance between the center of gravity and the heel of upright section
- U: total uplift pressure per unit extension of upright section
- W: weight of upright section per unit extension in still water
- μ: coefficient of friction between the upright section and the rubble mound

The safety factors against sliding and overturning are dictated to be equal to or greater than 1.2 in Japan. The friction coefficient between concrete and rubble stones is usually taken as 0.6. The coefficient seems to have a smaller value in the initial phase of breakwater installment, but it gradually rises to the value around 0.6 through consolidation of the rubble mound by the oscillations of the upright section under wave actions.

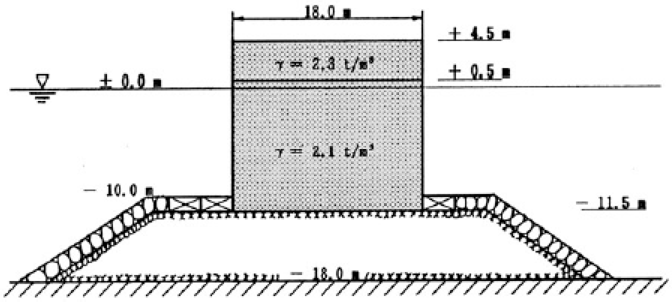

Fig.7 Sketch of upright breakwater for stability analysis.

The fact that most of breakwater displacements by storm waves occur during the construction period or within a few years after construction supports the above conjecture.

The bearing capacity of the rubble mound and the sea bottom foundation was used to be examined with the bearing pressures at the heel of upright section and at the interface between the rubble mound and the foundation. However, a recent practice in Japan is to make analysis of circular slips passing through the rubble mound and the foundation, by utilizing the simplified Bishop method (see Kobayashi et al. 1987). For the rubble mound, the apparent cohesion of c = 2 tf/m^2 and the angle of internal friction of ϕ =35^o are recommended.

E.4.5. EXAMPLE OF WAVE PRESSURE CALCULATION

An example of calculation is given here in order to facilitate the understanding of the breakwater design procedure. The design wave and site conditions are set as in the following:

Waves:	$H'_0 = 7.0$ m	$T_{1/3} = 11$ s	$\beta = 10^o$	
Depth etc.:	$h = 18$ m	$d = 10$ m	$h' = 11.5$ m	$h_c = 4.5$ m
Bottom slope:	$\tan\theta = 1/50$			

The incident wave angle is the value after rotation by the amount up to 15^o. The geometry of upright breakwater is illustrated in Fig. 7.

DESIGN WAVE HEIGHT H_{max} AND THE MAXIMUM ELEVATION OF WAVE PRESSURE η^*

The coefficients for wave height calculation are evaluated as

$L_0 = 188.8$ m, $H'_0/L_0 = 0.0371$, $h/L_0 = 0.0953$, $K_s = 0.94$
$\beta_0 = 0.1036$, $\beta_1 = 0.566$, $\beta_{max} = \max\{0.92, 0.84\} = 0.92$
$\beta_0^* = 0.1924$, $\beta_1^* = 0.680$, $\beta^*_{max} = \max\{1.65, 1.39\} = 1.65$

Then, the wave heights and the maximum elevation are obtained as

$H_{1/3} = \min\{10.91, 6.44, 6.58\} = 6.44$ m
$h_b = 18.0 + 5 \text{ x } 6.44/50 = 18.64$ m
$H_{max} = \min\{14.02, 11.55, 11.84\} = 11.55$ m
$\eta^* = 0.75\times\,(1 + \cos(10^o) \times 11.55 = 17.19$ m

PRESSURE COMPONENTS

The wavelength at the depth 18 m is $L = 131.5$ m. The coefficients for wave pressure are evaluated as

$k_h = 2\pi \times 18/131.5 = 0.860$
$\alpha_1 = 0.6 + 0.5 \times [2 \times 0.860/\sinh(2 \times 0.860)]2 = 0.802$
$\alpha_2 = \min\{\,[(18.64 - 10.0)/(3 \times 18.64)] \times (11.55/10)2, 2 \times 10/11.55\,\} = \min\{0.206, 1.732\} = 0.206$
$\alpha_3 = 1 - 11.5/18.0 \times [1 - 1/\cosh(0.860)] = 0.820$

Then, the intensities of wave pressure and uplift pressure are calculated as

$p_1 = 0.5 \times (1{+}0.9848) \times [0.802{+}0.206 \times (0.9848)2] \times 1.03 \times 11.55 = 11.83$ tf/m^2
$p_2 = 11.83/\cosh(0.860) = 8.49$ tf/m^2
$p_3 = 0.820 \times 11.83 = 9.70$ tf/m^2
$p_4 = 11.83 \times (1 - 4.5/17.19) = 8.73$ tf/m^2
$p_u = 0.5 \times (1{+}0.9848) \times 0.802 \times 0.820 \times 1.03 \times 11.55 = 7.76$ tf/m^2

The symbol p_4 denotes the pressure intensity at the top of upright section.

TOTAL PRESSURE AND UPLIFT, AND THEIR MOMENTS

Total pressure and uplift, and their moments:

$P = 0.5 \times (11.83{+}9.70) \times 11.5{+}0.5 \times (11.83{+}7.76) \times 4.5 = 167.9$ tf/m
$M_P = 1366.2$ tf-m/m
$U = 0.5 \times 18.0 \times 7.76 = 69.8$ tf/m
$M_U = (2/3) \times 69.8 \times 18 = 837.6$ tf-m/m

STABILITY OF UPRIGHT SECTION AGAINST WAVE ACTION

The specific weight of upright section is assumed as in the following:
The portion above the elevation + 0.5 m : $\gamma_c = 2.3$ tf/m^3
The portion below the elevation + 0.5 m: $\gamma'_c = 2.1$ tf/m^3

The difference in the specific weight reflects a current practice of sand filling in the cells of concrete caisson. The weight of upright section is calculated for the dry and in situ conditions, respectively, as

$W_a = 2.1 \times (11.5 + 0.5) \times 18.0 + 2.3 \times (4.5 - 0.5) \times 18.0 = 619.2$ tf/m
$W = 619.2 - 1.03 \times 11.5 \times 18.0 = 406.0$ tf/m

The safety factors against sliding and overturning of the upright section are calculated as in the following:

Against sliding: $S.F. = 0.6 \times (406.0 - 69.8)/167.9 = 1.20$
Against overturning: $S.F. = (406.0 \times 9.0 - 837.6)/1366.2 = 2.06$

Therefore, the upright breakwater with the uniform width of $B = 18.0$ m sketched in Fig. 7 is considered stable against the design wave of $H'_0 = 7.0$ m and $T_{1/3} = 11.0$ s.

E.5. DISCUSSION OF SEVERAL DESIGN FACTORS

E.5.1. PRECAUTIONS AGAINST IMPULSIVE BREAKING WAVE PRESSURE

The universal wave pressure formulas described hereinbefore do not address to the problem of impulsive breaking wave pressure in a direct manner. The coefficient α_2, however, has the characteristic of rapid increase with the decrease of the ratio d/H_{max}. This increase roughly reflects the generation of impulsive breaking wave pressure.

Though the impact pressure of breaking waves exerted upon a vertical wall is much feared by coastal and harbor engineers, it occurs under the limited conditions only. If waves are obliquely incident to a breakwater, the possibility of impact pressure generation is slim. If a rubble mound is low, the sea bottom should be steep and waves be of swell type for the impact pressure to be generated. A most probable situation under which the impact pressure is exerted upon an upright breakwater is the case with a high rubble mound with an appreciable berm width (see Tanimoto et al. 1987). Most of breakwater failures attributed to the action of the impulsive breaking wave pressure are due to the wave forces of normal magnitude, which could be estimated by the universal wave pressure formulas described in the present lecture note.

The impact pressure of breaking waves last for a very short time duration, which is inversely proportional to the peak pressure intensity. In other words, the impulse of impact pressure is finite and equal to the forward momentum of advancing wave crest which is lost by the contact with the vertical wall. The author has given an estimate of the average value of the impact pressure effective in causing sliding of an upright section, by taking into account the elastic nature of a rubble mound and foundation [Goda 1973a]. Because the major part of impact is absorbed by the horizontal oscillations and rotational motion of the upright section, the impact pressure effective for sliding is evaluated as $(2 \sim 3)\, w_0 H_{max}$.

Nevertheless,the pressure intensity of the above order is too great to be taken into the design of upright breakwaters: the mean intensity of wave pressure employed for the stability analysis of the breakwater sketched in Fig. 7 is only $0.91 w_0 H_{max}$. Engineers in charge of breakwater design should arrange the layout and the cross section of breakwater in such way to avoid the danger of impact pressure generation. If the exertion of impulsive breaking wave pressure on the upright section seems inevitable, a change in the type of breakwater structure, such as a sloping type breakwater or a vertical breakwater protected by a mound of concrete blocks, should be considered.

E.5.2. STRUCTURAL ASPECTS OF REINFORCED CONCRETE CAISSON

The upright section of vertical breakwater is nowadays made by reinforced concrete caisson. The width is determined by the stability condition against wave action. The height of caisson or the base elevation is so chosen to yield the minimum sum of the construction cost of rubble mound and upright section.

The length of caisson is governed by the capacity of manufacturing yard. In March 1992, Kochi Port facing the Pacific in Shikoku, Japan, set a breakwater caisson with the length 100 m in position. It is of hybrid structure with steel frames and prestressed concrete.

A concrete caisson is divided into a number of inner cells. The size of inner cells is limited to 5 m or less in ordinary design. The outer wall is 40 to 50 cm thick, the partition wall 20 to 25 cm thick, and the bottom slab 50 to 70 cm thick. These dimensions are subject to the stress analysis of reinforced concrete. As the upright breakwater withstands the wave force mainly with its own weight, the use of prestressed concrete for breakwater caisson is not advantageous in the ordinary situations. For the caisson of special shapes for enhancing wave dissipation such as the caisson with circular arc members, prestressed concrete is utilized.

E.5.3. ARMOR UNITS FOR RUBBLE MOUND

The berm and slope of a rubble mound needs to be protected with armor units against the scouring by wave action. Foot-protection blocks weighing from 15 to 50 tf are placed in front of an upright section. The rest of the berm and slope are covered by heavy stones and/or specially shaped concrete blocks. The selection of armor units is left to the judgment of engineers, with the aid of hydraulic model tests if necessary.

A formula for the weight of armor stones on the berm of rubble mound has been proposed by Tanimoto et al. [1982] as the results of systematic model tests with irregular waves. The minimum weight of armor stones can be calculated by a formula of the Hudson type:

$$W = \gamma_r H_{1/3}^3 / \left[N_s^3 (S_r - 1)^3 \right] \tag{E.16}$$

in which W is the weight of armor stones, γ_r the specific weight of armor stones, S_r the ratio of γ_r to the specific weight of seawater, and N_s, the stability number, the value of which depends on the wave conditions and mound dimensions.

For waves of normal incidence, Tanimoto et al. [1982] gave the following function for armor stones:

$$N = \max\left\{1.8, \left[1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left(-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right)\right]\right\} \tag{E.17}$$

in which the parameter κ is calculated by

$$\kappa = \left[2kh'/\sinh 2kh'\right]\sin\left(2\pi B_M/L'\right) \tag{E.18}$$

and where h' denotes the water depth at which armor stones are placed, L' the wavelength at the depth h', and B_M the berm width.

Though the stability number for concrete blocks has not been formulated, a similar approach to the data of hydraulic model tests on concrete blocks will enable the formulation of the stability number for respective types of concrete blocks.

E.6. CONCLUDING REMARKS

The design and construction of upright breakwaters is a well established, engineering practice, at least in Japan, Korea, and Taiwan. A large number of these breakwaters have been built and will be built to protect ports and harbors. In these countries, the problem of impulsive breaking wave pressure is rather lightly dealt with. The tradition owes to Prof. Hiroi, who established the most reliable wave pressure formula in shallow water and showed the upright breakwaters could be successfully constructed against breaking waves.

This is not to say that no breakwaters have failed by the attack of storm waves. Whenever a big storm hits the coastal area, several reports of breakwater damage are heard. However, the number of damaged caissons is very small compared with the total number of breakwater caissons installed along the whole coastline. Probably the average rate per year would be less than 1%, though no exact statistic is available. Most cases of breakwater damage are attributed to the underestimation of the storm wave condition when they were designed.

In the past, the majority of breakwaters were constructed in relatively shallow water with the depth up to 15 m, for example, because the vessels calling ports were relatively small. In such shallow water, the storm wave height is controlled by the breaking limit of the water depth. One reason for the low rate of breakwater failure in the past could be this wave height limitation at the locations of breakwaters. The site of breakwater construction is moving into the deeper water in these days. Reliable evaluation of the extreme wave condition is becoming the most important task in harbor engineering, probably much more than the improvement of the accuracy of wave pressure prediction.

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F

OPTIMUM BREAKWATER DESIGN

In this appendix a method is presented to calculate an optimal breakwater armour size. This method has been published by VAN DE KREEKE AND PAAPE [1964]. The total cost of the breakwater armour consist of the initial cost and the total capitalised maintenance cost. An economic optimum can be found. However this method has not been applied often in practice. The reason is that often no economic optimum is looked for, because the money for the building costs usually come from other funds than the cost for the maintenance (e.g. the initial costs are paid from a World Bank loan while maintenance comes from the yearly budget of the Port Authority). However, it is from a viewpoint of national economy wise to look for this economic optimum. In this example we use the following wave climate:

Wave Height H (m)	Probability of Exceedance (times per annum)
4	1.11
5	$1.58*10^{-1}$
5.2	$8.4*10^{-2}$
5.5	$7.62*10^{-2}$
5.8	$3.8*10^{-2}$
6	$2.47*10^{-2}$
6.5	$7.35*10^{-3}$
7.15	$3.0*10^{-3}$
7.25	$2.63*10^{-3}$
7.8	$9.0*10^{-4}$
7.98	$8.0*10^{-4}$
8.7	$1.5*10^{-4}$

Table F.1: Long-term wave climate

Also is given that in case of a storm with an actual wave height more that the the design wave height, damage will occur. The amount of damage is given in Table F.2. These values can be calculated e.g. using a Van der Meer equation.

Actual Wave Height H	Damage in % of armour layer
$H < H_{nd}$	0
$H_{nd} < H < 1.3H_{nd}$	4
$1.3H_{nd} < H < 1.45\ H_{nd}$	8
$H > 1,45\ H_{nd}$	Collapse

Table F.2: Long-term wave climate

The initial construction cost I of the breakwater is estimated to be EUR 8620 for the core and EUR $1320 \cdot H_{nd}$ for the armour layer. For design wave heights of 4, 5, 5.5 and 6 m this results in initial construction cost as per Table F.3.

Design wave height H_{nd}	Initial cost breakwater "C"	Initial cost Amour Layer "A"
(m)	(€) per running meter	(€) per running meter
4	13900	5280
5	15220	6600
5.5	15900	7280
6	16540	7920

Table F.3: Initial construction cost per running meter

H_{nd}	$1 < H < 1.3\ H_{nd}$ n = 4% damage			$1.3\ H_{nd} < H < 1.45\ H_{nd}$ n = 8% damage			$H > 1.45\ H_{nd}$ Collapse		
	Δp	Δw	$\Delta p.\Delta w$	Δp	Δw	$\Delta p.\Delta w$	Δp	Δw	$\Delta p.\Delta w$
(m)	(1/year)	(€)	(€/year)	(1/year)	(€)	(€/year)	(1/year)	(€)	(€/year)
4	1.02	420	430	$4.6\ 10^{-2}$	860	40	$3.8\ 10^{-2}$	13900	530
5	$1.5\ 10^{-1}$	530	80	$4.7\ 10^{-3}$	1060	5	$2.6\ 10^{-3}$	15220	40
5.5	$7.4\ 10^{-2}$	580	40	$2.2\ 10^{-3}$	1160	-	$8\ 10^{-4}$	15900	10
6	$2.4\ 10^{-2}$	630	15	$7.5\ 10^{-4}$	1260	-	$1.5\ 10^{-4}$	16540	3

Table F.4: Annual risk for various values of H_{nd} per category of damage level

Note:
Δp = $p_I - p_{I+1}$ probability of occurrence of the wave height in the indicated interval
p_I = probability of exceedance of the wave height at the lower limit of the interval
p_{I+1} = probability of exceedance of the wave height at the upper limit of the interval
Δw = cost of repair of the armour layer $(2 \cdot n \cdot A)$ respectively cost of replacement (C)

This leads to the values of average annual risk $s = \sum(\Delta p \cdot \Delta w)$ as shown in Table F.5. For a lifetime of 100 years, which is a reasonable assumption for a breakwater, capitalisation on an interest rate of 3.33% leads to the figures as given in Table F.6. It is now a simple exercise to add the initial cost I and the capitalised maintenance cost S as in Table F.7. The optimum values are printed in bold.

H_{nd}	$s = \Sigma(\Delta p.\Delta w)$		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	(€ per year)	(€ per year)	(€ per year)
4	1000	570	530
5	125	45	40
5.5	50	10	10
6	18	3	3

Table F.5: Average annual maintenance cost for various maintenance strategies

H_{nd}	Capitalised risk S		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	(€)	(€)	(€)
4	30000	17100	15900
5	3750	1350	1200
5.5	1500	300	300
6	540	90	90

Table F.6: Capitalised maintenance cost for various maintenance strategies

H_{nd}	Total cost $I + S$		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	(€)	(€)	(€)
4	43900	31000	29800
5	18970	16570	16420
5.5	17400	**16200**	**16200**
6	**17080**	16630	16630
6.5	17300		

Table F.7: Total cost for various maintenance strategies

G

Construction equipment

G.1. General

This appendix (based on the Rock Manual [2007]) provides a review of the types of equipment used for placing armourstone in rock fill structures and distinguishes land-based from waterborne operations. Included within land-based operations is the use of land-based equipment to place armourstone below the waterline. Some typical plant capacities are dis-cussed along with related construction aspects. An important factor governing the choice of equipment is the distinction between direct dumping of bulk material, for example in the core of a breakwater, and controlled placement of individual pieces of armourstone, such as in armour layers and underlayers of slope and/or bed protection works. Typically, controlled placement involves dumping of limited quantities per cycle or placement of individual stones.

Land-based operations

For land-based operations, dump trucks may be used for direct dumping of bulk material, if necessary in combination with bulldozers, wheel loaders, hydraulic excavators and wire-rope cranes. Hydraulic cranes and excavators can be used for individual placement, while wire rope cranes are often used for concrete armour units. Manufacturers of construction equipment maintain catalogues with full specifications for all their products. Many manufacturers also make this information available on the Internet.

For most projects, the client will commission contractors possessing specific machinery, some of which may have been modified to optimise performance of common tasks. General characteristics of these types of equipment are set out in Table G.1, while specific information can be obtained from contractors engaged on site.

Waterborne operations

For waterborne operations, the following types of vessel are used for the direct dumping of bulk material:

- split-hopper barges and side stone-dumping vessels
- crane barges equipped with rock trays
- flat-top barges with wheel loader.

For controlled placement the following can be used:

- side stone-dumping vessels
- pontoons with hydraulic excavator or wire-rope crane
- flexible fall-pipe vessels
- trailing suction hopper dredgers, equipped to place gravel via a pipe.

G.2. Land-based equipment – dumping of material

Rockfill is placed by directly dumping (bulk) material using trucks or loaders, hydraulic excavators or wire-rope cranes. Table G.1 lists some commonly used land-based equipment. The values of engine power, capacity, own mass and width are merely indicative and are approximate ranges for small to very large pieces of equipment. The type of equipment required depends on the size of the job and the working conditions on site.

Equipment	Engine power (hp*)	Own mass (t)	Capacity	Operating width (m)
Bulldozer	140–410	17–79		3.26–4.31
Excavator	140–515	22–85	1.2–4.6 m3	2.80–3.50
Wheel loader	235–475	23–50	3.6–6.6 m3	3.15–3.90
Articulated dump truck	280–415	23–35	23.6–38.1 t	2.90–3.45
Off-highway dump truck	485–730	24–76	39.3–66.5 t	5.00–5.10
Highway dump truck	225–375	12–20	12.5–25.0 t	2.55
Grab and wire-rope crane	150–375	50–160	65–325 tm	4.30–6.45
Lift/wire-rope crane	350–750	150–350	500–1500 tm	6.00–8.50

Note: 1 hp = 0.746 kW

Table G.1: Overview of equipment types with the ranges for power, mass, capacity and width

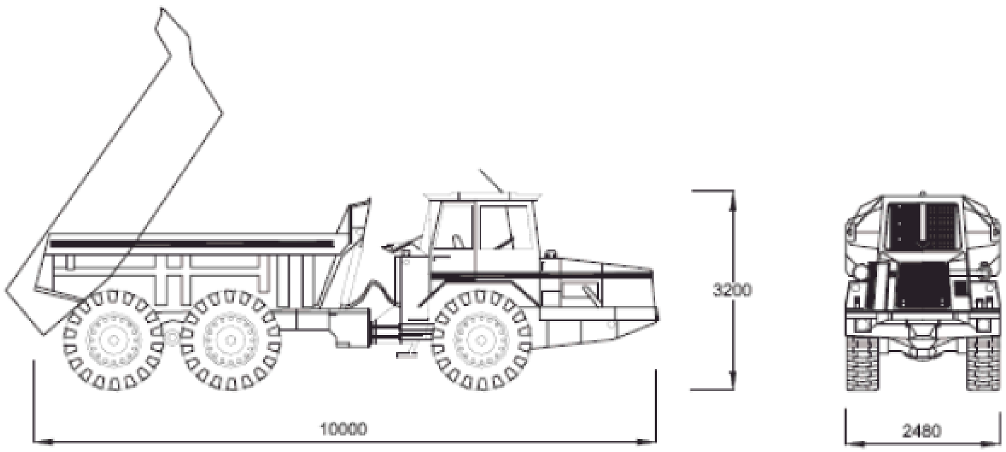


Figure G.1: Large articulated dump truck, typical dimensions (mm)

DUMP TRUCKS

The simplest method of placement is to dump bulk material directly by highway or off-highway dump trucks, usually carrying loads varying between 20 t and 50 t (larger if quarry plant is available) and often with the assistance of a bulldozer to spread the dumped materials. These trucks require an access or haul road that is at least 4 m wide. The size of truck required depends on the armourstone grading. If there is only single-lane road access, regularly spaced passing places at least 7 m wide should be provided.

There are two types of dump truck: highway and off-highway. Off-highway dump trucks are suitable for driving with heavy loads, heavy armourstone and over rough terrain, e.g. on stones up to a size of about 300 kg. The trucks are subject to considerable wear when loaded with armourstone so they should have strengthened or protected bodies. Loaded off-highway dump trucks are not permitted on public roads because of damage caused by their high axle loads. Rubber and rubber-coated bodies for these vehicles are now available, the use of which reduce both wear and noise. Figure A7-1 illustrates a large articulated dump truck (ADT) with typical dimensions.

Both productivity and resistance to breakdown are improved by the provision of good-quality haul roads on site. This is especially important when highway vehicles need to deliver materials directly to the site.

Dump trucks are used to transport the armourstone from temporary stockpiles to the final placement position. In the UK, armourstone is often delivered to the beach at high water by barge or side stone-dumping vessel. At low water, the armourstone is recovered and loaded into the dump trucks for transport to the placement location.

Dump trucks are not designed to drive over armourstone; small material should therefore be used to blind

Figure G.2: Wheel loader working from beached barge (Royal Boskalis Westminster)

off the armourstone. This blinding may need to be replenished at every tide and may be removed at the end of its use, to maintain the porosity of the core or underlayer.

Wheel loaders

Wheel loaders (see Figure G.2) may be used when the armourstones can be obtained from a stockpile directly adjacent to the work site, such as in small breakwaters or for the construction of embankments and revetments. Compared with trucks, wheel loaders facilitate stone placing further out from the crest and in a more controlled manner.

The use of wheel loaders to place stone in bulk is limited to gradings up to 300 kg, i.e. for the placement of core material, and in some cases for the secondary layers. Wheel loaders with buckets tend to scoop up surface material when digging into a stockpile, which may result in contamination. If the bucket is replaced with forks, larger stones can be handled individually without contamination.

Excavators

All excavators (see Figure A7-3) should have heavy-duty, waterproofed undercarriages, which will improve their life. Biodegradable oil should be used whenever possible in the hydraulic systems of excavators working in pollution-sensitive environments, so that problems do not arise if a hydraulic hose breaks. It is important that all the excavators carry oil spill kits to mitigate the effects of leaks of engine oil or diesel. Plant refuelling should take place in a compound away from the beach or riverbank that is equipped with bunded tanks and quick-release hoses. Long-reach equipment (see Figure A7-4.) is often used to extend the period of tidal working, but this reduces the excavator's capacity, necessitating the use of larger machines. Table G.2 relates the minimum excavator size to the stone size.

Armourstone grading	Excavator size (t)
Core material	15
1–3 t	20
3–6 t	30
6–10 t	45
10–15 t	60
15–20 t	70

Table G.2: Excavator size in relation to stone size

Figure G.3: Excavator working on the crest (courtesy J.D. Simm)

HYDRAULIC WIRE-ROPE OR CRAWLER CRANES

Stone delivered by dump trucks or wheel loaders can also be placed by wire-rope cranes (see Figure G.4). When placing bulk material these cranes can work with skips or rock trays that are filled at the quarry or stockpile and transported to the construction site by trucks, or trays loaded directly at the site. In these cases, heavy cranes are used, which require plenty of space.

The production capacity of this type of crane is determined by the volume of armourstone that it can lift, its working radius and its rotation and lifting speed. Manufacturers provide tables and figures giving lifting capacities that depend on the boom length, boom angle and working radius. If armourstone material is tray-placed the ratio of container to payload is in the order of 1:2 to 1:6.

G.3. LAND-BASED EQUIPMENT – CONTROLLED PLACEMENT

Controlled placement is defined here either as bulk armourstone placement in relatively small quantities per cycle or as the individual placement of heavier pieces of armourstone. The equipment used for this type of armourstone placement is either a hydraulic excavator (Figure G.3) or a wire-rope crane (Figure G.4). For cyclic placement of relatively small quantities of armourstone, hydraulic excavators are more suitable because of their quick duty cycle. Excavators are often equipped with an orange-peel or open-tine grab (see Figure G.6) to dig into the stock of core material dumped by trucks. Alternatively, a bucket or long-reach equipment can be used for this purpose. Wire-rope cranes are suitable for heavy stones and stones that require placing at a greater reach.

The options for the individual lifting of armourstone, sometimes provided with lifting aids, depend on the stone size itself and the handling required and include:

- grabs, chains or dogs
- chain slings
- wire-rope slings
- epoxy-grouted eyebolts or hooks.

The selected method should be assessed for safety. Such assessments usually give preference to the use of grabs and grouted hooks, which only partly depends on the contractor or the equipment he employs. Individual stones may be carried to the site on flat-bed lorries or by barge. The smaller excavators require a

Figure G.4: Crawler crane working on breakwater crest (courtesy Brien Wegner, USACE)

work platform at least 4 m wide, the exact size being governed by the counterweight radius. Larger cranes require a platform up to 8.5 m wide. These are minimum operational widths and make no allowance for passing. Figure G.5 indicates the relationship between the excavator size needed to place a given average armourstone mass and the maximum reach for a given load and excavator size.

provide the operator with considerable flexibility when placing armourstone to a desired position and orientation. A tight packing density is possible, which is important for amenity and safety reasons. A non-rotating grapple provides the operator with less control over the orientation of armourstone pieces than the powered orange-peel grab, but permits positive placing, including pushing and easy pick-up from a stockpile. Another type of grab commonly used is the three- or five-tine power fork (see Figure G.6) on a hydraulic excavator. Although rotating the individual pieces of armourstone is often impractical during placement, tight and rapid placement can be achieved because the armourstone can be pushed into place and does not have to be dropped from the vertical position, as is the case with the other grapples. A power fork can achieve denser placing than a grapple, which requires more space in which to open the tines. Note that where energy absorption is the prime requirement of the design, the armourstone needs to be placed as openly as possible.

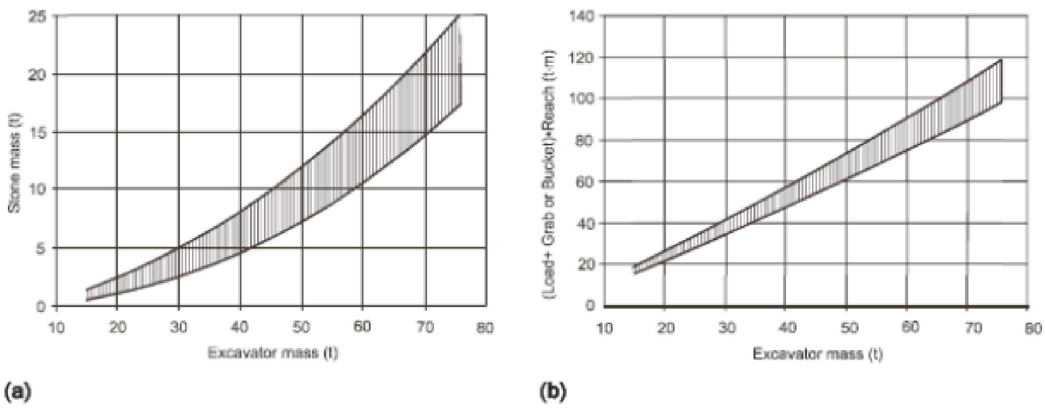


Figure G.5: Indication of the relation between stone mass, required excavator size and maximum reach)

Cactus/orange-peel/closed-tine grab (often termed a grapple in scrap iron works)

Open-tine grab

Five-tine rock fork or power fork (also called a Bofors grab)

Figure G.6: Examples of grabs used for armoured placement (Royal Boskalis Westminster)

In this case, placement demands particular accuracy to achieve stability.

The available bucket mounted on a hydraulic excavator may be used on occasions, although this has the disadvantage that once stones have been placed they are difficult to move, making smooth profiling of the final surface and close or accurate packing harder to achieve. A normal bucket is well suited for levelling and profiling smaller materials, up to about 300 kg.

In all cases the quality of the resulting armour layer depends on the skill of the individual machine operator.

The production capacity of the excavators depends on the volume of the grab, rotation control and speed, and lifting speed. The volume of armourstone per cycle and the maximum mass of each individually placed stone depend on the size and reach of the excavator. The average speed of operation for a rope crane is lower than that for hydraulic excavators. However, in deep water conditions when placing toe armour at large radii it is often necessary to use a wire-rope crane to achieve the necessary reach.

The mass, reach and hoist moment of hydraulic excavators and crawler cranes (also called wire-rope cranes) are interrelated. Table G.3 highlights some of the indicative relationships.

G.4. WATERBORNE EQUIPMENT – DUMPING OF BULK MATERIAL

A successful dumping operation attains the design layer thickness, specified by a mean value and a minimum value, and an optimal rate of armourstone dumped, volume or tonnage per square metre. The dumping process, and consequently the result, are governed by the type of equipment used, water depth, current velocity and by stone characteristics such as density, grading, size and shape. Several types of self-unloading barges can be used, such as:

- split-hopper barges
- bottom door barges (see Figure G.8)
- flat-top barges with wheel loader (see Figure G.2)
- crane barges equipped with rock trays/skips
- side stone-dumping vessels or side-unloading barges (see Figure G.9 and Figure G.10).

Characteristic	Unit	Relationship
Mass of hydraulic grapple	kg	3.25 × grab volume (litres) - 1910
Mass of power fork	kg	55 × excavator mass (tonnes) + 200
Mass of mechanically closed grapple	kg	3.5 × grab volume (litres)
Mass of mechanically open grapple	kg	2.5 × grab volume (litres)
Reach of hydraulic excavator	m	5.8 + 0.06 × excavator mass (tonnes)
Hoist moment over front	tm	1.6 × excavator mass (tonnes) + 2.3
Hoist moment over side	tm	1.2 × excavator mass (tonnes) - 7.6
Reach of crawler cranes	m	5.2 × (crane mass (tonnes))0.4
Hoist moment of crawler crane	tm	0.4 × (crane mass (tonnes))1.31
Rope-operated grabs for crawler cranes:		
• closed-tine grab	t	3.5 × grab volume (m^3)
• open-tine grab	t	2.5 × grab volume (m^3)
Hydraulic grabs for hydraulic		
excavators:	t	2.25 × grab volume (m^3)
• closed-tine grab	t	1.55 × grab volume (m^3)
• open-tine grab	t	0.06 × excavator mass (tonnes)
• power fork		

Table G.3: Indicative relationships between various machine characteristics

These types of vessel are usually employed to dump large quantities of bulk material for core construction, for example in breakwaters, sills or closure dams, where initially there is less need for accuracy of the levels. Figure G.7 shows the use of a skip (or rock tray) for breakwater core construction.

Split-hopper barges are towed or self-propelled, using special propellers for steering and propulsion. They operate by opening the bottom by splitting along the length of the keel. As soon as the opening of the barge exceeds a certain limit, the armourstone is rapidly dumped as a single mass. Dumping usually takes less than one minute. The mass of the material remains concentrated in a cloud, resulting in a fall velocity exceeding the equilibrium fall velocity (V_e) of each individual stone. As a result, the cloud of stones and water will reach the bed with a velocity two to three times V_e . In addition the stones can undergo a wide horizontal displacement after hitting the sea bed. The impact of this kind of dumping is very heavy and may result in damage when covering pipelines or cables, particularly in free spanning. When dumping gravel or coarse and light armourstone, a degree of controlled dumping may be necessary by blocking the opening mechanism at a certain reduced width.

NOTE: because of bridging effects in the armourstone mass and an irregular falling pattern, the opening should not be too small.

The use of these vessels is, in most cases, restricted to coarse and light armourstone to prevent bridging and damage to bottom seals during discharge. These vessels usually carry a maximum of around 900 t and need sufficient water depth beneath the keel to allow for the full cargo to be discharged without grounding.

For dumping from flat-top barges and side stone-dumping vessels the location and distribution of the dumped berm on the bottom can be effectively, considering the gradation, water depth and current velocities. Types of self-unloading barges used for direct dumping of bulk material are shown in Figure G.8. The bottom door barge (left) has a high production capacity, while the side stone dumper (right) give much more control on the dumping process.

The mechanism of unloading from side-unloading barges is by sideways movement of the sliding shovels, as shown in Figure G.9. This is an example of a side stone-dumping vessel. When working with a flat-top barge, the unloading is effected by the use of a wheel loader (or a bulldozer). The principle of unloading is the same for both types of barge.

Figure G.7: Placing armourstone from floating barge using a rock tray/skip

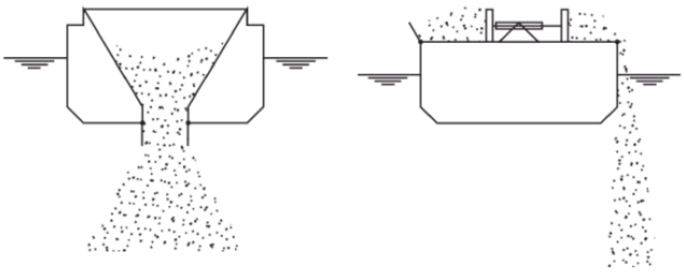


Figure G.8: Types of self-unloading barges for direct dumping of bulk armourstone or core material

G.5. Waterborne equipment – controlled placement

Side stone-dumping vessel or barge

An important feature of these barges is that relatively large quantities of armourstone can be dumped in a controlled manner. The armourstone is either gradually pushed off the loading deck by sliding shovels (see Figure G.9) or transported and passed off the deck by chains or a vibrating-floor system. The speed at which the stones are pushed overboard is an important process parameter with respect to the quality, especially thickness of the dumping.

Depending on construction requirements, the armourstone can either be placed in layers of a prescribed mass per square metre, such as for bed protection works, or in relatively narrow ridges of a prescribed thickness, such as pipeline covers. In the first case, the vessel will be moved slowly in a lateral direction at a specified controlled speed, allowing placement in layers of the order of 0.3–0.5 m thick, on the sea bed or on the core. In the second case, the vessel remains stationary or slowly moves either forward or laterally, depending on the required dimensions of the structure and the local water depth. For this purpose these vessels are often equipped with special propellers for lateral control and a dynamic positioning system that is operated in combination with the moving velocity of the shovel blades. For a controlled discharge operation it is essential that the dumping rate, in kg/s or m^3/s, is low and that each stone may be considered to fall individually. For a side stone-dumping vessel of 1000 t the dumping time is approximately 15 minutes.

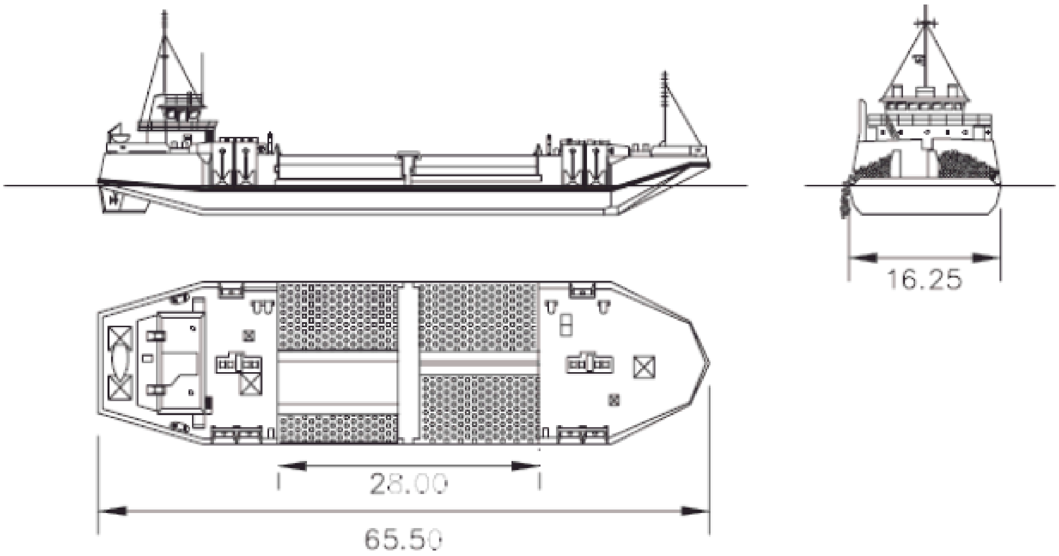


Figure G.9: Plan view and cross-section of the 1000 to side stone dumping vessel Frans, dimensions in m (Van Oord)

The deck of this type of vessel is divided into sections that can be unloaded separately, permitting different types of armourstone to be placed from each section. This may be required when, to ensure the stability of the smaller armourstone in a strong current, a bottom layer of smaller stones has to be covered by bigger ones during the same dumping operation.

For loading capacity, a wide range of suitable vessels is available. The loading capacity varies from 500 t to 2000 t for larger vessels.

Large armourstones can be dumped by side stone-dumping vessels, even very close to existing structures, as shown in Figure G.10

Flat-top barges with wheel loader or excavator

These barges can be used to place relatively large quantities of stone to a reasonably high degree of accuracy. The barges are positioned by using a system of mooring wires and onboard winches. They may also be equipped with special propellers for lateral movement and with a dynamic positioning system. The advantage of flat-deck barges is that, compared with the types of barge described above, they require less specialised equipment (apart, possibly, from a dynamic positioning system) and for this reason they can be used in circumstances where specialised equipment is less readily available. This type of equipment can also place armourstone of different gradings during the same dumping operation. The capacity of these barges can be much higher; typically reaching 5000 t. Figure G.11 shows the placement of reef armourstone with a long-reach excavator from a flat-top barge.

It is possible to use side-unloading barges or side stone-dumping vessels as described above for the construction of an armour layer of relatively small armourstones, for example in breakwaters or for slope protection works. The characteristics of the barge or vessel also depend on the sea conditions in which it has to operate.

Pontoon or vessel with a wire-rope crane

With this type of equipment small quantities of armourstone are placed at a time during each cycle and larger armourstone are placed individually. For example, bed protection works for bridge abutments should be placed in small quantities. Use of side stone-dumping vessels may be less preferable in these circumstances because:

- the area for manoeuvring is limited or
- the total quantities required are small, which makes the use of those vessels uneconomical.

This equipment can also be used for trimming the side slopes of breakwaters or embankments as an alternative to the operation of land-based equipment when the required reach is too large for that type of

Figure G.10: Side stone dumping vessel Cetus, dumping 1-3 t armourstone near a jetty (Royal Boskalis Westminster)

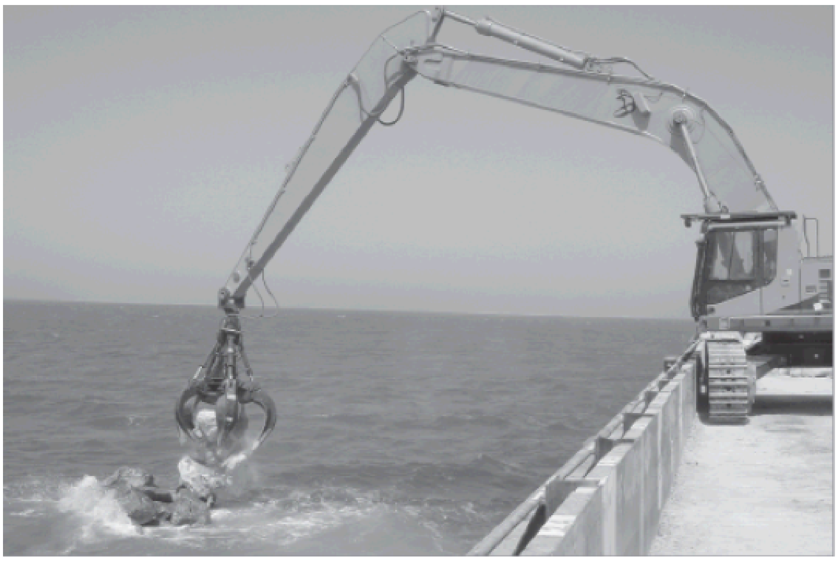


Figure G.11: Placing armourstone with an excavator (Van Oord)

Figure G.12: Crane mounted on the rock supply vessel Jan Steen (Van Oord)

equipment. A barge-mounted crane may also be used to construct submerged dams, sills or bunds with a number of horizontal layers.

Cranes are also used when the accurate placement of individual stones is necessary – for example, when constructing a two-layer system in a breakwater, armourstone is positioned piece by piece. The crane operates from a barge and remains stationary on the site, using an anchoring system, while the armourstone is supplied by separate barges. However, materials supply and placement may also be combined in the same vessel, as shown in Figure G.12. A disadvantage of floating equipment is the sensitivity to wave action. Therefore sometimes a jack-up platform is used, on which a crane or other equipment is placed. Figure G.5 shows an example of a jack-up platform for coastal works and breakwater construction.

G.6. MOVING ON IMPASSABLE SITES

In closure design, access on the worksite for the heavy equipment is sometimes difficult because of the poor conditions of the road. Bearing capacity is not the only determining factor that makes a site passable. This is clear for everyone who tried to drive with a normal car on a sandy beach. On the dry beach a four-wheel driven car with very low gear is needed. In non-granular soil the difference shows for instance in heavy consolidated laterite clay, which is a good base for driving when dry but absolutely impassable after some rain. Since damming activities generally take place in deltaic areas in which sand, sandy clay, clay, silt and peat frequently occur, moving across the site with all sorts of equipment and vehicles under various weather conditions may be a problem. Besides, driving on top of a quarry stone dam will be a problem if the stone is coarser than about 0.5 m in diameter. A distinction has to be made between the heavy operational equipment like dump trucks, cranes, hydraulic excavators, bulldozers and the exploration-equipment used for soil-investigation, positioning finding and measuring. This last group generally consists of lightweight equipment, which is used frequently however, in original terrain conditions. Very special vehicles have been developed for these specific circumstances, although usually they are not readily available and quite expensive sometimes. Hovercraft or Amphirol are examples of these vehicles. The hovercraft supports itself by an airbell confined within rubber skirts, maintained underneath the vehicle by continuous pumping. It can be used on land, on mud flats and on water. The Amphirol (Figure G.14) is supported by two horizontal cylinders provided with opposite winded Archimedean screws. Motion is obtained by rotating the cylinders. Steering is realised by the horizontal orientation of the cylinders and the rotation direction. The Amphirol moves well on soft grounds, mud flats and on water. These vehicles are very useful for small equipment and special assignments.

For the heavy equipment the right system of movement has to be selected. If moving speed is important

Figure G.13: Jack-up “Zeebouwer” platform for coastal works (Deme-group)

Figure G.14: The Amphirol

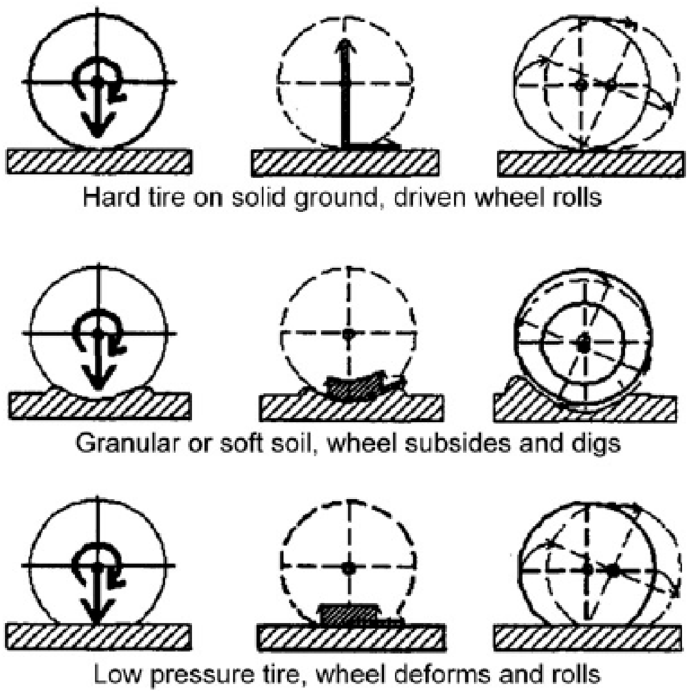


Figure G.15: Motion of a driven wheel

or if driving on site and on public roads has to be combined, the only solution is to use pneumatic tires. Variables with tires are the dimension of the tire, the pressure inside, the number of tires and how many are driven and can be steered.

A wheel has to transfer the forces onto the ground and its motion depends on the reaction forces of the ground. A non-driven wheel transfers a vertical load and a lateral force, while a driven wheel has the load and a rotational momentum. Usually the pressure in a truck's tire is quite high (300 to 500 kN/m^2) and the tire will hardly deform. The reaction force of the road surface or the ground is also high. The maximum lateral force is the friction coefficient times the support force. Deep profiles on the outside of the tire may give it sufficient grip in loose ground and then the shear force in the soil determines the friction coefficient. If the friction is exceeded, the wheel slips. This is the case in the above-mentioned example of wet laterite clay.

If the soil cannot stand the point load of the wheel, the tire will sink into the ground until the load is spread over sufficient bearing area. Consequently, when turning, the wheel has to move up against a slope, by which the support area shifts to the front side of the wheel. A much larger momentum is required to achieve driving and quite soon the friction reaches its maximum. Then the wheel starts turning without lateral movement and it digs itself further down into the soil. This is the case in the dry-beach sand example. A momentum exerted on all wheels (four-wheel drive) and very slow turning will improve the situation.

With a low air pressure in the tire, it is able to deform which gives it a larger support area without subsiding into the soil. Therefore, vehicles with special low-pressure tires (e.g. wheel-loaders) can move much easier but their motion is very springy. For heavy transport that is generally not allowed and then increasing the number of tires is the only solution. For exceptional heavy transport a large number of axles, each with a set of wheels is used. In that case all the wheels are provided with integrated steering capacity.

The next step is to take crawlers instead of wheels (e.g. crawler cranes and bulldozers). They spread the load over a very wide area. Support during driving is not very determining but the shifting of centre of the load and sufficient lateral force during pushing or pulling is important. For different purposes, wide or narrow crawler-tracks can be used. Driving on large quarry stones, for instance, requires narrow tracks. The high point loads exerted by the spanned stones have to be kept close to the centre-line of the track in order not to damage it.

Another solution is to prepare a number of roadways across the area along which surfacing is made to allow vehicles to move along. Those roads are of a temporary nature and should be removable and relatively inexpensive. Two systems are frequently used. One is to pave with large, steel-framed wooden slabs or steel planking. Draglines and cranes can position these themselves and then drive on them.

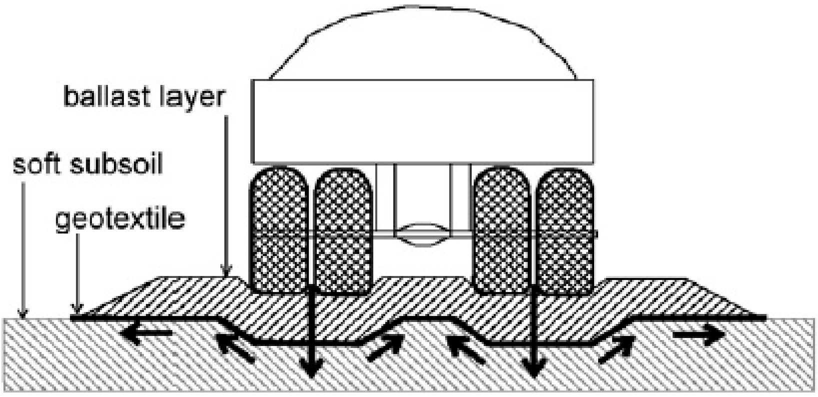


Figure G.16: Temporary road on soft subsoil

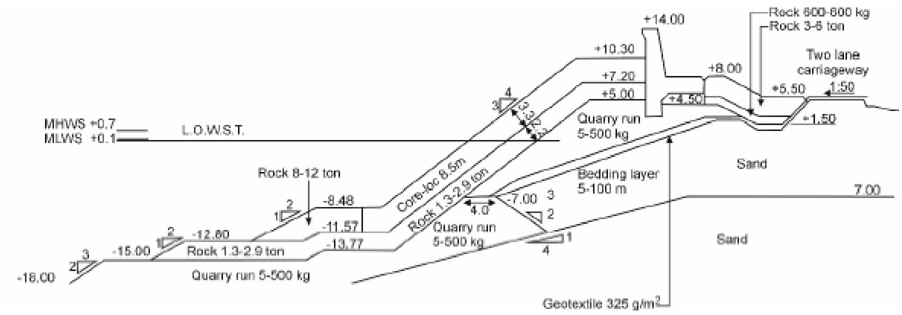
The other system is to provide a road-base direct on the existing soft ground. A geotextile sheet is unrolled and ballasted by a layer of sand or gravel. As soon as the wheels of a truck move on to the ballast layer, the road-base underneath is pressed down into the soil. Due to the sag-bend in the sheet part of the vertical load is transferred sideways into tension in the textile. This horizontal force is taken by friction in the soil for which sufficient length and ballast has to be available. If the trail of the wheel into the ballast bed is too deep, direct sheer of the tire on the sheet will occur and the sheet will tear. Therefore the ballast layer needs to be quite thick and trucks have to prevent deep ruts by regularly shifting tracks. The geotextile has three functions. It separates the subsoil and the ballast material; it transfers the surcharge to the subsoil and spreads the load via tension and elongation to the sides. Besides, due to the separation between ballast and subgrade, the removal of the temporary road is simpler and the ballast material can be re-used easily. A typical event occurs with bulldozers driving on a hydraulic fill. The freshly settled sand still has a large pore volume and is saturated with water. The vibrations of the dozer and its weight (although spread over a large area), will fluidise the top layer of the sand. Generally, this remains within such limits that the bulldozer can continue driving while the sand resettles in a denser grain-structure. Bulldozing leads to densification. However, sometimes the liquefaction covers a too large area and the bulldozer may sink down into the fluidised sand. A fill of fine silty sand (smaller than about 120 micron) is very sensitive for this and hardly passable. Then a long time is needed to await evaporation of the pore water before driving is possible.

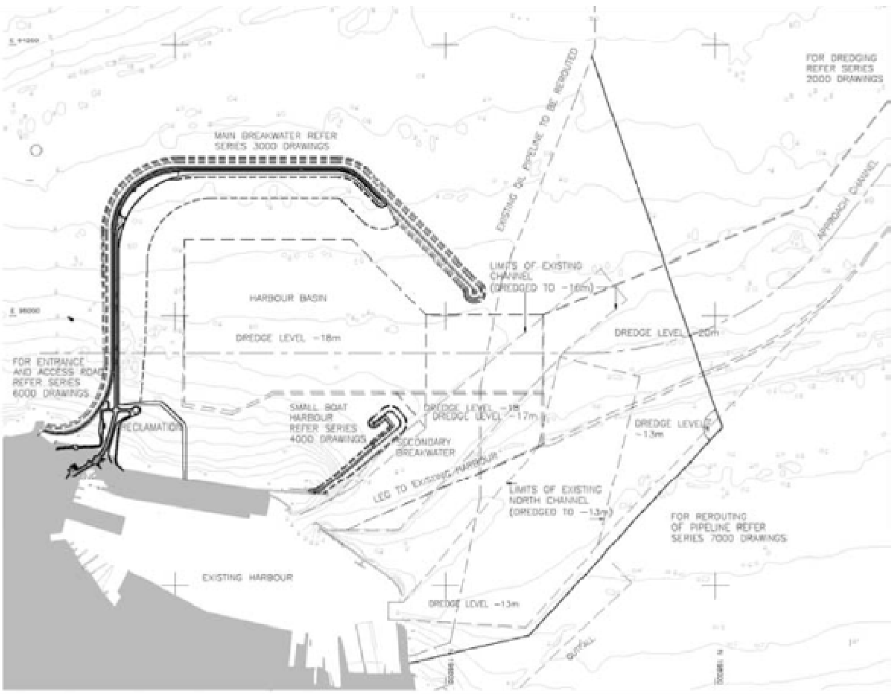
H

BREAKWATER EXAMPLES

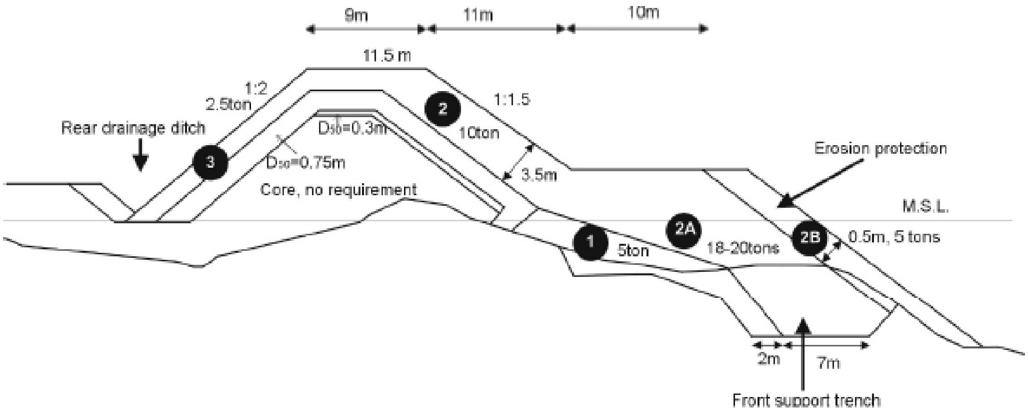
In this appendix some cross sections and plans are given of recently constructed breakwaters.

Colobo, Sri Lanka
(Courtesy Scott Wilson and Colombo Port Authorities)
Design wave H_s = 6.4 m (1/200 years), Core-loc armour units, design 2007, under construction



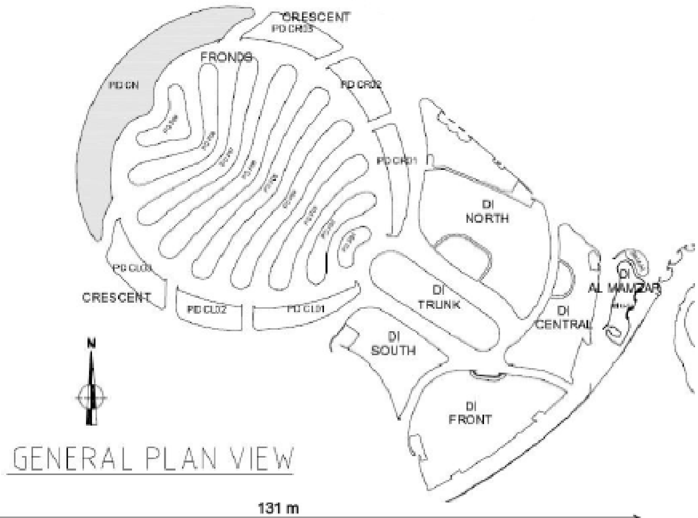


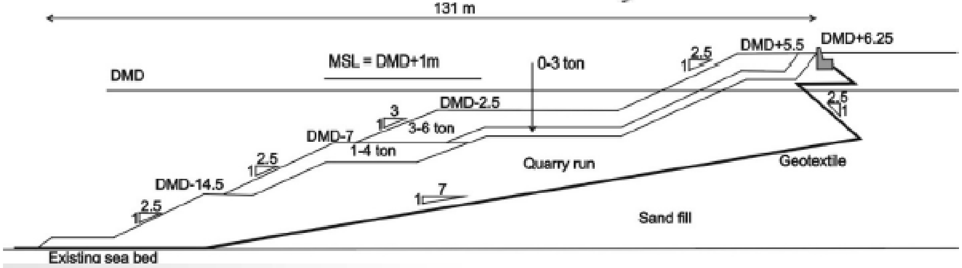
Gismeroy, Norway
(data from Lothe & Birkeland)
Berm breakwater (in fact it is a revetment), $H_s = 5.7$ m, $T_p = 12$ sec



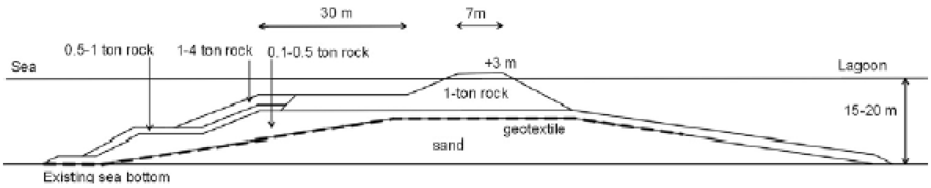
Zone	W_{min}	W_{50}	W_{max}	W'_{max}	remarks
1	3.5	5	no limit	7	Quarried rock
2	7	10	no limit	15	Quarried rock, minimum thickness of layer 3 m
2A		18-20			Uniformly sized, non-local granite boulders Used only the first 25 m from the west
2B	3.5	5	no limit	7	Erosion protection, Quarried rock, minimum thickness of layer 2 m
3	1.5	2.5	3.5		Quarried rock

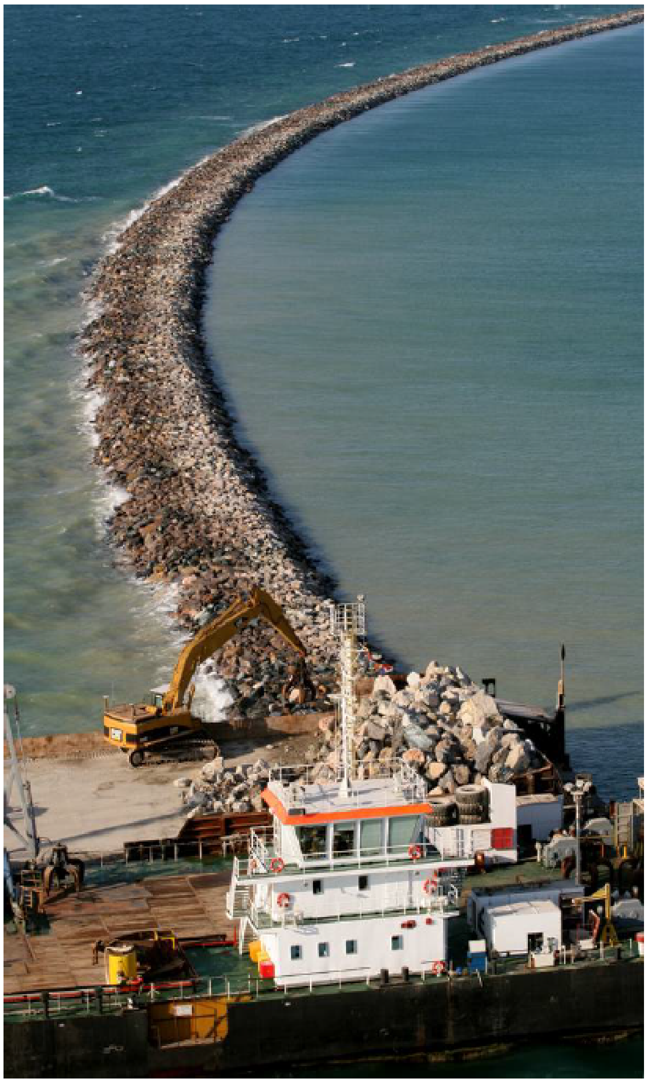
Palm Deira, Dubai, United Arab Emirates
(Courtesy Nakheel, Royal Haskoning, Van Oord)
H_s = 5.5 m during Shamal wind with wind set-up



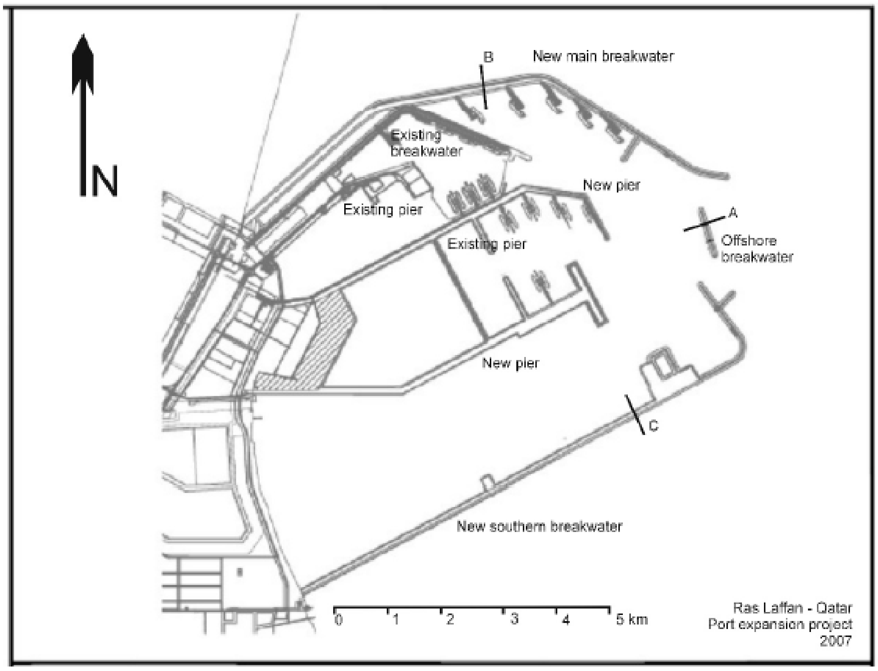


The World Shamal reef, Dubai, United Arab Emirates
(Courtesy Nakheel, Royal Haskoning, Van Oord)
H_s = 5.5 m during Shamal wind with wind set-up

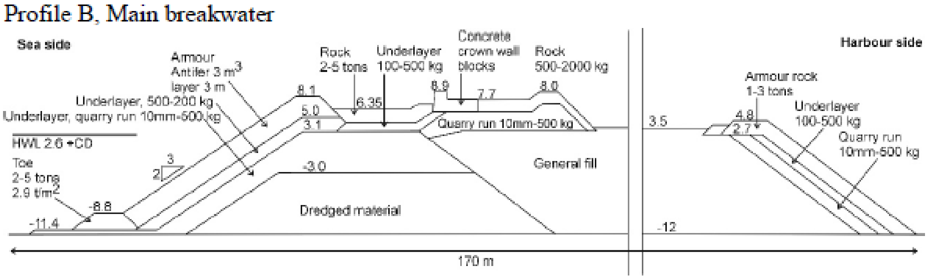




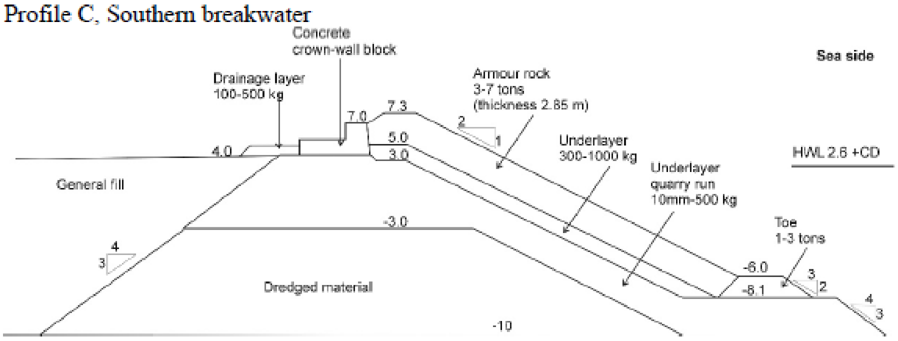
Ras Laffan, Qatar
(Courtesy Qatar Petroleum, Royal Boskalis Westminster/Jan de Nul, Sogreah Consultants)
Accropodes, 3 m^3 and 4 m^3; Antiferblocks 1.5 m^3, 3.0 m^3 and 3.5 m^3.
A: H_{m0} =3.9 m, T_p = 9.4 s B: H_{m0} = 4.5 m, T_p = 11.0 s C: H_{m0} = 2.9 m, T_p = 8.7 s



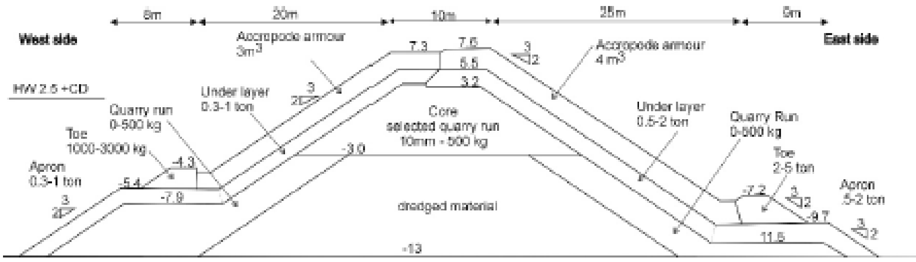
Profile B, Main breakwater



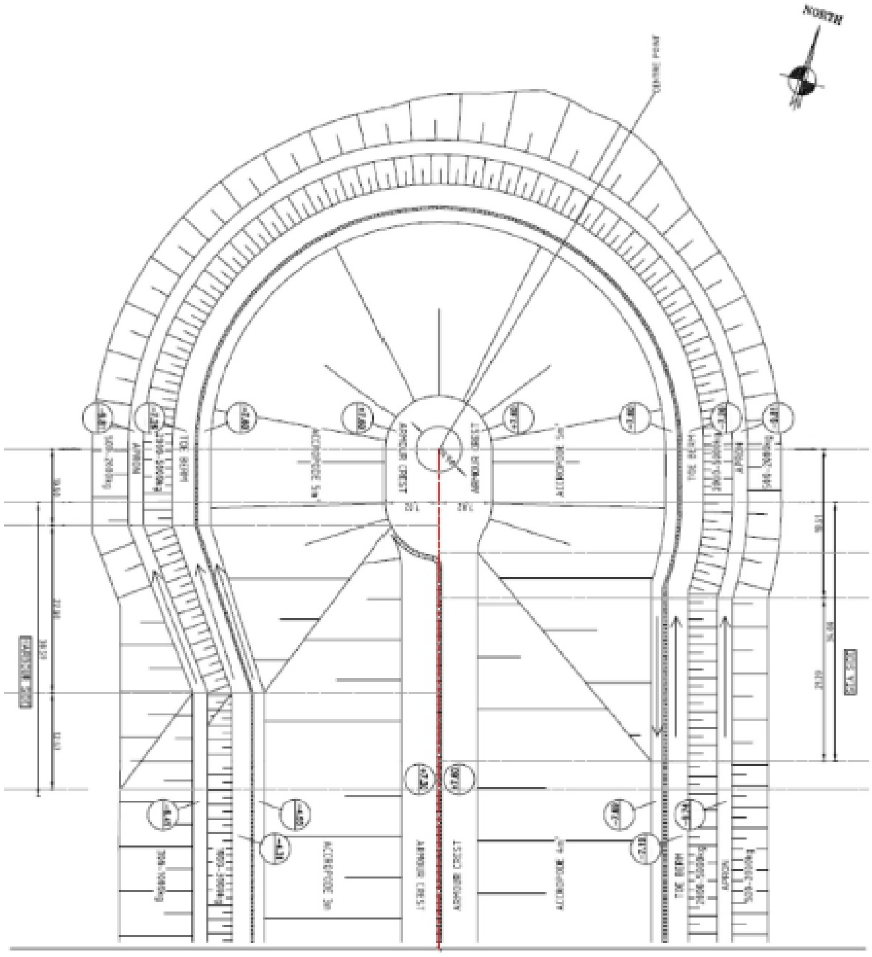
Profile C, Southern breakwater



Offshore (detached) breakwater , profile A

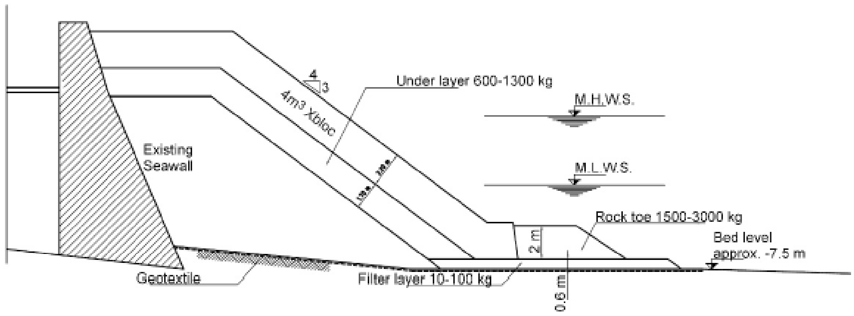


Cross section of the detached breakwater



Head section of the detached breakwater

Port Oriel, Ireland
(Courtesy Delta Marine Consultants and Louth Council)
H_s 5.7 m, Length of breakwater 170 m, completed in 2007, Xbloc



Port Oriel, Ireland

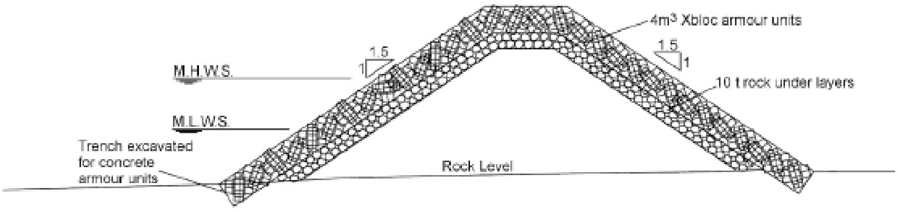
Reference:

VAN DEN BERGE, A., BAKKER, P., MUTTRAY, M., KLABBERS, M., MCALLISTER, D. [2006] Reconstruction of the Port Oriel breakwater, first Xbloc application in Europe, *ICCE San Diego, USA*

Caladh Mor, Ireland

(Courtesy Delta Marine Consultants)

H_s = 4.5 m, Length of shore protection 125 m, completed 2008, Xbloc



Caladh Mor, Ireland

Limbe, Cameroon
(Delta Marine Consultants, rock breakwater)

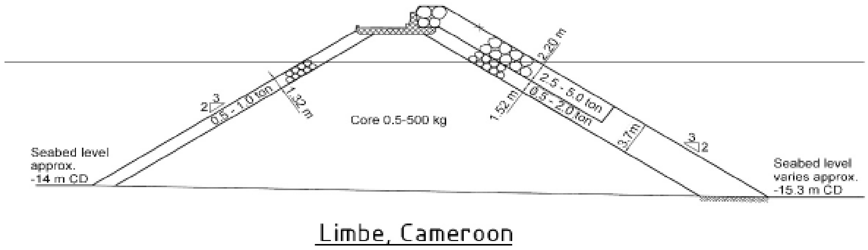
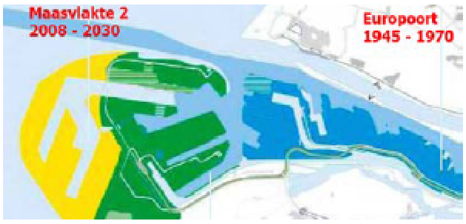


Figure H.1: Limbe breakwater during construction and the official inauguration

Reference:

VOS-ROVERS, I; REEDIJK, B., EKELMANS, S. [2008] Crown-wall with extended base slab; *proc. ICCE2008, Hamburg, Germany*

Rotterdam – Europoort (Netherlands)



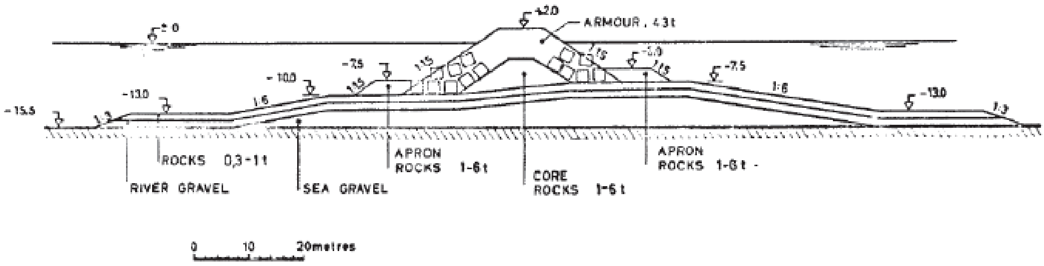
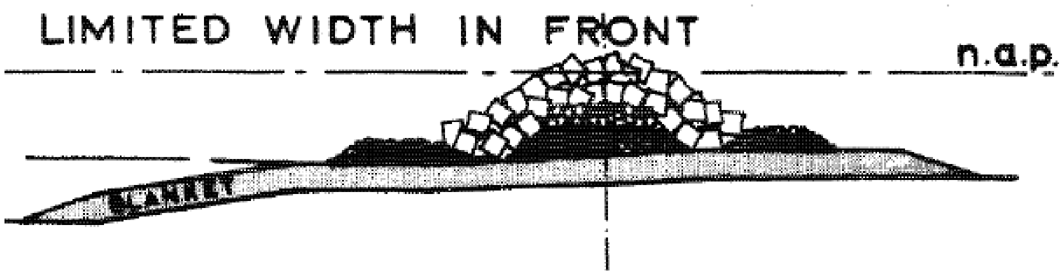


Figure H.2: Cross section of the Hoek van Holland northern breakwater [BERE AND TRAETTEBERG, 1969].

Northern breakwater The entry to the Port of Rotterdam is located at Hoek van Holland (Hook of Holland), a small village some 25 km west of the centre of Rotterdam. The entrance is protected by a northern breakwater with cubes (constructed in 1974) and by a revetted reclamation (Maasvlakte) at the southern side. For the Northern breakwater a design wave height of 8.5 m has been used [FERGUSON, 1969]. The design was one of the first based on model tests with irregular waves.

Maasvlakte 1 In 1974 a new industrial area was reclaimed just south of the entrance of the port. This reclamation is formed as a landfill using dredged sand from the North Sea. Given the reclamation technology of those years it was decided to make first a (low) breakwater at the seaward edge of the reclamation. This breakwater decreases the wave attack and allows traditional land fill methods. The breakwater is composed of a bed protection with on top concrete cubes of 37- 45 tons (2.5 x 2.5 x 2.5 m^3). For practical reasons, the same blocks were used as for the Northern breakwater, so in fact a design wave height of 8.5 m was used. The blocks were placed with special designed block dumping vessels (Libra and Norma). These vessels did also place the blocks on the northern breakwater.

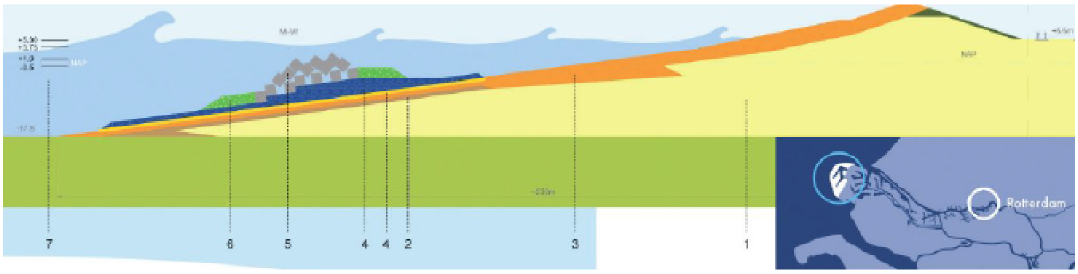


Maasvlakte 2 In 2008 a start was made with the extension of the Maasvlakte reclamation. For this extension also a revetment was needed. Because the outer edge is in deeper water, the hydraulic boundary conditions were different:

Significant wave height	7.95 m	
Spectral peak period	13.5 s	
Main wave direction	300-330 degrees	This design level includes a sea level rise for 50 years.
Directional spreading	25 degrees	
Storm surge level	5.3 m +MSL.	

Figure H.3: Block dumping vessel Norma (Rockdumping.eu)

Figure H.4: Norma and Libra near the harbour light of the (not yet constructed) northern breakwater (rockdumping.eu)



Because the cube reef breakwater at Maasvlakte 1 did perform quite well, is was decided to re-use the existing cubes of the reef breakwater. And like for the Maasvlakte 1 reclamation, the southern part is protected with a beach and dune coast. Only for the northern 3.5 km a hard revetment was needed.

Because of the increased capability of hopper dredges and further development in the rainbow technique to place sand, there was no need any more to make the protected reef breakwater before constructing the reclamation.

Note that between the cobble layer and the sand core no additional filter is applied. In fact the cobble beach is a geometrically open filter.

The blocks from the Maasvlakte 1 reef breakwater were removed by special crane, cleaned and placed on their new location with the same crane (blockbuster).

Because the concrete block reef breakwater lowers the wave height, behind this reef a cobble revetment is sufficient to stabilize the coast. This cobble layer has a d_{50} of 60 – 100 mm. The toe of the reef structure consists of an armour layer of quarry stones (1-10 tons). Below the concrete cubes, four grade quarry stone layers function as a hydraulic filter, thereby preventing internal erosion as well as erosion of the underlying layer.

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GLOSSARY

The glossary below explains a number of terms used in this book. An online version of this glossary (including translations in some languages, e.g. Dutch) is available on the website:
`http://www.kennisbank-waterbouw.nl/Glossary.`

Armour Unit A relatively large quarry stone or concrete shape that is selected to fit specified geometric characteristics and density. It is usually of nearly uniform size and usually large enough to require individual placement. In normal cases it is used as primary wave protection and is placed in layers that are at least two units thick.

Bathymetry The measurement of depths of water in oceans, seas, and lakes; also information derived from such measurements.

Berm breakwater Rubble mound structure with horizontal berm of armourstones at about sea side water level, which is often allowed to be (re)shaped by the waves.

Boulder clay Clay type formed in the ice age, including rocks from moraines; very resistant against erosion.

Breakwater A structure protecting a shore area, harbour, anchorage, or basin from waves

By-passing system When the natural sand transport along a coast is blocked by the construction of a breakwater, a by/passing system transports this sand artificially to the leeside of the breakwater.

Caisson Concrete box-type structure

Chart datum The plane or level to which soundings (or elevations) or tide heights are referenced (usually the level of low water).

Composite breakwater Breakwater consisting of a caisson and a rubble layer in front of it.

CPT (Cone penetration test or Dutch cone test) Test to measure some soil characteristics and soil stratigraphy by pressing a small cone several meters into the subsoil and observing the required force to do so.

Design storm Sea walls will often be designed to withstand wave attack by the extreme design storm. The severity of the storm (i.e. Return Period) is chosen in view of the acceptable level of risk of damage or failure.

Downrush The seaward return of the water following the uprush of the waves. Dutch Cone test see CPT

Dynamic stability A rubble construction is dynamically stable when there are some changes in the profile during extreme wave attack, but on average the same profile will remain unchanged; so after the storm the profile will reshape again to its original shape.

Echo sounder An electronic instrument used to determine the depth of water by measuring the time interval between the emission of a sonic or ultrasonic signal and the return of its echo from the bottom.

Energy density spectrum In ocean wave studies, a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency. The spectrum may be based on observations or theoretical considerations.

Falsework Temporary structures which is used in construction to support spanning or arched structures in order to hold the component in place until its construction is sufficiently advanced to support itself. Falsework also includes temporary support structures for formwork used to mould concrete to form a desired shape and scaffolding to give workers access to the structure being constructed.

Fault tree Schematic presentation of the dependence of failure of the basic top event from basic failure mechanisms. Fault tree analysis is a failure analysis in which an undesired state of a system is analysed using Boolean logic to combine a series of lower-level events.

Freeboard The additional height of a structure above design high water level that is added to prevent overflow. Also, the vertical distance between the water level and the top of the structure at a given time. On a ship, the distance from the waterline to main deck or gunwale.

Grading Distribution with regard to size or weight, of individual stones within a bulk volume. Heavy, light and fine gradings are distinguished.

Gully gully is a watercourse created by running water eroding sharply into soil, typically on a hillside or in intertidal areas.

Liquefaction saturated sand behaving like a fluid

Mattress bed protection made of brushwood and geotextile to prevent erosion of the subsoil due to currents and waves.

Metacentre When a ship is tilted the centre of buoyancy of the ship moves laterally. The point at which a vertical line through the tilted centre of buoyancy crosses the line through the original, non-tilted centre of buoyancy is the metacentre.

Monolithic breakwater A breakwater mad from “one piece of stone”, usually a caisson breakwater or a breakwater made from place blockwork.

Notional permeability Not a real permeability, but an imaginary number representing the permeability of a structure

Osier Branches from the Willow trees (salix)

Peak period The period of the waves with the highest energy in the energy density spectrum

Plunging wave Breaking wave where the crest curls over an air pocket; breaking is usually with a crash. Smooth splash up usually follows.

PoT analysis Statistical method were only peak values above a certain minimum threshold are considered.

Quarry stone Any stone processed from a quarry.

Return Period The average time between two extreme events.

Revetment A facing of stone, concrete, etc., built to protect a scarp, embankment, or shore structure against erosion by wave action or currents.

RMS (root-mean-square) wave height The wave height calculated by taking the square root of the average of the square of all wave heights in a wave field. The RMS wave height represents the energy of the wave field

Rubble mound breakwater A mound of random-shaped and random-placed stones protected with a cover layer of selected stones or specially shaped concrete armour units. (Armour units in a primary cover layer may be placed in an orderly manner or dumped at random.)

Scour Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

Sea wall A structure separating land and water areas and primarily designed to prevent erosion and other damage due to wave action.

Shallow water Water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water

Short basin A basin with a length short in respect to the length of a tidal wave. Basins in the order of 20 km can be considered short.

Significant wave height The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, the average height of the highest one-third of a selected number of waves, this number being determined by dividing the time of record by the significant period.

SLS - Serviceability limit state Condition which a structure should be able to survive without repair

Spectrum see Energy density spectrum

(Standard penetration test) The SPT is an in-situ dynamic penetration test (by counting the number of blows needed to lower a tube over a given distance) designed to provide information on the geotechnical engineering properties of soil. It is mainly used for a gravel, rock and sand.

Squeeze Plastic deformation of soil due to excessive vertical load

Stockpile A stockpile is a pile or storage location for bulk materials, like armourstone or sand for feeding a beach.

Storage area approach Simplified method to calculate flow in an inlet by assuming a short basin with no inertia, and only storage capacity.

Surge (storm surge) A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress.

Surging wave Breaking wave where the wave peaks up, but bottom rushes forward from under wave, and wave slides up beach face with little or no bubble production. Water surface remains almost plane except where ripples may be produced on the beach face during runback.

SWL - Still water level The water level that would exist in absence of sea and swell (instantaneous water level in absence of waves)

Tidal bore A tidal bore is a tidal phenomenon in which the leading edge of the incoming tide forms a wave (or waves) of water that travel up a river or narrow bay against the direction of the current. As such, it is a true tidal wave (not to be confused with a tsunami).

Trade wind The trade winds (also called trades) are the prevailing pattern of easterly winds found in the tropics near the Earth's equator.

Triaxial test Getoechnical test on a sample, loaded on all sides to measure failure behaviour of the soil

ULS - Ultimate limit state Condition which a structure should be able to survive in the most extreme design condition; significant repairs are needed after this condition

Uprush The rush of water up onto the beach following the breaking of a wave.

Waist ratio (of a Dolos) Ratio between the length of a Dolos and the diameter of the central part (the waist). Dolosse may have different waist ratios (in contradiction to other concrete armour units)

Wave set-up Increase of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

Wave transmission The wave height at the lee side of a breakwater due to overtopping waves and-or transmission through the breakwater

Wind set-up The difference in still-water levels on the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water.

List of Symbols

Basic units:	kg, m, s, EUR		
Other units:	N = kg·m/s^2	Pa = N/m^2	
	J = kg·m	W = J/s	

Symbol	**Unit**	**Definition**
A	-	slope of a regression line y = Ax + B
A	m	half of the tidal amplitude
A	m	water depth above a sill
A_g	m^2	cross-sectional area
B	(y)	intercept of a regression line y = Ax + B
B	m	buoyancy of a block
B	m^2	storage area of an estuary or inlet
B	m	berm width, crest width
C	m$^{1/2}$/s	Chézy coefficient
d	m	rock diameter
d_n	m	nominal rock diameter, size of a concrete cube, nominal size of a armour unit (=$V^{1/3}$)
E	m^2	wave energy
F	N	Force
f	-	average frequency of an event per year
f	Hz	frequency of a wave or tidal component
G	-	Gumbel reduced variate
g	m/s^2	acceleration of gravity
H	m	energy head in front of structure
H	m	wave height (regular waves)
H_0	m	deep water wave height
$H_{2\%}$	m	wave height exceeded by 2% of the waves in a wave field
H_{m0}	m	wave height calculated from the zero-th moment of the spectrum (ie. from the total wave energy)
H_o	-	stability number, as used in berm breakwater calculations ($H_o = H_s/\Delta d_{n50}$), not to be confused with deep water wave height H_0
H_s	m	significant wave height
H_s'	m	a randomly selected significant wave height
H_{ss}	m	characteristic significant wave height during a storm (in PoT-analysis it is the maximum observed H_s during the storm
$H_{s,1/500}$	m	characteristic significant wave height during a 1/500 per year storm
H_t	m	threshold significant wave height in PoT-analysis
h	m	water depth
h'	m	water depth over a sill
h_s	m	water depth in front of structure
i	-	hydraulic gradient, slope of water level
K_r	-	refraction coefficient
K_r	-	reflection coefficient
K_T	-	transmission coefficient
k_s	m	bed roughness
k_t	-	layer thickness

Symbol	Unit	Definition
L	m	wave length
L	m	length of bed protection
L	m^3/s	sand loss
L_0	m	wave length on deep water
M	kg	mass of an element
m	-	discharge coefficient
m_c	m	metacentre height
m_0	m^2	first order moment of the wave spectrum; total wave energy
m_i	m^2	i-th moment of the wave spectrum
N	-	number of waves in a storm
N_s	-	number of events per year
N_{od}	-	number of displaced units
N_{or}	-	number of rocking units
N_{omov}	-	number of moving units
n_v	-	void ratio
P	-	cumulative probability of non-exceedance
P	-	notional permeability (in Van der Meer formula)
p	-	probability density
p	-	probability of occurrence of an event one or more times in a timeperiod T_L
Q	-	probability of exceedance
Q	m^3/s	discharge
Q_r	m^3/s	river discharge
Q_s	-	probability of exceedance of a given storm
R	-	strength parameter in probabilistic computations
R	m	hydraulic radius (for wide channels usually equal to the water depth)
Rec	m	recession length of a berm breakwater
R_c	m	crest level (above Still Water Level)
R_u	m	Run-up (vertically measured)
S	-	load parameter in probabilistic computations
S	-	damage number
s	-	wave steepness (for indices see the indices of T)
T_0	s	deep water wave period
T_L	years	design life time of a structure
T_{m0}	s	wave period calculated from the zero-th moment of the spectrum (so: not the period related to L_0)
$T_{m-1,0}$	s	wave period calculated from the first negative moment of the spectrum
T_r	years	return period of a storm
T	s	period of a wave
T_m	s	mean period of a wave
T_o	-	dimensionless wave period , used in berm breakwater calculations (); not to be confused with T_0 (deep water wave period)
T_p	s	peak period of a wave
t_s	hrs	storm duration
u	m/s	flow velocity
u_0	m/s	flow velocity not including contraction effects
V	m^3	volume of a block
W	-	Weibull reduced variate
W_s	-	Weibull reduced variate for storms
W_g	m	flow width in a closure gap

Symbol	**Unit**	**Definition**
α	-	parameter in the Weibull distribution
α	-	angle of the slope
β	m	parameter in the exponential, Gumbel and Weibull distribution
γ	m	parameter in the exponential, Gumbel and Weibull distribution
γ	-	safety coefficient
γ_f	-	roughness coefficient
Δ	-	relative density (= $\rho_s/\rho_w - 1$)
η	m	water level elevation
μ	-	contraction coefficient
μ	-	friction coefficient
ρ_b	kg/m^3	bulk density of material (including voids)
ρ_s	kg/m^3	density of stone or concrete
ρ_w	kg/m^3	density of water
σ	N/m^2	specific strength of material
ξ	-	Iribarren number (for indices see the indices of T)
Φ	-	dimensionless sediment transport, sand transport parameter
ϕ	-	phase angle
Ψ	-	Shields number, stability parameter
ω	Hz	frequency of the tide

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In this book several times is referred to Ph.D. and M.Sc. theses from Delft University of Technology. All these publications are in full text version easy downloadable form `http://library.tudelft.nl`. Select on the main screen "TU Delft Publications" and search via the name of the author.

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