

Breakwaters and closure dams

On the cover picture:

Breakwater at Trouville, Low Tide - Claude Monet, 1870

Oil on canvas, 54 x 66 cm (21 1/4 x 26 in); Szepmuveszeti Museum, Budapest

Monet was the unparalleled master of painting water. Here he has succeeded in reproducing the shallowness of the ocean at low tide. The dark rock, pier, and far bank of the water recede to a vanishing point located at the smallest sailboat in the distance. The strong perspective conveys the sensation that the water has flowed out in that direction. A pair of fishermen provide points of interest in the foreground that call attention to the shallowness of the remaining water. The small patch of flat water behind the man seated on a crate reveals the figures to be standing on a sand spit. The bare feet of the standing figure can almost be heard to squish in the wet sand. On this overcast day, there are no shadows. This allows the water reflections to stand out. The sails of the boats cast very flat and stationary reflections on the water surface that contribute to the perception of shallowness. The deeper water to the right provides a subtle reflection of the clouds that is easily overlooked at first glance. By painting low tide, of course, Monet suggests the eventual return of the water, which imparts a transience to the scene. Yet he accomplishes the portrayal of this moment in time with such artistry that the setting is transformed into something timeless and eternal. That is the central paradox of Monet's work: the transfiguration of an evanescent impression into an image of everlasting permanence.

Breakwaters and closure dams

2nd edition

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First edition 2001

Second edition 2008

Published by

VSSD

Leegwaterstraat 42, 2628 CA Delft, The Netherlands

tel. +31 15 27 82124, telefax +31 15 27 87585, e-mail: hlf@vssd.nl

internet: <http://www.vssd.nl/hlf>

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First edition: ISBN-978-90-407-2127-4

Second edition: ISBN 978-90-6562-173-3

NUR 956

Keywords: coastal engineering, breakwaters, closure dams

Preface

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 and closure dams 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.

At first sight it seems strange to combine in one book the design of two rather dedicated types of structures with distinctly different purposes, however from an educational point of view this is not so.

In both cases the design process requires that due attention should be paid to:

- the functional requirements
- the various limit states to which a structure will be exposed in relation to the requirements
- the various limit states that occur during construction phases
- the relation between these limit states and the occurrence of certain natural conditions

The differences between closure dams and breakwaters will enable us to focus attention on the above mentioned considerations.

In addition to this, there are also quite a number of similarities. In this respect, we refer to the construction materials, such as quarry stone, concrete blocks and caissons, which are widely used in both types of structures. The same applies to a wide range of construction equipment, both floating and rolling, and, last but not least, the interdependence between design and construction.

It is good to mention here that the design of closure dams, and more specifically closure dams in estuaries, has undergone a major development in the period between 1960 and 1985, when the Delta Project in the Netherlands was being executed. Only recently in Korea similar closures works have been executed. Also some experiences from these works are included in this book. In the view of the accelerated sea level rise it is anticipated that more works of this kind will be needed in future.

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.

This second edition has been updated by Henk Jan Verhagen. New additions to the book to be mentioned are the treatment of wave statistics, the spectral approach in the stability formula, the shallow water conditions and the Icelandic breakwaters. The book has brought in line with the Rock Manual (2007) and with the European Standard on Armour Stone (EN 13383).

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.

Henk Jan Verhagen, Kees d'Angremond
Delft, January 2008

Contents

PREFACE	v
CONTENTS	VII
1 INTRODUCTION	1
1.1 Scope	1
1.2 References	1
1.3 Miscellaneous	2
2 POSITIONING THE SUBJECT	3
2.1 General	3
2.2 Types of breakwaters	5
2.3 Types of closure dams	8
2.4 Historical breakwaters	11
2.5 Historical closures	14
3 THE DESIGN PROCESS	23
3.1 General	23
3.2 Abstraction level	24
3.3 Phases	25
3.4 Cyclic design	25
3.5 Consequences of systematic design	26
3.6 Probabilities	27
3.6.1 Basics of a probabilistic analysis and the use of safety coefficients	28
3.6.2 Additional problem in coastal engineering	29
3.6.3 Determination of a design storm	30

4	CONSIDERATIONS AT SYSTEM LEVEL	32
4.1	General	32
4.2	Functions of breakwaters and examples	32
4.2.1	Protection against waves	33
4.2.2	Guiding of currents	37
4.2.3	Protection against shoaling	38
4.2.4	Provision of dock or quay facilities	40
4.3	Side effects of breakwaters	41
4.3.1	Failure modes	41
4.3.2	Nautical characteristics	42
4.3.3	Morphology	43
4.4	Functions of closure dams and side effects	43
4.4.1	Closure of the rivers Rhine and Meuse	44
4.4.2	Side effects of the Enclosure Dike (Afsluitdijk)	47
4.5	Various dams and a few details	48
5	USE OF THEORY	50
5.1	General	50
5.2	Flow and hydrostatic stability	51
5.2.1	Flow through gaps	51
5.2.2	Modelling	56
5.2.3	Stability of floating objects	58
5.3	Waves	61
5.3.1	Irregular waves in deep water	61
5.3.2	Short-term statistics in deep water	62
5.3.3	Long-term statistics	67
5.3.4	Transformation of irregular waves in shallow water	69
5.3.5	Summary determination shallow water wave conditions	71
5.4	Geotechnics	72
5.4.1	Geotechnical data	72
5.4.2	Geotechnical stability	73
6	DATA COLLECTION	79
6.1	General	79
6.2	Meteorological data	80
6.3	Hydrographic data	80
6.3.1	Bathymetry	80
6.3.2	Tides	81
6.3.3	Storm surges	81
6.3.4	Waves	82
6.4	Geotechnical data	83

6.5	Construction materials, equipment, labour	85
6.5.1	Construction materials	85
6.5.2	Equipment	89
6.5.3	Labour	89
7	STABILITY OF RANDOMLY PLACED ROCK MOUNDS	90
7.1	Stability formula for rock	90
7.1.1	General	90
7.1.2	Iribarren	91
7.1.3	Modern stability formulae	94
7.1.4	Comparison of the Iribarren, Van der Meer and Hudson formulae	101
7.2	Concrete armour units	102
7.2.1	Historical overview	102
7.2.2	Stability calculations for concrete blocks	104
7.2.3	Single layer elements	106
7.3	Stability calculation	107
7.4	Special subjects	107
7.4.1	Shape of quarry stone	107
7.4.2	Grading of quarry stone	108
7.4.3	Stability of the toe	109
7.4.4	Breakwater head	110
7.4.5	Stability of crest and rear armour	111
7.4.6	Stability of low and submerged breakwaters	112
7.5	Near bed structures	113
8	BERM BREAKWATERS AND DYNAMIC STABILITY	115
8.1	Introduction	115
8.2	Seaward profiles	118
8.3	Longshore transport of stone	120
8.4	Crest, rear slope and head	121
8.5	The Icelandic breakwater	122
9	STABILITY OF MONOLITHIC BREAKWATERS	125
9.1	Introduction	125
9.2	Wave forces and their effects	126
9.2.1	Quasi static forces	126
9.2.2	Dynamic forces	127
9.2.3	A working compromise: the Goda formula	130
9.3	Influencing the forces	131
9.4	Failure mechanisms	133
9.5	Scour	134

9.6	Foundation	135
10	WAVE–STRUCTURE INTERACTION	137
10.1	Introduction	137
10.2	Reflection	138
10.3	Run-up	139
10.4	Overtopping for rubble mounds	143
10.5	Overtopping for vertical walls	146
10.6	Transmission by rubble mounds	147
10.7	Neural Networks	150
11	DESIGN PRACTICE OF BREAKWATER CROSS-SECTIONS	152
11.1	Introduction	152
11.2	Permeability/porosity and layer thickness	153
11.2.1	Permeability/porosity	153
11.2.2	Layer thickness and number of units	154
11.3	Berm breakwater	156
11.4	Traditional multi-layered breakwater	157
11.4.1	Classification	157
11.4.2	General design rules	158
11.4.3	Standard cross-sections	162
11.5	Monolithic breakwaters	165
12	DESIGN PRACTICE FOR CLOSURE DAMS	167
12.1	Basics of the storage area approach	167
12.2	The design methodology for closures	172
12.3	Stone closures	173
12.4	Caisson closures	176
12.5	Sand closure	180
12.5.1	Build up of a dam body	182
12.5.2	Sand losses	183
12.6	Cross-section of closure dams	186
12.7	Final remarks	188
13	CONSTRUCTION METHODS FOR GRANULAR MATERIAL	189
13.1	Introduction	189
13.2	Scour prevention by mattresses	190
13.3	Construction and use of mattresses	193
13.4	Construction of granular filters	194
13.5	Providing and handling of quarry stone	195
13.6	Use of rolling and floating equipment	197
13.6.1	Rolling equipment	199

13.6.2	Floating equipment	203
13.6.3	Combination of floating and rolling equipment	205
13.7	Very specific techniques and ancillary equipment	206
13.7.1	Closure by hydraulic filling with sand only	206
13.7.2	Use of a temporary bridge or a cable way	210
13.8	Minimizing risks during construction	211
13.9	Survey	214
14	CONSTRUCTION METHODS FOR MONOLITHIC STRUCTURES	218
14.1	Introduction	218
14.1.1	Caissons, closed or provided with sluice gates	218
14.2	Monolithic breakwaters	220
14.2.1	Monolithic breakwaters constructed by assembling small units	220
14.2.2	Monolithic breakwaters consisting of large units constructed in-situ	221
14.2.3	Prefabricated large units	221
14.3	Caissons	222
14.3.1	Building yard	222
14.3.2	Transport	222
14.3.3	Preparation of foundation and abutments	223
14.3.4	Floating stability during transport, positioning and ballasting	224
14.3.5	The sinking operation	226
14.3.6	Work-window of flow conditions during the sinking operation	228
14.3.7	Number of caissons and/or sluice gate caissons	230
15	FAILURE MODES AND OPTIMIZATION	231
15.1	Introduction	231
15.2	Failure mechanisms	232
15.3	Fault trees	235
15.4	Optimisation	240
15.4.1	Micro level	240
15.4.2	Macro level	240
16	FLOW DEVELOPMENT IN CLOSURE GAPS	242
16.1	Calculation of flow in a river channel	242
16.2	Calculation of flow in the entrance of a tidal basin	244
17	REVIEW	250
17.1	Breakwaters	250

17.1.1 Rubble or monolithic	250
17.1.2 Quarry stone or concrete armour units	251
17.1.3 Which design formula?	251
17.1.4 Service limit state	252
17.2 Closure dams	252
17.2.1 Decisive circumstances	252
APPENDICES	259
APPENDIX 1 EXAMPLE OF THE DETERMINATION OF A DESIGN STORM	261
A1.1 Statistics of individual observations	261
A1.2 The Peak over Threshold method	264
A1.3 What to do if only random data are available?	273
A1.4 Computation of the armour units	278
APPENDIX 2 QUARRY OPERATIONS	289
A2.1 Reconnaissance	289
A2.2 Blasting	294
A2.3 Operation of the quarry	298
APPENDIX 3 CONCRETE ARMOUR UNITS	299
APPENDIX 4 GODA'S PRINCIPLES FOR BREAKWATER DESIGN	307
1 Introduction	308
2 Historical development of upright breakwaters in Japan	308
3 Review of wave pressure formulae for vertical wall	313
4 Design formulae of wave pressures for upright breakwaters	316
5 Discussion of several design factors	324
6 Concluding remarks	326
References	326
APPENDIX 5 OPTIMUM BREAKWATER DESIGN	328
APPENDIX 6 CLOSING SEQUENCE IN CASE OF MULTIPLE CHANNELS	331
A6.1 Introduction	331
A6.2 Blocking the shallows first	332
A6.3 Blocking the main channel first	336
A6.4 Closure over the full dam length	342
APPENDIX 7 CONSTRUCTION EQUIPMENT	346
General	346
Land-based equipment – dumping of material	347

Land-based equipment – controlled placement	351
Waterborne equipment – dumping of bulk material	354
Waterborne equipment – controlled placement	356
Moving on impassable sites	360
REFERENCES	364
LIST OF SYMBOLS	368
INDEX	373

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SmartSoils	Geodelft, Netherlands Soft Rock Naue Fahsertechnik, Germany
Waverider	Datawell, Haarlem, Netherlands
Xbloc	Delta Marine Consultants, Netherlands

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 both breakwaters as well as closure dams. This means that in each step of the procedure attention is paid to both subjects and that every time the two types of structures are compared the similarities and differences are emphasized.

With respect to breakwaters, all existing 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.

With regard to closing dams, it is emphasized that only the actual closing dams are considered in this book. This means that only the closing operation itself is treated; the transformation of the closing dam into a permanent structure like an embankment is beyond the scope of this book.

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 comprehensive 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 and closure dams are mentioned here.

For *breakwaters* 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. For *closure dams* reference may be made to: The Closure of Tidal Basins (HUIS IN 'T VELD, STUIP, WALTHER, VAN WESTEN [1984]) and the Manuals of the Expertise Network Waterdefences (ENW, formerly TAW, in Dutch). Useful periodicals include the Journals of the ASCE, the journal "Coastal Engineering" (from Elsevier) as well as 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 available 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, mainly to provide shelter from wave action. This protection is primarily designed for vessels in port and for port facilities, but sometimes breakwaters are also used to protect valuable habitats that are threatened by the destructive forces of the sea or to protect beaches from erosion. Although the threat is usually caused by 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.

Closure dams are constructed for a variety of very different purposes, such as the creation of a separate tidal basin for power generation or as sea defence structures to increase safety.

Compared to closure works, few engineering works have such an extensive impact on the environment in all its aspects. For instance, the main purposes of the construction of the Afsluitdijk in the Netherlands, which changed part of the Zuiderzee into IJsselmeer, were to provide protection against high storm surge levels and to facilitate land reclamation. Additional advantages were fresh water conservation and the road connection (a railway was considered but never realized). The purpose of the closure may be one or more of such objectives, but these are automatically accompanied by other side effects. A thorough study of these impacts is part of the design process. A feasibility study that does not mention and estimate the negative aspects of the closing work is incomplete and valueless. For example, the negative effects for the Afsluitdijk include: the drastic change in tidal amplitude in the Waddenzee, with its consequential impact on the morphological equilibrium of the tidal flats and channel system, the social impact on life and employment in the

bordering cities, the influence on drainage and ground water table in the surrounding land areas, the changes to the fisheries, and changes to flora and fauna.

Several aspects are non-technical in nature and some, like 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, the engineer must identify the consequential effects to the best of his ability and present them in such a way that they are understood by decision-makers.

This book focuses on the technical aspects of the construction of a closure dam in a variety of circumstances. Every closure operation is a struggle with nature. Flowing water on an erodable bed has to be controlled. Every action taken to obstruct the flow will immediately be counteracted in some way or another by nature itself. Of course this happens within the laws of nature, many (but not all) of which are known. The knowledge gained from both good and bad experience is supplemented by the results of advanced research and experiment. Nevertheless, the changes in conditions during the progression of the closure are sometimes difficult to predict. Flexibility in operations that is incorporated in the design provides an important tool.

For a design to be made, the hydrology of the water body or watercourse to be closed has to be fully understood. The main distinction is made between tidal and riverine regimes. Tides are characterized by short-term variations in water level and in flow direction. The design must cater for quick action during high or, more typically, low water periods and during the daily occurring slack water periods. River flows are steadier in the short term, generally one-directional and never cease. Damming rivers is therefore a completely different process.

Comparison of the designs for breakwaters and closure dams shows some identical aspects but other aspects require a completely different approach. For instance:

Comparable:

- Many construction materials used are similar: bottom protection, quarry stone, concrete blocks, specially designed concrete structures (caissons).
- In both cases the equipment used is either land based or water-borne: for example hydraulic excavators and cranes, dump trucks and dump-vessels, barges and bulldozers.

Differences:

- The main determining parameter for breakwater design is wave-action, while for closure design it is flow velocity.
- The estimated design wave is unlikely to occur during the construction of a breakwater, but may occur in its lifetime. The estimated maximum flow during closure will occur during construction and will never occur after closure.

- The breakwater construction is the final design intended to withstand all future attack. The closure dam is a temporary construction that halts the flow, after which, for future safety, the desired definite dam profile can be made. This is based on construction in no-flow conditions.

2.2 Types of breakwaters

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

Mound types

Mound types of breakwaters are actually no more than 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) on one hand and size and the relative density of the elements (Δd) on the other hand. One extreme example is a gravel beach that is subject to changes in the equilibrium profile as the wave characteristics change and also to longshore transport phenomena. Another extreme is the ‘statically stable breakwater’, where the weight of the elements in the outer layer (armour) is sufficient to withstand the wave forces. Between these 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 $H/(\Delta d)$

Monolithic types

Monolithic types of breakwater have a cross-section designed in such a way that the structure acts as one solid block. In practice, one may think of a caisson, 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 type and the monolithic type of breakwater are caused by the interaction between the structure and the subsoil and by the behaviour at failure. The mound-type structures are more or less flexible; they can follow uneven settlement of the foundation layers, whereas the monolithic structures require a solid foundation that

can cope with high and often dynamic loads. The behaviour of the structures 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 disappear one after another. However, because of the slope the footprint of a rubble mound breakwater is much larger and therefore sometimes a vertical wall breakwater is the only option.

Composite types

Composite types of breakwaters combine a monolithic element with a berm composed of loose elements. In fact, there is an abundance of alternatives that combine a rigid element and a flexible structure.

Special (unconventional) types

Many methods can be used to break the 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 are used, or their use has been proposed, in exceptional cases under exceptional conditions. Under standard conditions their use usually appears to be either unfeasible or uneconomic. Floating, pneumatic and hydraulic breakwaters require either large dimensions or a lot of energy to damp longer waves that occur at sea. Usually they are only economic in case of relative small waves in very deep water (eg in the Italian lakes). Pile breakwaters and horizontal plate breakwaters require very high structural strength to survive wave loads under extreme conditions.

Apart from a distinction 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 a submerged crest. 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 breakwater are shown in Figure 2-1 to Figure 2-4.

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).

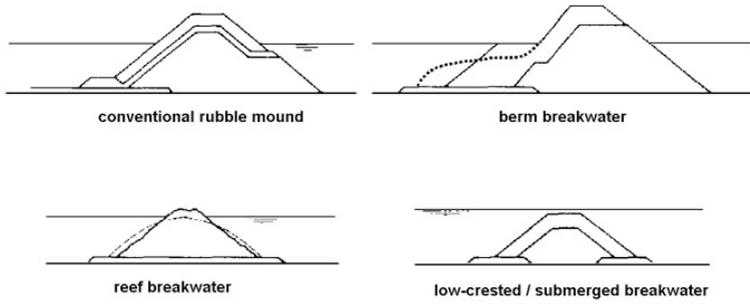


Figure 2-1 Mound breakwater types.

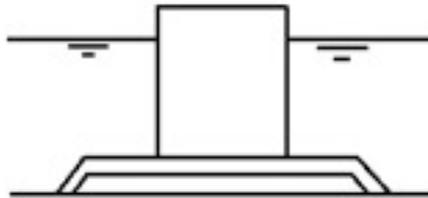


Figure 2-2 Monolithic breakwater type.

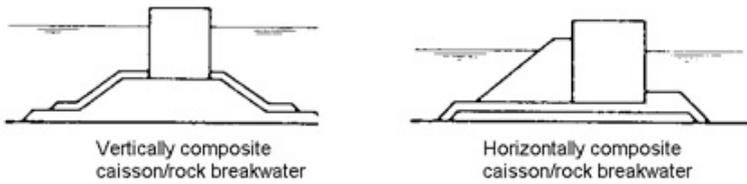


Figure 2-3 Composite breakwater types.

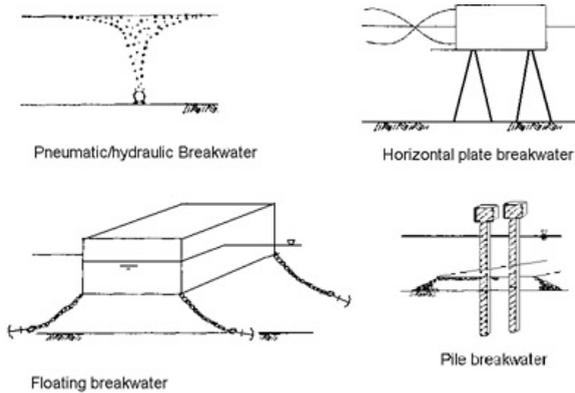


Figure 2-4 Special breakwater types.

In the present book, attention will be mainly focused on the traditional types of breakwater, i.e. the mound type and the monolithic type.

2.3 Types of closure dams

Several names have been adopted to distinguish various types of closure operations. The names used may refer to different aspects. However, the adoption of names has been random rather than systematic. Some names are typically Dutch and there may be no literal English translation.

A main distinction can be made according to the construction method. This is illustrated in Figure 2-5.

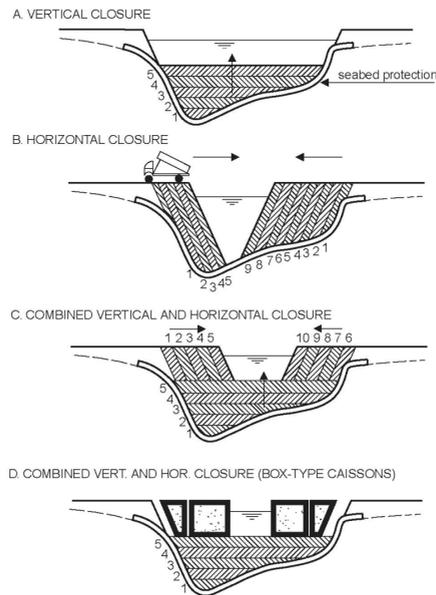


Figure 2-5 Basic methods of closure.

The construction method is related to the equipment used, which is either land-based or water-borne. This leads to a distinction between horizontal or vertical closure and the possible combination of these two methods. Using large structures (caissons) gives a type of horizontal closure with very large units. Figure 2-5 illustrates these methods.

There are two basic methods of closure:

- Gradual closure:

Relatively small sized, flow resistant material is progressively deposited in small quantities into the flow until complete blockage is attained. This can be used for either a vertical or a horizontal or a combined closure:

- Horizontal (gradual) closure: sideways narrowing of the closure gap.

- Vertical (gradual) closure: layer by layer upward closing of the gap.
- Combined vertical and horizontal closure: a sill is first constructed, on which sideways narrowing takes place.
- Sudden closure:
Blocking of the flow in a single operation by using pre-installed flap gates or sliding gates, or by the placing of a caisson or vessel.

Methods of closure may also be distinguished according to:

The *topography* of the gap to be closed, as is illustrated in Figure 2-6:

- Tidal gully closure [stroomgat-sluiting]: closure of a deeply scoured channel in which high flow-velocities may occur.
- Tidal-flat closure [maaiveld-sluiting]: closure across a shallow area that is generally dry at low water. This is characterized by critical flow at certain tide-levels.
- Reservoir dam (beyond the scope of this book): used in mountainous areas; this requires temporary diversion of the flow in order to obtain solid foundation in the riverbed at bedrock level.

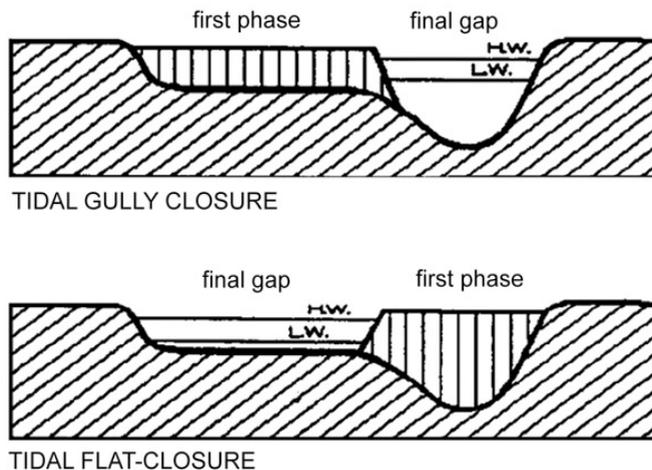


Figure 2-6 Closure named after topography

The hydrologic conditions that determine the type of closure (see Figure 2-7):

- Tidal-basin closure: characterized by regularly changing flow directions and still water in between; mainly determined by the tidal volumes and the storage capacity of the enclosed basin.
- Partial tidal closure: a closure in a system of watercourses, such that after closure there is still a variation in water-level at both sides of the closure dam.
- River closure (non-tidal): closure determined by upland discharge characteristics and backwater curves.

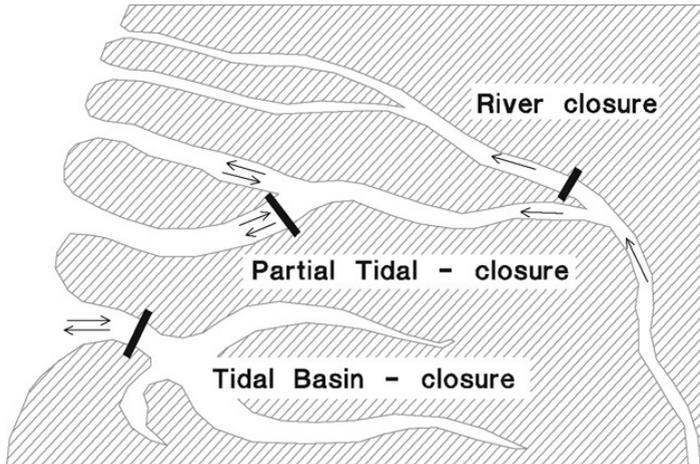


Figure 2-7 Closures named after hydrologic conditions

The *materials used*, which may vary according to the method of closure:

- Stacking-up willow mattresses [opzinken]: Closure realized by successively dropping mattresses (made of willow faggots, ballasted by clay or cobbles) onto each other.
- Sand closure: Closure realized by pumping sand at a very high rate of production.
- Clay or boulder-clay closure: Lumps of flow-resistant clay, worked up by grabs from floating cranes.
- Stone-dam closure: Closure realized by dumping rock, boulders or concrete blocks in the gap, either by using dump-barges and floating cranes, or by cableway.
- Caisson closure: Closure by using large concrete structures or vessels, floated into position and then sunken in the gap (possibly provided with sluice gates).

The *equipment used* (typically used for vertical closure):

- Bridge closure: Closure realized by dumping material from a bridge, pre-installed across the gap.
- Cableway closure: Dropping materials from a pre-installed cableway.
- Helicopter closure: Dropping materials from a helicopter.

Special circumstances leading to typical closure types:

- Emergency closure, is characterized by improvisation; the basic idea is that quick closure, even at the high risk of failure, prevents escalation of conditions; mainly used for closing dike breaches; needs strengthening later.

- Temporary closure: used to influence the conditions elsewhere, for instance by stepwise reducing the dimensions of the basin; needs to be sufficiently strong during the required period but easily removable afterward.

2.4 Historical breakwaters

The first breakwaters that are described in traceable sources date back to the ancient Egyptian, Phoenician, Greek and Roman cultures. Some 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 also constructed true monolithic breakwaters, since 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 2-8). 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])

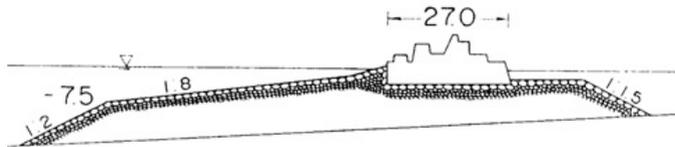


Figure 2-8 Rubble mound breakwater at Civitavecchia.

In modern times similar breakwaters were constructed at Cherbourg (1781/1789/1830), and at Plymouth (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 2-9 and Figure 2-10).

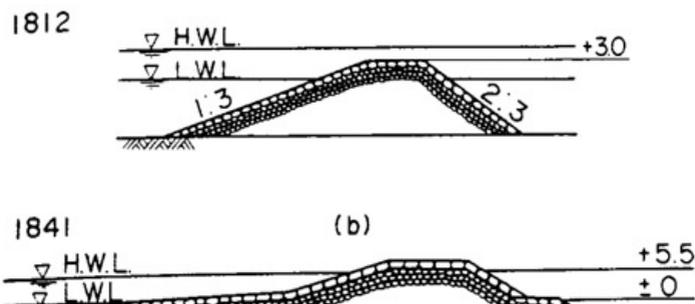


Figure 2-9 Breakwater at Plymouth.

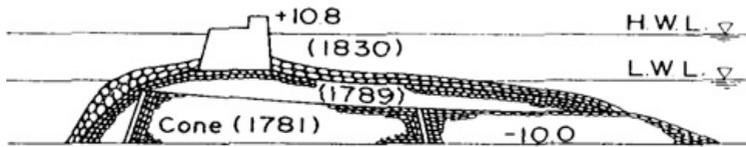


Figure 2-10 Breakwater at Cherbourg.

In view of the difficulties encountered in Cherbourg and Plymouth, in 1847 it was decided that a monolithic breakwater should be built at Dover. The construction posed a lot of problems, but the result was quite satisfactory since this breakwater has survived without major damage (Figure 2-11)

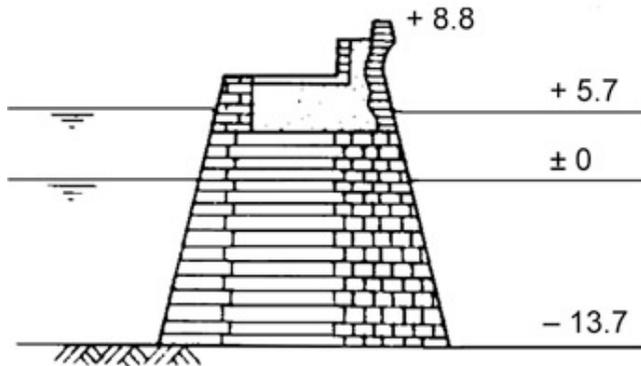


Figure 2-11 Monolithic breakwater at Dover.

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. The British engineers in particular took the lessons from the Dover breakwater to heart. To avoid the problems of construction in deep water, rubble mound berms were used for the foundation of a monolithic superstructure, 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 to too high a level.

In France, engineers tried to solve the 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 systematically in the core of the structure. The breakwater of Marseilles (1845) became a successful example for the French speaking world, just like the Dover breakwater in the English speaking world. 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 2-12).

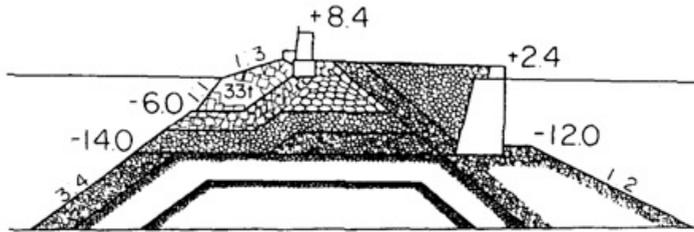


Figure 2-12 Breakwater at Marseilles.

These developments made the composite breakwater the most widely used type in the early 20th century, especially in Italy, where a lot of breakwaters were constructed in relatively deep water along the Mediterranean coast. The logical solution therefore seemed to be a composite structure consisting of a berm to about the half 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 2-13). However, these breakwaters were not a success, since the mound caused waves to break and to slam against the vertical wall, which subsequently tended to fail.

These failures worried the port engineers gathered in PIANC² so much that they decided to set up an international association for hydraulic research (IAHR). The failures of the vertical-wall breakwaters around the Mediterranean in the first half of the 20th century marked the end of this type of breakwater in W. Europe.

The French continued their efforts to optimize their rubble mound concept and to reduce the required weight of the armour blocks they developed the idea of interlocking them. Thus, in 1949, P. DANIEL [1953] of the Laboratoire Dauphinois d'Hydraulique (later Sogreah) designed the Tetrapod armour unit, which was the start of a long series of similar blocks. The Dolos (South Africa) seemed to provide the ultimate solution, until the limited mechanical strength of this block triggered a new series of mishaps. One of the most spectacular mishaps was the failure of the breakwater of Sines (Portugal) in February 1978. 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 was developed, the Core-Loc. In the Netherlands Delta Marine Consultants came with the Xbloc.

² Permanent International Association of Navigation Congresses.

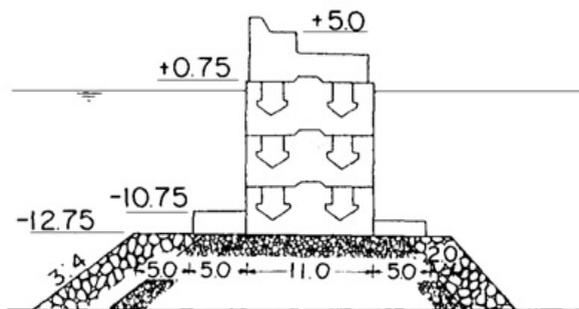


Figure 2-13 Typical breakwater along the Mediterranean coast.

In the meantime, the Japanese continued to build and develop the monolithic breakwater. There is no other country where so many monolithic and composite breakwaters have 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.

2.5 Historical closures

Closure dams have probably been constructed since mankind started agriculture and needed water for irrigation. Another reason for their construction could be political strategy because of the need for road or navigational connections. There is little evidence of these activities in ancient times, but the irrigation projects that once existed in ancient Babylon and Egypt suggest the presence of such works. As such dams would have been constructed from locally available perishable materials, no ruins (like pyramids) are found, even though they might have been quite extensive, considering that the builders were able to construct pyramids.

The damming Rhine and Meuse in the late Middle Ages

In the delta area of the rivers Rhine and Meuse, the damming of rivers and watercourses developed in the early Middle Ages. Because of the need for agricultural expansion, areas of marshland that are flooded only during extremely high tides or when rivers are in spate, were artificially drained. This caused the soil, mainly peat, to settle and as a result the incidence of flooding increased. Therefore, small earthen walls surround the areas and the natural drainage channels are dammed off. Many cities and villages in Holland are named after such dams (Rotterdam, Amsterdam). In the period 1100 to 1300, damming activities drastically changed even the courses of the two main rivers.

In order to prevent the river Rhine to be choked by sediments from overflowing its banks, the ruler of Utrecht dammed the river at Wijk bij Duurstede around the year

1200. The flow was diverted via the Lek river-branch and the original river mouth near Katwijk shoaled and disappeared.

In 1270 the river Meuse was diverted by damming it at Maasdam (near the city of Dordrecht) and upstream near Heusden, where the flow was directed towards the town of Woudrichem.

From the Middle Ages to 1920

Historic sources give a fair idea about the old methods used. The dams had to be constructed from locally available materials that could be handled by hand and simple equipment. These materials were not stable under conditions of high flow velocities. Therefore the essence of the process followed was to limit the flow velocities during closure process in accordance with limitations on the size and weight of these materials. One way to achieve this was to split the basin area into separate small compartments and then to close these compartments successively. Experience indicated the maximum area that could be taken in relation to tidal rise. Furthermore, flow velocities were kept low by using the vertical closing method, as will be clarified in Section 5.2. Branches cut from willow trees (osiers), were the main construction materials. With these an interwoven structure (fascine mattress [zinkstuk]) was made. When ballasted with clay this could be sunk onto the bottom. The closure was realized by sinking these mattresses successively one on top of the other on every tide during the short period of slack water. In this way a stack of mattresses created a sill in the closure gap. This continued up to about low water level. Further sinking was then impossible, as the mattresses could not be floated above the sill. The closure was completed by using a different type of structure. This was again composed of willow (osier) and clay, but this time built out from the sides of the gap and directly positioned on the sill.

The closure of the Sloe between the isles of Walcheren and Z-Beveland in the year 1871 is a good example of this procedure. The gap was 365 m wide at low water level and had a maximum water depth of 10 m. The tidal range was about 4 m. By sinking mattresses a sill was constructed up to the level of about low water. This sill had side slopes of 1 in 1 and a crest width of 18 m. The next stage was to construct an osier revetment [rijspakwerk] on top of the sill. In consequence of the added weight, the sill settled 1.80 m so in order to fabricate this wall up to the level of high water (at a height of 4 m above the original height of the sill), a 5.80 m high dam had to be made and that took a full month. Part of the final profile was made by adding a clay profile against the osier revetment.

In cases where the construction of an osier revetment failed, an attempt was made to position a vessel in the final gap and sink that onto the sill. This was not a simple operation, as transport was done by sailing or by rowing, and winching by hand was the only driving force. Timely ballasting and the prevention of the escalation of

pipings under and around the vessel were very critical. This method can be seen as the precursor of the caisson closure.

A historic example is found in the closure of the "Bottschlottertief" near Dagebuell (NW-Germany) in 1633. Clay had to be transported over a long distance by sailing vessels and it took 5500 labourers to execute the job. The closure was done by sinking a vessel in the gap. This was then ballasted and surrounded by clay, for the transport of which some 350 carts were used.

1920 until 1952

Gradually mechanization started to influence the work methods. The steam engine had already been in use for decades but the equipment was voluminous and heavy, both of which were troublesome in swift water and on soft ground. However, steam power could be used to drive winches, to drive sheet-piles and poles, to power the cranes used to transfer materials and for ship propulsion. Transport across the foreshore and newly constructed dam bodies was easier when locomotive engines, for which a stable railway had to be constructed, were used. Therefore initially the only change was the substitution of hard manual labour by engine work. However, better foundations for the transport roads and rails were needed since these were vulnerable to settlement in freshly worked ground and transport over water required greater water depth.

The difficulties encountered in building such closure dams are illustrated by the closure of the Hindenburgdam. This connection between the Isle of Sylt and the mainland of NW-Germany was completed between 1923 and 1927. The area was very shallow and sailing was impossible. The average tidal range was 1.70 m, but local wind effects much influenced the tides. The selected working method was to extend a wooden sheet-pile wall into the gap. The piling process was followed by the tipping of quarry stone on both sides to support the wall. The stone was transported on rails laid on a bridge that was constructed alongside the sheet-pile wall. Progress was much slower than anticipated and the erosion in front of the works consequently much more severe. The piling thus had to be done in highly turbulent water in a scour hole that preceded the sheet-pile construction and therefore more stone was needed for stabilization. On the inshore side, the railway was installed on made-ground, which often subsided, and derailments frequently occurred, thus escalating the problems. Later, the work method was adapted. The preceding scour was prevented by laying a 10 m wide stone protection on the bottom and the railway foundation was improved. Thus the problems were overcome.

Apart from the above-mentioned problems, a disadvantage of this type of steam driven equipment is that failure of the engine (damage) leads to major break-down of the complete works. The system is less flexible than one using manual labour.

Learning how to adapt the methods and use of the new equipment also stimulated the development of new methods. The engines could handle heavier units and reach higher production capacities. The advantages of this are:

- Heavier units:
 - can deal with higher flow velocities,
 - give reduced material losses.
- Higher production capacities:
 - give a shorter critical phase,
 - permit more progress in a still water period.
 - lead to shorter execution time, thus greater production during the workable periods and reduce the risk of incidental bad weather.

Owing to these new techniques larger projects and projects with more critical conditions became feasible.

For instance, in 1932 a very large closure was realized in the Netherlands when the former Zuiderzee was cut off from the sea by the Enclosure Dike. The 32-km long dam crossed two main gully systems. During the execution of the works large deposits of boulder-clay (a glacial till) were found. This material appeared to be very stable in the flow and could be handled by large cranes. A complete set of newly-designed floating cranes and transport barges was built and the closure was entirely constructed by these large floating units.

Another important change in the closure design was the development of mathematical modelling. Originally, designing had been a matter of experience and feeling, but calculations now started to replace the trial and error system. This reduced the risk of failure and was essential for the very large projects. For the damming of the enormous tidal basin, the Zuiderzee (now called IJsselmeer) in 1932, the differential equations for tide-propagation had to be solved. Professor Lorentz, a Nobel Prize winner in physics, achieved this. Three questions had to be answered before the job started:

- How would the tide change when the works were in progress, as this would affect the closing conditions?
- How would the tide change when the works were completed, as this would affect the design water level of the dike?
- What other design condition would affect the profile of the dike in this new equilibrium state of the sea (storm set-up and waves)?

Another challenge was presented in 1944, when for military reasons (World War II), the island of Walcheren was inundated by the bombing of the surrounding dike in four places.

This action dislodged the enemy troops and opened the fairway to Antwerp for the allied army fleet but at the same time it demolished the sea-defences and opened the low-lying island for tidal penetration. Restoring the sea-defences had to be done quickly in order that the island would not be permanently lost. Again, the mathematical basis for calculating tide-propagation improved. The four gaps, (three of these affecting one storage-basin), each with its own tidal amplitude and phase, and the propagation over inundated land with obstacles and ditches, and partial drying out at low tide, were a very complex system for a mathematical approach. And this was needed to establish the most favourable order of progress and also to ascertain risks that would arise if different path should occur in practice. Moreover, owing to the progressive erosion of gullies, the hydraulic resistance changed with time.

Immediately after the bombing the gaps in the dike were still relatively small. With the tide flowing in and out twice daily with ranges of 3.5 to 4 m, erosion deepened the gaps and a system of gullies was scoured out eating back into the inland area (Figure 2-14).

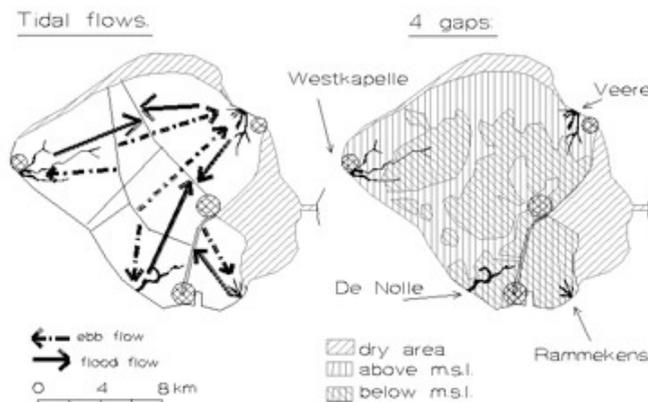


Figure 2-14 Walcheren - four gaps on one island.

There was no material or equipment available (wartime) and the areas were covered with mines. In June 1945, when at last a start could be made, closure of the gaps was nearly an impossible task. The traditional methods of closure failed because they progressed too slowly, or because the equipment and materials could not cope with the circumstances. The four gaps had to be closed simultaneously within a period of four months (before winter storms) and these closures were inter-related.

The only available suitable means to achieve these closures were the caissons of the Mulberry Harbour, used temporarily a year before during the invasion of the Allied Army in Normandy (France). After providing scour-protection in the gaps, a variety of large units, like pontoons, caissons, concrete and steel vessels, and even large

quantities of anti-torpedo-nets, were dropped or positioned in the gaps. The job was not finished before the winter and conditions worsened. Several times, initial success was followed by failure a few days later due to storm surges and piping. By the end of January 1946 however, the gaps were closed. A very good description of the difficulties encountered is given in the novel "Het verjaagde water" by A. den Doollaard.

Much experience was gained in the handling of caissons and vessels in closure gaps and ideas for the design of purpose-made caissons developed. The closure process could be improved by either creating a gap profile in accordance with the shape of the caisson or constructing a caisson to fit the requirements of the desired gap profile. In addition, the sinking could be controlled in a better way by regulating the water inlets by means of valves and separate chambers.

Different plans to improve the sea defences of the delta area of the Netherlands were drawn up and several closures were made. In 1950 the river mouth of the Brielse Maas was closed, using a purpose made caisson. In 1952 the Braakman, an estuary along the river Western Scheldt, was closed using two caissons, one of which was equipped with sluice gates. These temporary gates could be opened after the positioning of the caisson in the gap in order to reduce the water head in the basin after the closure and thus restrict the forces.

1953 and the Deltaworks

A new flood disaster occurred in the southern North Sea on the 1st of February 1953. A storm surge, together with spring tide-high water inundated 2000 km² of land in the Dutch Delta, creating 73 major dike-breaches and very many smaller ones. Again, all available technical experience and equipment and improvisation had to be used on many sites simultaneously, to close these gaps before the next winter season. Initially, not all the gaps had the same degree of difficulty or dimension. However, various gaps could not be dealt with immediately because of the disrupted infrastructure and as a result they scoured to tremendous dimensions. This is illustrated in Figure 2-15 for the Schelphoek breach on the Isle of Schouwen along the Eastern Scheldt.

This was one of the major dike breaches that occurred in the flood disaster on the 1st of February 1953. The scouring process continued during the actual closure works as well. The gap increased from an initial 40 m width to 525 m after 6 months, while the maximum depth increased from 10 m to over 35 m.

A typical example of successful quick improvisation is the closure of the gap at Ouderkerk on the IJssel. The storm surge at this spot reached a level of 3.75 m above MSL, overtopping the dike. The unprotected inner slope of the dike slid down over a length of about 40 m and the top layer of the dike slid and scoured away. However, the slope protection on the outer side remained intact up to the level of +1.70 m, as it

rested on a centuries-old clay-core. Six hours later, at tidal-low water (still reaching a level of +2.00 m), two small vessels were positioned on the outer slope, which broke the force of the falling water, although piping underneath was severe. Jute-bags filled with sand were carried in by hand and a small embankment was created on top of the remains of the dike. At the next high water (+2.80 m), the emergency provision remained intact and could be strengthened.

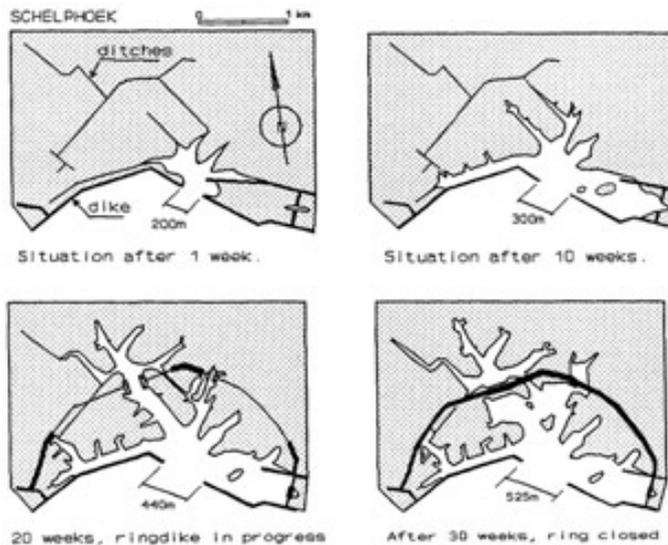


Figure 2-15 Development of erosion gullies.

The many very difficult circumstances led to various innovative actions, which resulted in repair within 10 months. Table 2-2 illustrates the enormous achievement: Once more, the experience was used in later developments of the closing technology. This is shown by the following example. The principle of a temporary closure made in 1953 near Kruiningen (Waarde) was copied on a much larger scale, in 1985 to close a major estuary in Bangladesh (Feni River). In this case 1,000,000 bags filled with clay, totalling about 20,000 m³ and stored in 12 stockpiles along the alignment, were carried by 12,000 Bangladeshi labourers into the 1000 m long gap to construct a dam in 5 hours.

The disastrous flooding in 1953, with all its negative aspects (1835 people drowned), had an offspring in the decision making process for the reconstruction of the sea defence in the Netherlands. In order to avoid the need to strengthen all existing dikes it was decided to shorten the lengths of the defence works by closing the estuaries. This was accomplished during the succeeding 25 years. Although many closures were beyond the scope of the current experience, it was considered possible to develop the required methods during that period, working from the small to the

large-scale projects. This period was therefore characterized by many experiments, a lot of research and the introduction of new materials and technology.

Date	no. of gaps closed	remaining gaps	inundated area (km ²)
2 February	3	70	2000
8 February	+ 8 = 11	62	2000
15 February	+ 6 = 17	56	2000
1 March	+20 = 37	36	1400
1 April	+17 = 54	19	800
1 may	+ 7 = 61	12	220
1 June	+ 4 = 65	8	150
1 July	+ 3 = 68	5	150
1 November	+ 4 = 72	1	100
December	+ 1 = 73	-	getting dry

Table 2-2 Closure scheme of gaps after the flood disaster of 1953.

Period after 1975

Around 1975 it was decided to adapt the plan to the changed views regarding ecological importance and the largest estuary (Eastern Scheldt) was provided with a storm-surge barrier, which took another 8 years to construct. Since parts of the closure dam had already been made and the creation of the new design and its execution were parallel, many problems arose in this period. A lot of new ideas had to be generated and tested. The much-improved computer and measuring facilities played important roles. As a result of all these efforts, the present day designer has many rules, formulas, graphs and test-results at his disposal.

The experience gained during the execution of the Deltaworks has been applied by other closing works in the rest of the world. Important closure works to mention are a number of estuaries closed in the period 1980-1985 in Bangladesh and a series of closures in Southern Korea. The closures in Korea (see table 2.3) are very significant with tidal ranges up to 10 meters and velocities in the closure gaps of more than 6 m/s.

Name of Estuary	Total length of closure dike	Tidal range (m)	Area (km ²)	Closing date
Saemanguem	29	7.00	400	April 2006
Hwaong	19	9.40	62	Mar 2002
Siwha	13	9.30	173	Jan 1994
Sukmun	11	9.42	37	Nov 1991
Busa	3	7.48	13	Mar 1988
Yongsan	4	5.59	109	Feb 1983
Sabkyo	3	10.4	28	Mar 1978

Table 2-3 Recent closures in Korea, from YOON [2003].



Figure 2-16 Flow in the Saemangeum closure gap just before closure.

3 THE DESIGN PROCESS

In the context of the subject “breakwaters and closure dams”, 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. For the closure dams discussion of the pros and cons of a closure, such as the environmental, social and other consequences, the location and function of the final dam, is beyond the scope of this book. 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 to. This implies that one has to design a construction which fulfils the functional requirements, but also that the construction will not fail, collapse or be damaged seriously 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.

General terms	Macro level	Meso level	Micro level
	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
Example 2 ^a	Regional water management plan	Fresh water basin	Closure dam for fresh water basin
Example 2 ^b	Fresh water basin	Closure dam (location, cross section)	Closing method
Example 2 ^c	Decision to construct the Delta project	Dam in Brouwers-havense Gat	Closing method north-gap
Example 2 ^d	Dam in Brouwers-havense Gat	Closing method north-gap	Design of caisson

Table 3-1 Examples of different scale levels.

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
- 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

Similar levels of abstraction can be distinguished for the design of a closure dam.

3.3 Phases

During the design process, one can also recognise certain 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 is:

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:

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 Schematisation of the design process.

Analysis:

Assembling of available data and arrange for the provision of missing data;
Drawing up a set of criteria that the design must fulfil (List of Requirements),
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, 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

The effects of the systematic design procedure on the purpose of the present book are
obvious. 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 kind 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

Only if these questions have been answered, can one start to think in more detail about 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.

Similar considerations apply to the design of a closure dam. Starting from the decision that a watercourse or dike gap has to be closed, the most suitable location or alignment is still to be selected. One must have insight into the hydraulic system of the flow, the subsoil conditions in the area and probably the infrastructure of the region (road connections), before one can start considering where and how the final dam should be made. For the closing process it may be even more important to realise at which abstraction level one is working, since the closure dam often is a structure with a temporary function. As soon as the watercourse has been closed, a new situation has been created. The final design for the scheme may involve a different step. For instance, the definite sea defence dam could be made in the lee of the temporary closure structure, enabling the construction elements of the closure dam to be used elsewhere. Consideration may also be given to splitting the actual closing operation into two or three compartments to keep the construction process and the construction materials within a workable scale.

Considering these remarks, one can conclude that a study book on the design and construction of breakwaters and closure dams 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 with 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 the construction 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, like loss of lives, 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 many 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 with a 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:

$$Z = R - S < 0,$$

where R is the strength and S is the load³. 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:

$$\text{strength: } R = A \cdot \sigma$$

$$\text{load: } S = M \cdot g$$

$$Z = R - S = A\sigma - Mg$$

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

$$A_{crit} = \frac{Mg}{\sigma}$$

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:

³ S as a symbol for load and not for strength does not seem 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.

$$A_{crit} = \gamma \frac{Mg}{\sigma}$$

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 Section 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, like protection of a museum or a religious site.

In the first case one can calculate the optimal “design storm”, in the second case this cannot be calculated but is subject to political decision making. For breakwaters it is usually a pure 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. And that may 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:

$$p = 1 - \exp(-f T_L)$$

in which:

p probability of occurrence of an event one or more than one 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 year 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%⁴. This must not be interpreted as a recommendation, but just as an example!

It means that the storm frequency becomes:

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

In case one accepts a probability of failure of 20% during a lifetime of 50 years, one should apply a $1/225$ ($= 4.4 \cdot 10^{-3}$) 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 $4.4 \cdot 10^{-3}$ 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.

⁴ The value of 20% is selected because this value is also used in the examples in various PIANC publications; from economical 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 and closure dams 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, when 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 (Section 4.2.1). This can be subdivided into firstly, protection of ports and shipping and secondly, shore protection.
- Guiding of currents (Section 4.2.2)
- Protection against shoaling (Section 4.2.3)

- Provision of dock or quay facilities (Section 4.2.4)

4.2.1 Protection against waves

Ports and shipping

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 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 at its most vulnerable when it is moored alongside a rigid structure such as a quay or 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.

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.

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 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].

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.

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 Service Limit States (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.



Figure 4-1 Harbour of Marseilles (France)

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 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 annum.

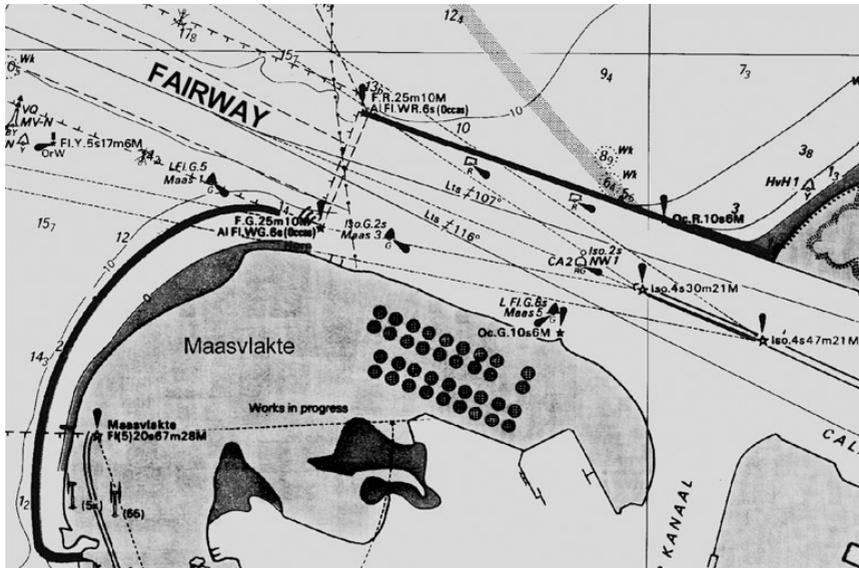


Figure 4-2 Breakwater at the Europoort entrance.

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 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]).



Figure 4-3 A system of detached breakwaters at Fiumicino, Italy.

4.2.2 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 crosscurrents 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 crosscurrent that is important but also the gradient in the crosscurrent, 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 (plan) 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.

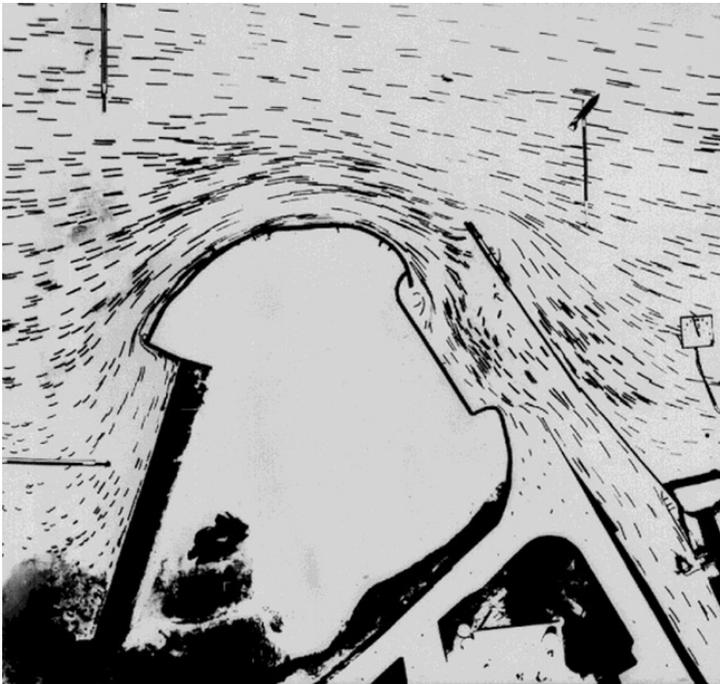


Figure 4-4 Flow pattern at the Europoort entrance.

4.2.3 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, 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).

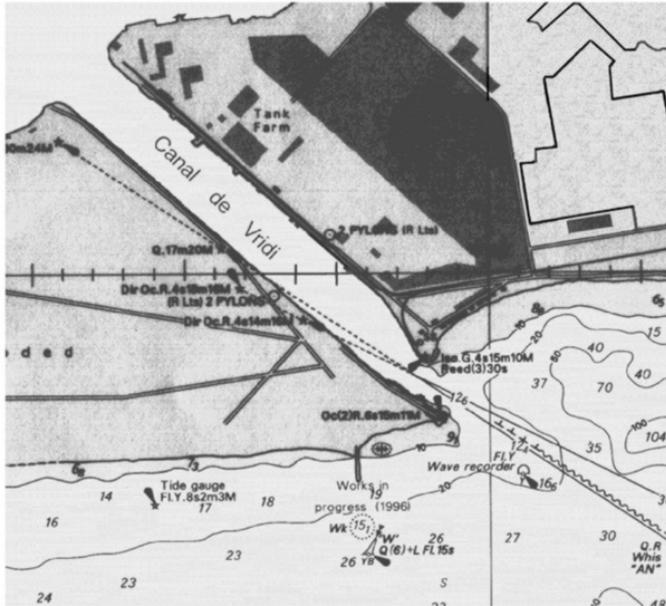


Figure 4-5 Entrance to the port of Abidjan.

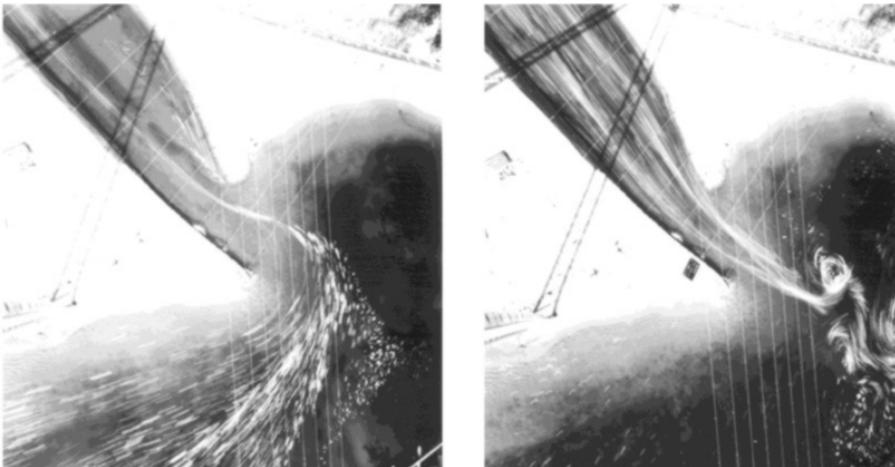


Figure 4-6 Flow pattern at the port of Abidjan.

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 its 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.

4.2.4 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.

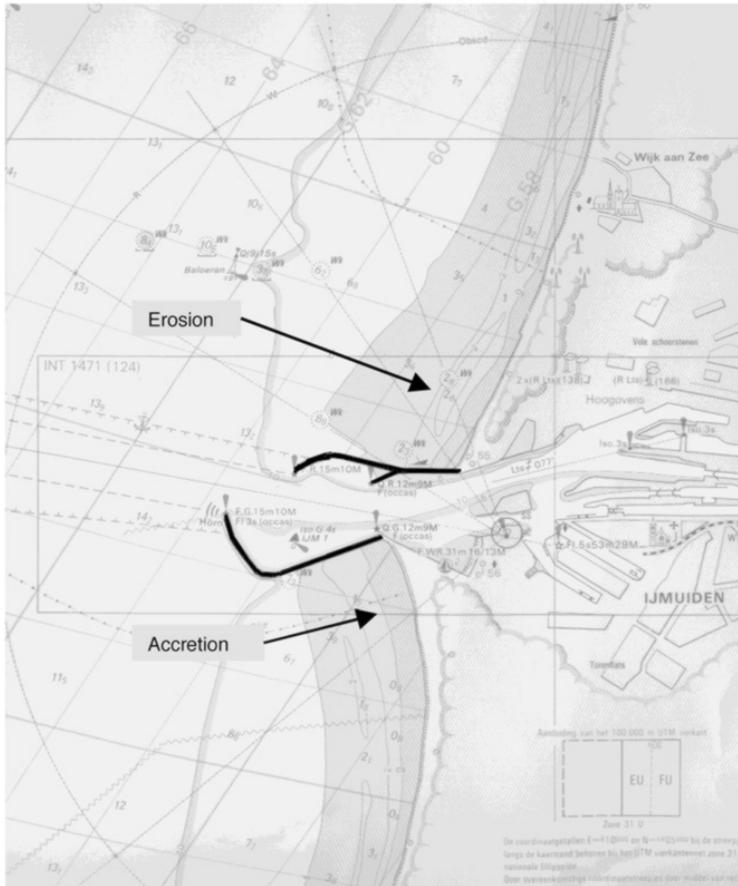


Figure 4-7 Port and breakwaters at IJmuiden.

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:

- In the 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.
- In the 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.

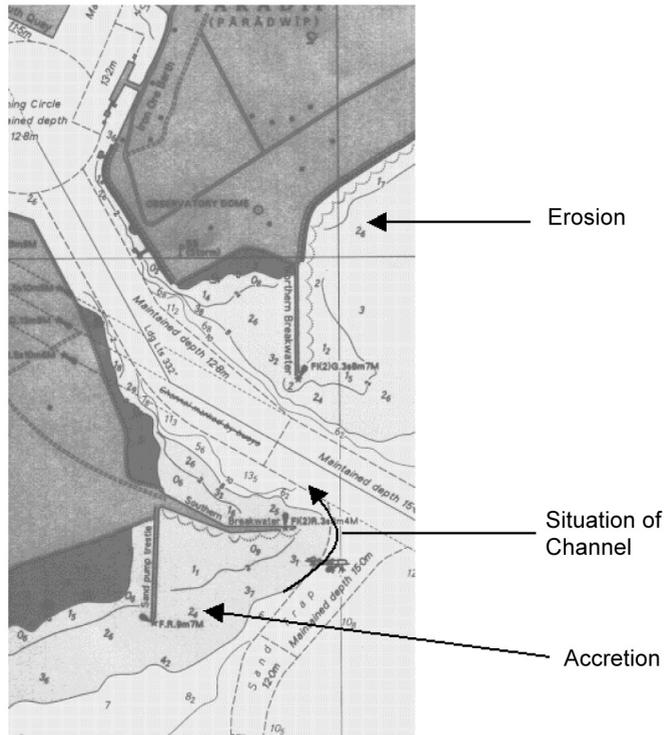


Figure 4-8 Siltation at entrance to port of Paradip.

- In the 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).

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.

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 close his eyes to 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

4.4 Functions of closure dams and side effects

A number of purposes and side effects are listed below. Sometimes it is difficult to determine why a specific effect is termed a side effect and in historic cases it has turned out that what were initially side effects became important aspects of the situation that was created.

Main purpose of closing a watercourse:

- land reclamation
- shortening the length of sea defence
- creating of fresh water reservoir
- creation of a tidal energy-basin
- creation of a fixed level harbour dock
- creating a construction dock
- providing a road or rail connection
- repair of a dike breach
- control of upland flow
- creating fish ponds
- cutting off river bends

Various possible side-effects (dependent on circumstances):

- change of tide (amplitude, flows) at the seaward side of the dam
- change in bar and gully topography, outside the dam
- disappearance of tides on the inner side of the dam

- change in groundwater level in adjoining areas
- alteration of drainage capacity for adjoining areas
- loss of fish and vegetation species
- loss of breeding and feeding areas for water birds
- rotting processes during change in vegetation and fauna
- stratification of water quality in stagnant reservoir
- accumulation of sediments in the reservoir
- impact on facilities for shipping
- impact on recreation and leisure pursuits
- change in professional occupation (fishery, navigation)
- social and cultural impacts

In the past, watercourses were mainly closed for the purposes of land reclamation and controlling the water levels on marshy land. In both cases this was linked to agricultural development. It is typical of these damming activities that the control of river and storm surge levels becomes essential. Follow-up action, like the repair of dike breaches and sometimes the cutting off of river bends has been necessary throughout the ages. The other purposes mentioned, like generation of tidal energy, harbour and construction docks, dams for road or rail connection and fish ponds are incidental works and have a smaller impact on the surroundings. Today, since the quality of life is becoming an important aspect for society, certainly in the industrially developed countries, damming activities are initiated to serve various other purposes. These include the creation of fresh water storage basins, the prevention of water pollution in designated areas, the provision of recreational facilities and the counteraction of salt intrusion or groundwater flow.

Depending on the circumstances, there will always be a number of side effects. These are sometimes temporary, but sometimes generate long-term developments that are difficult or impossible to predict with any degree of accuracy. The above list gives an indication of possible effects but does not pretend to be complete.

Below, a number of closures, some constructed centuries ago, are briefly described and comments are given on their purposes and side effects, in so far as these can be ascertained.

4.4.1 Closure of the rivers Rhine and Meuse

As mentioned in the historic review, the rivers Rhine and Meuse were dammed in the period 1200 to 1300. The Rhine debouched into the North Sea near Katwijk and, choked by sediment, regularly inundated the coastal area behind the dunes. First a closure dam was made on the borderline between the provinces of Holland and Utrecht, near Zwammerdam, resulting in inundations in Utrecht. Around the year 1200, after several years of conflict, the ruler of Utrecht dammed the river Rhine

further upstream at Wijk bij Duurstede. The purpose was clearly to protect the inland zone behind the dunes near Katwijk from inundation. The flow was diverted via the Lek river-branch. Of course, this dam had side effects. It excluded the downstream area from further silting up and the outer delta at the river mouth at Katwijk lost its sediment feeder. In the centuries that followed the coastline in the locality retreated by several kilometres and the Roman fortress "Brittenburg" disappeared into the North Sea.



Figure 4-9 The Rhine Meuse-delta before the year 1000.

The damming of the river Meuse (Maas) followed a different scheme. The town of Dordrecht had obtained staple rights (the right to store and sell certain goods) along the river Merwede. However, the payment of toll dues could be avoided by sailing along the River Meuse (see Figure 4-10, Oude Maas).

Probably because of this, the river Meuse was dammed in the year 1270. Although not problematic at first, in extreme conditions this distorted the discharge capacity of the delta and ultimately led to a major inundation after the dike breached in 1421 (St. Elizabeth's flood). This resulted in the permanent loss of the most developed agricultural area of Holland (the Grote Waard polder) by erosion of the topsoil. The region changed into a unique large tidal freshwater basin.

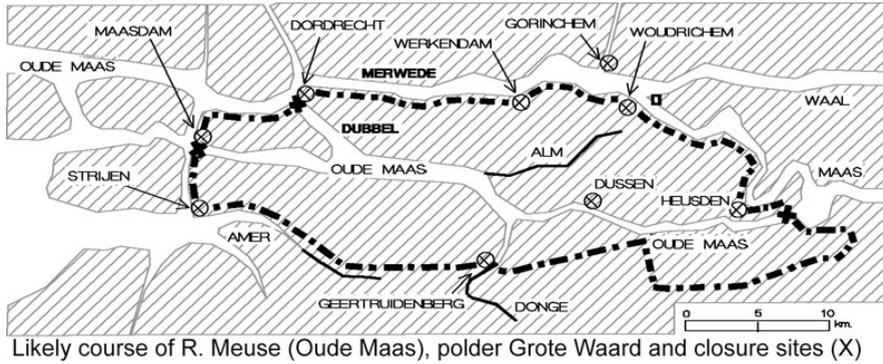


Figure 4-10 Situation after damming the River Meuse.

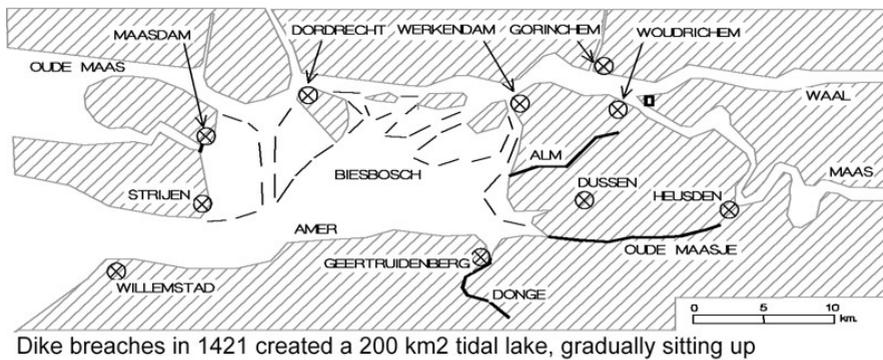
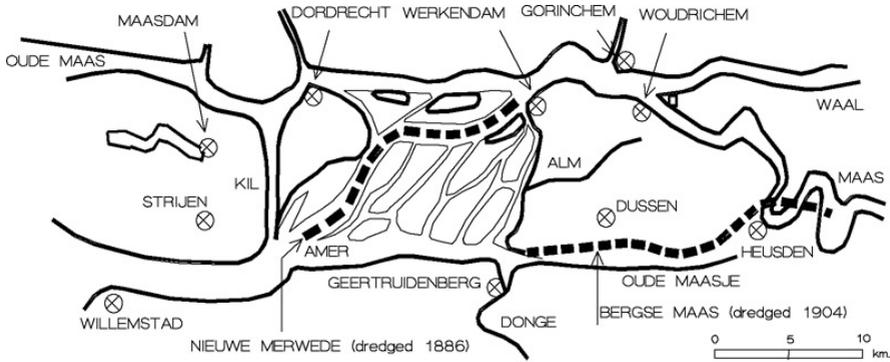


Figure 4-11 Situation after the St. Elizabeth's Flood.

In the period 1000 to 1400, very many areas were surrounded by embankments and drainage of these areas by rivers ceased. Whether or not the results of all these activities should be considered positive or negative is debatable. For nearly a thousand years all sediments carried down by the rivers were evacuated to the sea instead of regularly settling on the marshy land. The drainage lowered the water-table and this caused the peaty soil to settle. It changed the morphology of the landscape and its flora and fauna. Started as a simple water-level control system, it turned out to be a threat to the country. Gradually, the sea took large areas of the sinking ground. The side-effects, certainly when considered over very long periods, were tremendous. The people of today inherited a vast area below sea level that is continuously threatened by water and entirely dependent on its pumping capability for the evacuation of the water.

In some cases however, nature got an opportunity to show what could have happened otherwise. This natural restorative process is well demonstrated in the example of the lost "Grote Waard". The enormous lake created by the 1421-flooding, that is called the Biesbosch, formed a settlement basin and after 550 years this lake was nearly

completely silted up again and restored as a marshland. In order to prevent recurrence of the flooding, two main artificial rivers were dredged, the Nieuwe Merwede and the Bergse Maas, the latter restoring the historic discharge route of the river Meuse. Apparently the old scheme (at system level) could not be maintained.



500 years sedimentation refilled the basin and drainage-"rivers" were dredged

Figure 4-12 The Biesbosch area.

4.4.2 Side effects of the Enclosure Dike (Afsluitdijk)

As mentioned in Chapter 2, closing the Zuiderzee by the Enclosure Dike completely changed the tidal conditions on the seaward side, in the shallow sea called Waddenzee. The amplitude of the tide gradually increased to more than twice the former tide with the progress of the closure. This was studied before the works started. However there was a more difficult question to be answered:

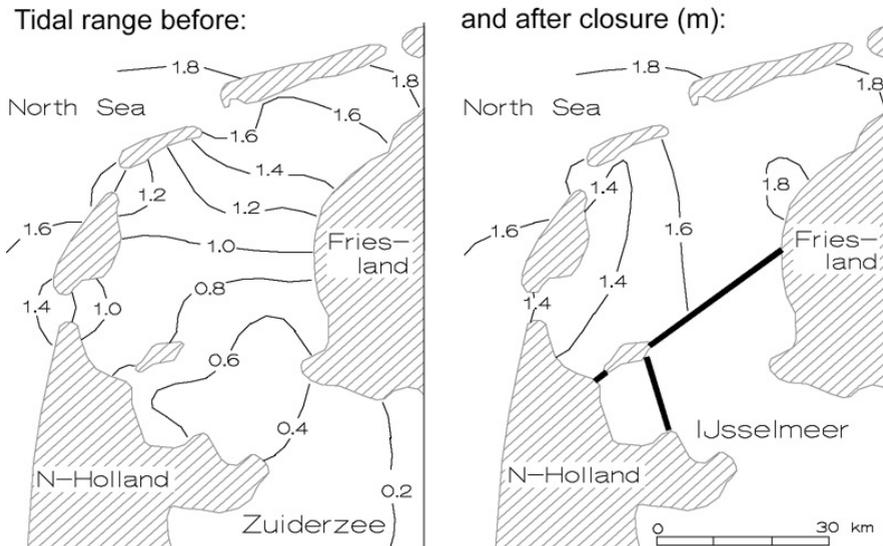


Figure 4-13 The Enclosure Dike and the tidal range.

How will the sea outside the dike adapt to the new conditions in the long run and change its topography and morphology?

By now, 60 years later, we know that this coastal water with its tidal flats and gully system is closely dependent on the exchange of water and sediment with the North Sea. Every change in the tidal volume passing between the islands separating the Waddenzee from the North Sea has a long-term effect on the balance of shoals and channels. Consequently, even the coastal balance in the North Sea on the outside of the islands must have been distorted.

4.5 Various dams and a few details

In this book various examples of closure works have been referred to. These are listed below in Table 4-3 with their name and/or the location, together with the year of closure. The list is not a complete list of closures executed in the past but is given because of its relevance to this book. Focus is on closures around the North Sea and in Korea.

Type	name or location	country or area	year	method or means
	Hindenburgdam	Sylt-Schleswig (Germany)	1925	Sheet-pile wall
	Dagebuell	German Bight (Schleswig)	1633	sunken vessel
	Meldorf, various gaps	Sylt-Schleswig (Germany)	1978	sand closure, sunken barges
	Lauwerszee	Waddenzee (NL)	1969	concrete caissons
	Zuiderzee	IJsselmeer (NL)	1932	boulder clay (crane pontoons)
E	4 Dike breaches Walcheren	Walcheren (NL)	1945	vessels and caissons
	Veerse-Gat dam	Walcheren (NL)	1961	caissons with gates
	Storm-surge-barrier	Eastern Scheldt (NL)	1986	gates between monoliths
E	Schelphoek, var. gaps	Schouwen (NL)	1953	caissons with gates
	Brouwersdam, 2 gaps	Schouwen-Goeree,(NL)	1972	caissons, blocks (cableway)
	Haringvliet-Sluices	Goeree-Voorne, (NL)	1971	concrete blocks (cableway)
	Brielse Gat	Brielse Maas (NL)	1950	caisson
	Braakman	Zeeuws-Vlaanderen (NL)	1952	sluice caisson
	Sloedam	Walcheren-Zd. Beverland (NL)	1871	sinking willow mattresses
E	Ouwerkerk	Duiveland (NL)	1953	caissons
	Grevelingendam, 2 gaps	Flakkee-Duiveland (NL)	1964	small caissons, quarry stone
E	Oudendoorn	Voorne-Putten (NL)	1953	caisson with side trap-doors
E	Kruiningen, var. gaps	Zd.-Beveland (NL)	1953	caissons; sandbags
	Krammer closure	St. Philipsland (NL)	1987	sand closure
E	Bath	Zd. Beveland (NL)	1953	ship
	Markiezaatskade	Bergen op zoom (NL)	1983	quarry stone, vertically
	Volkerakdam	Flakkee-N.-Brabant (NL)	1969	caissons with gates
E	Nieuwerkerk/IJssel	Hollandse IJssel (NL)	1953	small ship
E	Ouderkerk/IJssel	Hollandse IJssel (NL)	1953	sand bags and two vessels
E	Papendrecht	Alblasserwaard (NL)	1953	sand bags, quarry stone, clay
In other areas several major closure projects have been realized also, as for instance:				
	Feni	Bangladesh	1985	bags filled with clay
	Sabkyo	Korea	1978	
	Yongsan	Korea	1983	
	Busa	Korea	1988	
	Sukmun	Korea	1991	
	Sihwa	Korea	1994	
	Hwaong	Korea	2002	
	Saemangeum	Korea	2006	riprap and gabions

Table 4-3 Various dams.

5 USE OF THEORY

This chapter gives an overview of the theoretical knowledge needed for the design of breakwaters and closure dams. 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 or closure dams 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 this fields. Regarding flow the reader should be familiar with basics of open channel flow, the equation of Chézy:

$$u = C \sqrt{R i} \quad (5.1)$$

should be known and some basics regarding the flow over sills. This last point will be repeated in section 5.2, because in the calculations regarding the stability of closing elements this is essential basic knowledge.

Also basics of tides should be familiar to the reader. It is assumed that the origin of the tide, tidal constituents, spring and neap tide, diurnal and semi-diurnal tides, are all known to the reader. It may be useful to repeat that the travelling speed of a tidal

wave can be approximated by the celerity formula¹:

$$c = \sqrt{gd} \quad (5.2)$$

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. Finally, the wave enters estuaries and river-mouths. Shallows, funnel-shapes and upland discharges have impacts on the penetrating wave.

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 small 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 water, the basin is full and the inflow stops. This situation is reversed at low water. At the moment of mean level, the flows have their maximum. This is not true for propagating waves. 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.

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.

5.2 Flow and hydrostatic stability

5.2.1 Flow through gaps

During the progress of closing a watercourse a distinct constriction in the flow develops. This constriction can be vertical by the creation of a sill, or horizontal by the construction of dam-heads, or by a combination of both. Chezy's law for flow in

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

open channels no longer applies. Depending on the dimensions of the gap and the water depth, different flow patterns may occur. Consequently, different formulae for the calculation of the magnitude of the flow and the discharge capacity of the gap apply.

In the case of a horizontal constriction, the flow velocity can be approximated by the formula:

$$u = \sqrt{2g(H-h)} \quad (5.3)$$

in which g is gravity and $H - h$ stands for the difference in head over the gap. All potential energy is transferred into velocity and friction is ignored (conservation energy). When the water flows through the gap the flow lines contract and strong eddies develop behind the gap.

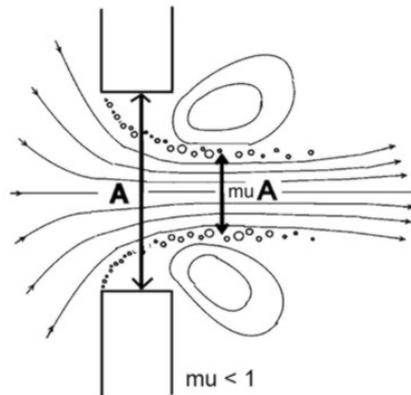


Figure 5-1 Flow pattern in a gap.

Where the flow is narrowest, the velocity reaches the above value, which is its maximum. Generally, this will be downstream of the gap. In the gap the average velocity is lower by a factor μ . This μ is the ratio between the cross-sectional flow areas of the gap (A) and the flow gorge.

The discharge capacity of the gap is:

$$Q = \mu A u = \mu A \sqrt{2g(H-h)} \quad (5.4)$$

If the dam heads have a rounded shape of considerable dimension, the flow lines follow the dam head contours and energy losses due to friction are rather small: μ approaches the value 1. An example of $\mu = 1$ is found for sand closures where the shape of the dam head is adapted by the flowing water itself. For closures made by tipping quarry-stone a steep, narrow dam head is formed. Then, the flow in the gap separates from the dam head. The profile of the gap does not contribute fully to the

effective discharge, so the value of μ becomes less than 1. Moreover, a lane of vortices develops in the gap in the shear layer between flowing and stationary water, resulting in high-energy losses. Instead of a " μ ", a discharge coefficient m is taken, which takes account of both effects. The value of m may be as low as 0.8 for these closures (see Figure 5-1 and Figure 5-3).



Figure 5-2 Flow through a closure gap.

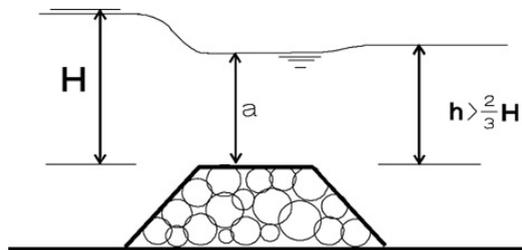


Figure 5-3 Flow in a gap with round dam heads.

For the closure operation, these aspects mean that the highest flow velocities develop downstream of the gap, above the protected bottom. The dam is attacked at the point where the flow separates and the shear layer starts developing. The most severe attack on the bottom occurs in the vortex lane along the wake. Since the highest velocity occurs downstream of the actual closing gap, in most cases it will be necessary to protect the original bottom against scouring. In extremely shallow gaps, it is possible that the bottom protection in the gap is sufficient to cause the flow pattern to change into super-critical flow when the head difference during closure

increases. In this case, the calculation is identical to that for a vertical closure (see below).

A vertical constriction gives a completely different flow pattern. Assuming a gap of infinite length, the flow pattern can be denoted two-dimensionally in a vertical section. The flow may be sub-critical flow or super-critical flow, depending on the relative levels of the water and the sill crest.

For sub-critical flow the discharge formula reads:

$$Q = mBh\sqrt{2g(H-h)} \quad (5.5)$$

in which H is the energy level of the upstream water and h is the water level at the downstream side of the gap, both measured with reference to the crest level of the sill and B is the flow-width (= gap length measured along the dam axis).

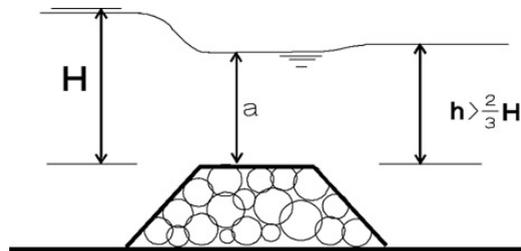


Figure 5-4 Flow over a sill.

In fact, the formula is derived by applying Bernoulli's law on the inflow-side, using the water depth a instead of h . However, a is unknown. Calculating with h decreases the head-loss ($H - a$) and increases the cross-profile ($B * a$). The resulting error that is introduced in this way is included in the discharge-coefficient, as determined by the shape and the roughness of the sill. Therefore it is called m instead of μ , and may reach values higher than 1. The magnitude of m goes from 1.3 for wide smooth sills with gentle slopes to 0.9 for sharp crested sills.

The mean flow velocity above the sill is:

$$u = \frac{Q}{B a} = m \frac{h}{a} \sqrt{2g(H-h)}. \quad (5.6)$$

If a closure gap with sub-critical flow is further constricted by progressively raising the sill, a situation will develop in which the flow becomes critical. This is the case when the vertical distance between sill level and downstream water level is $2/3$ of that at the upstream side. The discharge capacity is fully dependent on the upstream energy level in relation to the level of the sill. Then, $(H-a) = 1/3 H$.

$$Q = m B a \sqrt{2g \frac{1}{3} H} \quad (5.7)$$

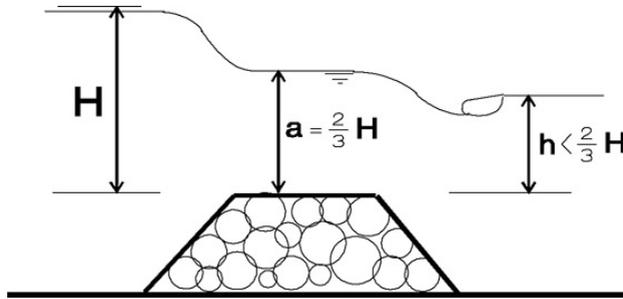


Figure 5-5 Critical flow over a sill.

If the downstream water level decreases further, the discharge capacity is no longer influenced and a remains constant viz. $2/3 H$. So,

$$Q = m B \left(\frac{2}{3} H\right) \sqrt{2g \left(\frac{1}{3} H\right)} \quad \text{and} \quad u = m \sqrt{2g \left(\frac{1}{3} H\right)} \quad (5.8)$$

The value of m depends on the inflow condition and the roughness of the sill, as the flow is independent of the downstream condition. Thus, since a is known and used in this case, the maximum value of m is 1. In the case of sharp crested rough sills, this value may go down to 0.8. The water falls down along the sill's downstream slope and changes into sub-critical flow via a hydraulic jump. Depending on conditions (roughness of the sill and Froude number) this may show different flow patterns.

For the closure operations, these downstream conditions may be more critical than the conditions on the crest of the sill. Suction forces will attack the downstream slope and endanger the stability of the sill. The down-pouring jet flow may reach the bottom in the case of a closure in relatively shallow water and endanger the bottom stability.

Finally, a combination of vertical and horizontal closure frequently occurs. Then, a situation may exist in which the gap has a vertical constriction caused by a sill and dam heads on both ends, which are progressively built out. This three-dimensional situation is more complex than the separate ones and difficult to describe mathematically. The general approach is to add the two effects as if they were occurring independently. However, the peak velocity downstream of the gap, usual for horizontal closure, may be compensated by the increase in water depth after the flow has passed the sill.

5.2.2 Modelling

To assess the hydraulic situation in the various stages of the closing process, extensive modelling has to be carried out. The modelling techniques that are available vary from physical scale models to electric analogous models and mathematical models. For many years, physical scale models were the best option. All closures in the Delta Project in the Netherlands have been accompanied by extensive physical model testing. Such models were built on different scales for different purposes. With the rapidly increasing capacity of digital computing techniques, mathematical models have replaced the physical scale models to a large extent. Sometimes, additional physical model tests are recommended to find the proper input values in a mathematical model (discharge coefficients, scour, etc.). Whatever model technique is used, one must be aware of the typical model effects when interpreting the results.

Any model investigation starts with an inventory of the boundary conditions and the estimation of the boundaries of the area that will not be affected by the closure. In tidal basins the latter is sometimes rather difficult to calculate, but when the effect is less than a few centimetres the result is sufficiently accurate. The resulting error will be less than the influence generated by the natural variation in tidal conditions.

External influences may be:

- Upland discharge(s) coming down the river(s)
- Tidal waves passing along or entering the basin to be closed
- Drainage water from surrounding areas
- Direct rainfall in the basin
- Wind set-up and draw down in the area outside the basin
- Wind set-up and draw down within the basin
- The earth's rotation (Coriolis acceleration)

The conditions, which determine the hydraulic behaviour within the basin, are:

- The network of gullies and shallows (if applicable)
- The flow profiles (depth/width relation) of the gullies
- The storage (level/surface area relation) of the shallows
- The hydraulic resistance of the gullies
- The existence of density (mainly salt/fresh water) currents
- The occurrence of tidal bores

The mathematics available to describe the hydraulic situation includes the basic formulae for conservation of mass and of moment.

$$\frac{\delta Q}{\delta x} + B_x \frac{\delta H}{\delta t} = 0 \quad (5.9)$$

$$\frac{\delta Q}{\delta t} + \frac{\delta(\alpha Q u)}{\delta x} + g A \frac{\delta H}{\delta x} + \frac{g |Q|Q}{C^2 A R} - W_x = 0 \quad (5.10)$$

These can best be solved by a numerical computation of unsteady flow in a network of watercourses (i.e. a one-dimensional flow model). The extent of the network, the number of storage areas, the schematisation of the flow profiles, the terms in the equations ignored (if any), and the correctness of the estimated hydraulic resistance determine the accuracy of the calculation. On the other hand, if the accuracy is not affected too much the mathematical system (and thus the quantity of deskwork) can be simplified. Besides, these simplifications are indispensable for a first orientation, and for the determination of an order of magnitude.

An extreme means of simplification is to ignore the conservation of momentum in the basin. What remains is the mass balance. For the gap the weir-formulae apply, while the water level in the basin is assumed to be horizontal over the full surface area at any time. To represent drying shallows, a variable area can be taken for different heights. The water level changes over time and this change, multiplied by the surface area of the water, must balance the quantity of water flowing through the gap during equal time lapses.

The tidal wave propagates at infinite speed within the basin and it neither reflects nor dampens. In addition, the simplification assumes that throughout the basin the water moves towards and away from the gap basin without driving forces and without friction.

The deviation is acceptable if the basin is relatively short in relation to the length of the tidal wave ($\leq 0.05 L$), and of course friction should be reasonably low, which in practical terms means that the area is well criss-crossed by gullies.

(Example: Suppose an estuary has a water depth of 10 m. The length of a (semi-diurnal) tidal wave in that case approx. 450 km. So $0.05L \approx 22.5$ km)

The quantity of water flowing through the gap per unit of time is determined by multiplying the flow velocity by the cross-sectional profile of the gap. The velocity changes as result of the changing head loss over the gap, while the cross-sectional profile changes as result of the changing water level in the gap. If a vertical closure is considered, it is necessary to ascertain whether the flow condition is critical or non-critical for every time step, as different formulae to calculate the flow velocity apply. More gaps than one can be dealt with by a simple summation of the water quantities, as long as it may be assumed that there is no head difference between any of the gaps within the basin. In this case, there is no propagating wave from one gap to the other.

One gap with variable depths can be schematised by the summation of a number of gaps, each having different dimensions (length and depth).

In vertical closures, various construction stages can be defined, each having a specified sill level. However, during the actual construction there will be intermediate stages in which only part of a new layer is present. Moreover, a failure may create a local depression in the sill level. In these depressions higher flow velocities will exist. Therefore, the calculation should always be executed with a gap of variable depth. **The plain horizontal sill is a theoretical case and does not represent a determining practical situation.**

When a dike breach is being closed, the storage basin may be an inundated area in which it is very difficult to define a system of gullies and shallows. Moreover, owing to erosion of the unstable ground, the system may quickly change. The Chezy value for this overland flow may be as low as 30. The area will fill during high water periods but will not drain fully during low water. In these calamitous situations, it is of the utmost importance to prevent the development of scouring gullies and to maintain the high resistance of the terrain (see Section 2.5, the gap called Schelphoek).

As an example, in Chapter 16 a calculation is made to illustrate the change in the tidal characteristics during the closure of a tidal basin.

5.2.3 Stability of floating objects

Any object of arbitrary shape submerged in an ideal fluid is exposed to pressures that act in a direction normal to the surface. If the total surface is divided into a large number of small surface elements, the forces acting on each element can be found by multiplying the area of the element with the external pressure. Integration of these elementary forces over the total surface of the body yields a resultant force. The force components in the horizontal plane nullify each other as can be proved mathematically. In the vertical direction, the resultant force proves equal to the weight of the fluid displaced by the submerged body (Archimedes). It is called the buoyancy and is directed upwards.

An object is floating when the buoyancy force is equal to the weight of the object. The body will position itself in such a way that the vector representing the weight of the body is equal but opposite to the vector that represents the buoyancy and works along the same line. The point of application of the weight is the centre of gravity G , the point of application of the buoyancy is the centre of buoyancy C . Within the condition "floating" two options can be distinguished:

- the object is completely submerged (example: submarine) and
- the object is partially submerged (example: surface vessel).

The position of the floating body can either be described as a stable equilibrium or as an unstable equilibrium. We refer to the term stable equilibrium if a small deviation from the equilibrium position will result in a return to that original position. We use the term unstable equilibrium if a small disturbance will not result in a return to the original position.

It is evident that a stable position exists when C is located vertically above G . Any small rotation of the object will result in a correcting turning moment (Figure 5-6). This condition can only be achieved when the mass density of the object is not distributed uniformly over its volume. One of the best examples is a submarine. Such vessel is only stable indeed if there is ballast at the keel or bottom.

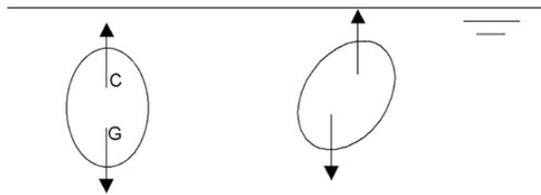


Figure 5-6 Stability of a submerged object.

A stable position can also exist, however, if C is located below G as is often the case when we consider objects with a uniform distribution of the mass density such as for instance a rectangular piece of wood (Figure 5-7). The centre of gravity G and the centre of buoyancy C can easily be found by construing the geometric centres of the complete body and the submerged part respectively.

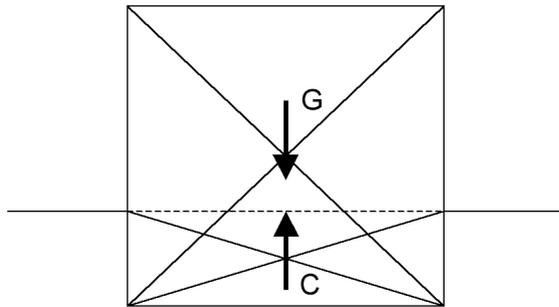


Figure 5-7 Stability of a floating object.

When we follow the definition of stable equilibrium, we should tilt the body slightly and see whether the turning moment by the shift of the gravity versus buoyancy vector is a stabilizing or a de-stabilizing factor. For this purpose, we first investigate the condition of a light weight box (Figure 5-8).

In the original (horizontal) position C is located in the centre line of the box, halfway the draft, and G is also located in the centre line, halfway the total height of the box (Figure 5-8, left). If the box is tilted slightly, rotation takes place around a point (or rather a line perpendicular to the page) at the intersection of the water line and the centre line of the box. This causes G to move slightly to the right. One would expect C to move slightly to the left, but this is not the case because the upward pressure against the bottom of the box is not longer uniformly distributed. Due to the trapezoidal shape of the pressure diagram, C moves to the right and in the example of a light weight box so much that the joint action of the buoyancy vector and the gravity vector yields a stabilizing moment. If the disturbance from the equilibrium position becomes too large, the box will loose stability and turn to its next stable equilibrium. This may be the case when a large external horizontal force is applied, for instance wind on the superstructure, a wave or flow force on the submerged part or pull by the wire of a tugboat.

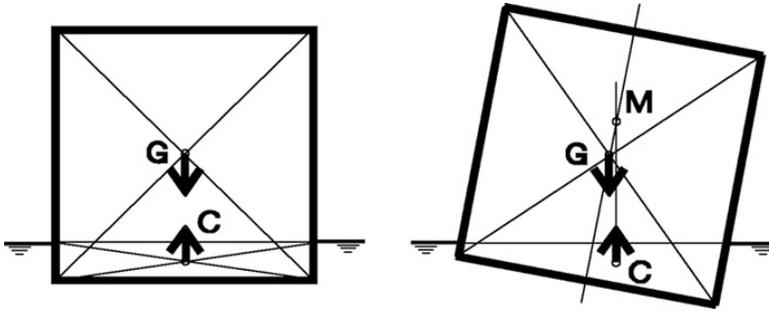


Figure 5-8 Stability of a light weight box.

The horizontal shift of the point of application of the buoyancy causes the working line of the buoyant force to intersect the centre line of the box in a point M , which is located above C . The distance between M and C (MC) can be calculated by rotating the body over an angle φ , and by subsequently calculating the magnitude and position of the force vectors.

$$\text{It appears that } MC = \frac{I}{V}, \quad (5.12)$$

in which I is the moment of inertia of the water plane about the axis of the body, and V is the displacement of the body.

This theory is often used to calculate the rolling stability of vessels. In that case, I and V can be calculated easily (see also Figure 5-9):

$$I = \int_{-\frac{1}{2}b}^{+\frac{1}{2}b} yx^2 dx, \text{ and } V = \frac{G}{\rho g} \quad (5.13)$$

For our box with width B and length L , I turns out to be $\frac{1}{12}LB^3$.

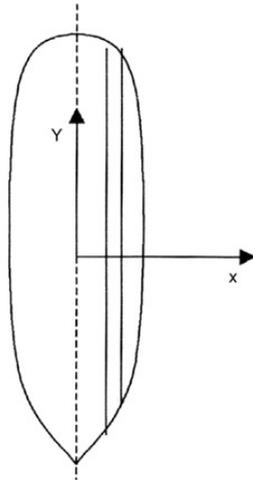


Figure 5-9 Moment of inertia.

It can now easily be understood that stability of rotation depends on the distance between M and G . This distance is called the metacentric height m_c . If M is located above G , the equilibrium is stable, if M is below G , it is unstable. In general, a value of 0.5 m for m_c gives sufficient stabilising moment for caissons used in hydraulic engineering.

If the mass density of the light weight box of Figure 5-9 increases, the following changes take place:

- G moves downward and C moves upward, which causes an increase in stability
- m_c decreases, (because I remains the same and V increases), which decreases stability.

This results initially in a smaller stability that will cause the box to float diagonally if the mass density is about half the density of water. A further increase of the weight of the box will turn it stable again, though the draft has increased considerably.

5.3 Waves

5.3.1 Irregular waves in deep water

Waves in nature are not small in amplitude and do not show the regular character with respect of 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 the 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 of Figure 5-10.

5.3.2 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)². The time span between these crossings is the wave period; the range between the highest and the lowest level is the wave height. 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.

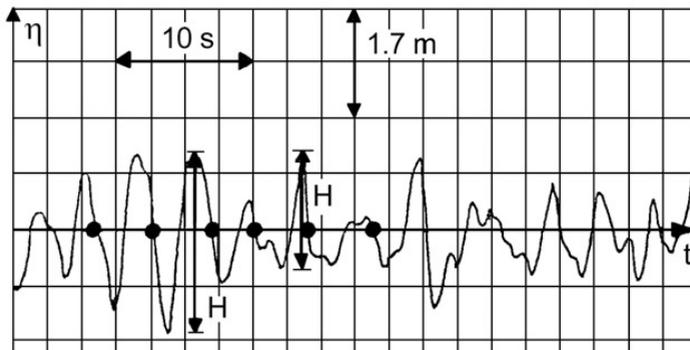


Figure 5-10 Irregular wave.

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

$$P(\underline{H} > H) = e^{\left[-2 \left(\frac{H}{H_s} \right)^2 \right]} \quad (5.34)$$

in which H_s , the significant wave height is equal to the average of the 1/3 highest waves. It can also be defined as the wave height that is exceeded by 13.5% of the waves. A third definition and method to determine H_s is given later.

² 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

A graphical presentation of the exceedance curve is often made on so-called Rayleigh graph paper (Figure 5-11). On such paper, a straight line through the origin represents a data set that follows the Rayleigh distribution. In view of the definitions, the value of H_s can be read at the 13.5% exceeding value. In this way, the strength of the storm considered is apparently determined by just one value: H_s . A stronger storm would lead to a steeper distribution curve, which is again defined by a specific value of the significant wave height.

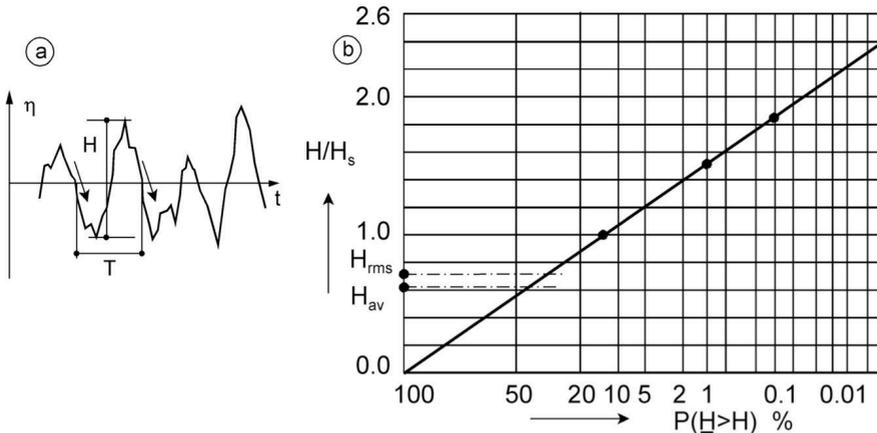


Figure 5-11 Rayleigh graph paper.

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:

$$\eta(t) = \sum a_i \cos(2\pi f_i t + \varphi_i) \quad (5.35)$$

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, see Figure 5-12. On the horizontal axis is the period, on the vertical axis the average H_s 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 or Nm/s.

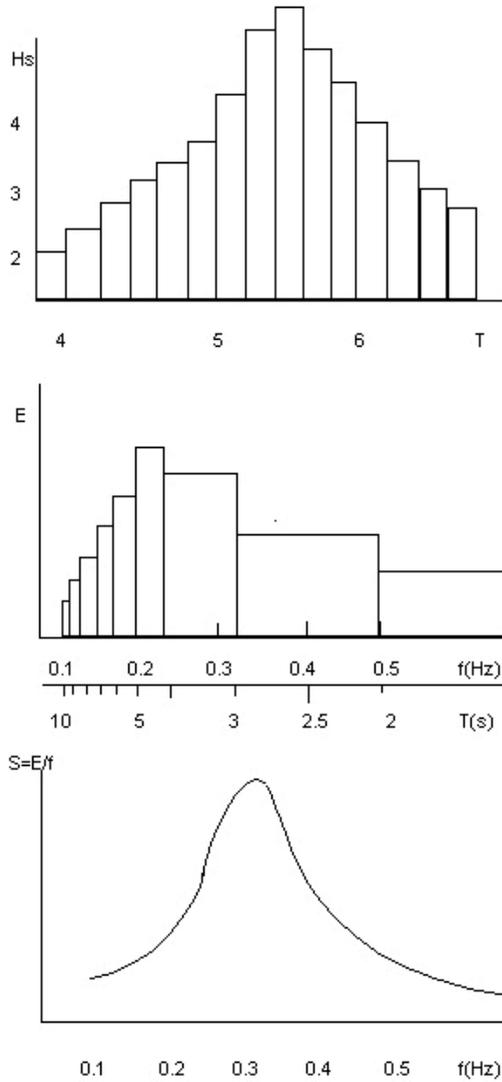


Figure 5-12 construction of a wave energy spectrum.

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

$$S(\omega) = \frac{1}{2} \sum a_i^2 / \Delta\omega \quad (5.36)$$

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:

$$\int_0^{\infty} S df = E_{tot} = m_0 \quad \text{or} \quad m_0 = \overline{\eta^2} = \frac{1}{2} \sum_{i=1}^n a_i^2 \quad (5.37)$$

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

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

$$m_n = \int_0^{\infty} f^n S(f) df \quad (5.38)$$

The first order moment is always defined as (arm x surface area), second order moment is (arm² x surface area). One can even define a (rather abstract) first order negative moment (arm⁻¹ x surface).

The determination of $S(\omega)$ 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

$$H_s = 4\sqrt{m_0} = H_{13.5\%} \quad (5.39)$$

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:

$$T_{m0} = \sqrt{\frac{m_0}{m_2}} \quad \text{or} \quad T_{m-1,0} = \sqrt{\frac{m_{-1}}{m_0}} \quad (5.40)$$

Usually T_{m0} 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 than 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 5-13.

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 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:

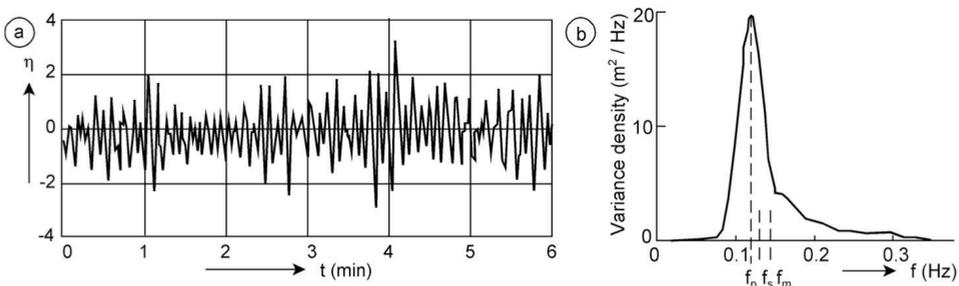


Figure 5-13 Typical wave spectrum, measured in the North Sea.

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	$\bar{H} = H_1$	$2\sqrt{\ln 2}$	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%	$H_{2\%}$		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.

5.3.3 Long-term statistics

It has been pointed out above 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, in Dutch *passaat*), 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 5-14).

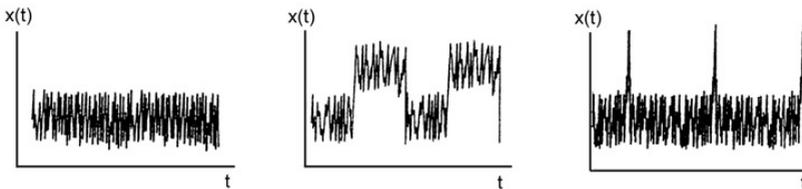


Figure 5-14 Typical types of wave statistics patterns.

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 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 so 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 is 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 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.

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 5-15.

One must realize that substituting T_p in the deep-water wavelength formula leads to a deep-water wavelength L_{op} . The fictitious wave steepness can then be expressed as $s_{op} = H_s/1.56 T_p^2$. Values of s_{op} are seldom 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 s_{op} 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.

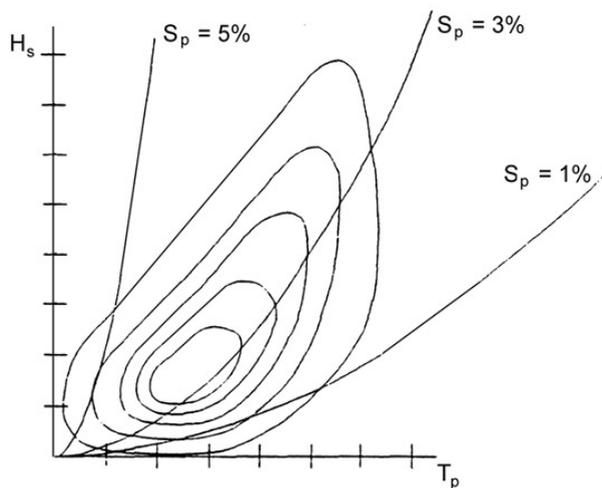


Figure 5-15 H/T diagram.

5.3.4 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. Owing to the variability of the wave period, there are no fixed orthogonals that can be calculated.

Modern wave models, like SWAN (see www.swan.tudelft.nl), are able to calculate wave growth and wave changes in near shore conditions. These models are spectral models, and they give the local wave spectrum (directional spectrum) at any desired location in the computational area. The method is based on considerations of energy flux through the boundaries of elements that cover the area of interest. 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.

Because of the variability in wave heights, one must expect that some waves will break owing to their extreme height, whereas other (lower) waves remain unaffected. This means that the Rayleigh distribution cannot be valid in shallow water. 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 a transitional value of the wave height (H_{tr}), the Rayleigh distribution remains valid. Above this value the exponent in the distribution function has a different value (≈ 3.6):

$$\Pr\{\underline{H} \leq H\} = \begin{cases} F_1(H) = 1 - \exp\left[-\left(\frac{H}{H_1}\right)^2\right] & H \leq H_{tr} \\ F_2(H) = 1 - \exp\left[-\left(\frac{H}{H_2}\right)^{3.6}\right] & H > H_{tr} \end{cases} \quad (5.41)$$

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 5-16a shows various wave heights as a function of H_{tr} , all made dimensionless with H_{rms} .

Figure 5-16a shows that the ratios between various wave heights and the H_{rms} change when the waves enter shallow water. For very shallow water, $H_{1\%} \approx 1.6H_{rms}$ instead of $2.15H_{rms}$ as would follow from equation (5.34). 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).

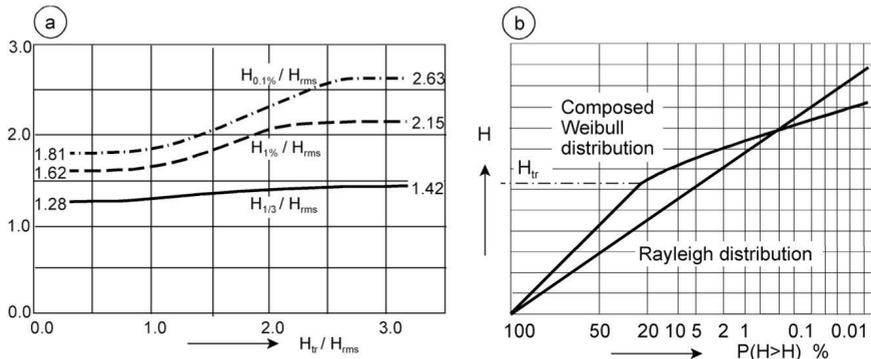


Figure 5-16 Wave height distribution in shallow water (Battjes/Groenendijk,2000).

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_{m0}

5.3.5 Summary determination shallow water wave conditions

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

- Determine the deep water wave condition (e.g. with one of the methods described in section 5.3.3. This means a wave height, a wave period and a spectrum shape is determined. Usually one finds 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_{m0} and $T_{m-1,0}$.
- Use the Battjes-Groenendijk method to determine the $H_{2\%}$.

5.4 Geotechnics

5.4.1 Geotechnical data

The construction of dams and breakwaters takes place along shores and river mouths with a great variety of subsoil conditions. Damming river branches and tidal basins generally takes place in alluvial deposits of sand, gravel, silt and clay, stratified in various compositions and of variable characteristics. 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, waves loads and differential water pressures. During the construction of dams and breakwaters, they are the sub-base for the vehicles and equipment driving across the site, while 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 and penetrometer-tests executed in the field and analysed in laboratories. 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 or by using the Dutch Cone, that 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. Far more accurate is triaxial testing of soil samples. 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. As described above, 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.

5.4.2 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:

- 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 slip is the surcharge on the subsoil by the own-weight of the embankment. 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 weight will be only 10 kN/m^3 . As a part of the dam may be submerged during high water, the weight gradually increases during falling water levels owing to loss of buoyancy. Such a dam, built out by tipping with dump trucks gets very steep slopes naturally, (gradients up to 1:3). Although extended without problems during the high water period, the dam may suddenly fail during low water.

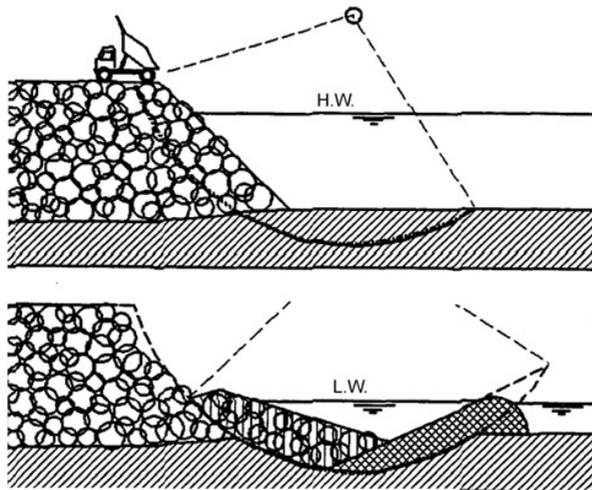


Figure 5-17 Dam profile after slide.

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 1 in 5 to 1 in 15 (depending on the grain size of the sand), and even flatter above water. The fill provides fully saturated material, which has a weight of about 20 kN/m^3 , and 10 kN/m^3 for the submerged part. In spite of the greater unit weight, the flatter slopes tend to provide a bit of extra security 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 that 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 his 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, the dam profile will be enlarged by further heightening and widening and finishing of the slopes. In many cases, with the knowledge that the soil will gain strength after some time, the construction of this 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 pressed away and that in later stages deformation will continue. For such a procedure the safety factor should be much lower than 1, as the embankment has to fail during construction in favourable conditions. Controlled failure is far more difficult to achieve than ascertained stability.

The subsoil encountered during damming activities (period 1970 to 1976) in the rear of the tidal basin called the Eastern Scheldt, consisted of large areas of peat, criss-crossed by former stream gullies of the River Scheldt that had been filled by clay and sand. Part of the Markiezaatsdam was hydraulically filled with sand up to a specified profile and several slips leading to the formation of heaps of peat along the side of the dam occurred. Another part was built of stone tipped from dump-trucks. This dam dropped almost instantaneously down into the soft material. It was then decided to replace the subsoil from the site of the final gap. Although the closure had been designed as a vertical closure, consolidation time between layers would have been too short. In a later design for the nearby Oesterdam in comparable conditions, hydraulic filling was again prescribed, but this time specifying that the profile should be given gentle slopes and a limited height in first stage. The filling required some extra equipment and the rehandling of sand, but no failures occurred. This proved that the consolidation method was a viable option, but that to ensure success it needed thorough planning and the definition of intermediate construction profiles.

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 that in 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 slip along a plane in basically undeformed bodies. For the calculation of instability by squeeze a different mathematical model, (e.g. Plaxis) has to be used.

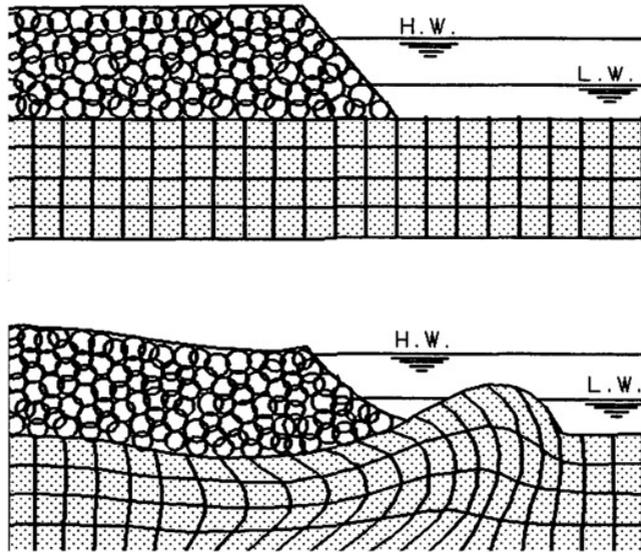


Figure 5-18 Squeeze.

The figures show that slip 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 rises up in front (or at the side) of the dam or breakwater.

Taking samples at the toe of the slope will show the facts. In practice, the difference between slip 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 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 conditions for the settling of the sand particles was ideal, 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. Then, 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.

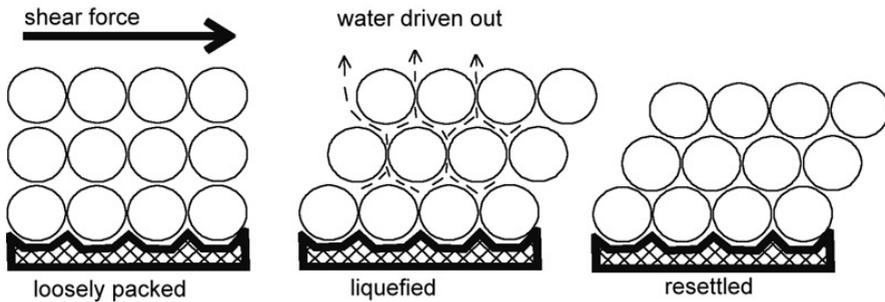


Figure 5-19 Liquefied sand.

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 are executed, give 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, over-stressed water is still present and the new situation will have 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 be only slightly speeded up. If it can possibly 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. Next, 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.

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 preparatory 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 come up with a surprising wealth of data. Calculation and calibration can then transform such data 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 role is less obvious. In this context we must mention 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 be taken into account where applicable.

Data are generally available from national meteorological offices; 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 dredged 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.

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

It is wise to pay attention to the type of echosounder used. When a high frequency (210 kHz) is used for the measuring beam, the depth indicated is the top of a soft

layer. When lower frequencies (30 kHz) are used, the beam penetrates into the soft mud layers. By using a dual frequency instrument, one can obtain an impression 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.getij.nl/>.

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.

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 recently (say 1970). The oldest observations of waves were carried out on board of ships that had a voluntary agreement with a meteorological office to carry out certain observations. Although the observations of waves were visual observations, their accuracy is acceptable since the officers were well trained and could compare the observed state of the sea 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>).

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 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 bottom level is a function not only of the wave height, but also of the ratio between wave length and water depth. One of the most popular 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 it possible to measure also wave directions. These buoys are always delievered with software, so standard output of these buoys is a directional wave spectrum each selected interval (e.g. every half hour). Because it is a spot measurement, the buoy gives the wave condition for this spot 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 that can be used to determine design wave heights is given in Appendix 1.

6.4 Geotechnical data

The geotechnical data required for a breakwater or closure dam certainly includes all data required to assess the bearing capacity of the subsoil, both during construction and later. 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 available permanently. In many cases, the works will be accompanied by substantial dredging, so that 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.

Macro instability			Macro-failure	Macro instability	Geotechnical information	
Slip failure	Liquefaction	Dynamic failure	Settlements	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	-	<i>In situ</i> 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).

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 little to estuaries and 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 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 such local observations should be combined with a geophysical survey. The geophysical survey uses electro-resistivity and electromagnetic 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.

	Geophysical methods				Penetration methods					Borings			
	Seismic	Electr. resist.	Electro-magnetic	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	-
Elasticity modulus	A	-	-	-	B	B	B	A/B	B	B	-	A	-
<i>In situ</i> 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

A: High applicability B: Moderate applicability C: Limited applicability

Table 6-2 In situ test methods and their perceived applicability (after Rock Manual 2007).

6.5 Construction materials, equipment, labour

6.5.1 Construction materials

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

Quarry stone

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

- 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 blocky). Some companies are specialised in trading dimension stone for use as armourstone. The quarries for the production of aggregates are, in general, not equipped to supply the size of stone required for large hydraulic engineering 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 size 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 no problem at all, 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 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_g .

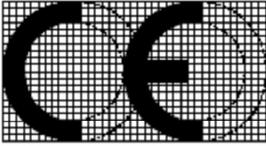
		
Any Co Ltd, PO Box 21, B-1050		
02		
EN 13383-1		
Aggregates obtained by processing natural, manufactured or recycled materials and mixtures of these materials for use as armourstone		
Particle shape	Category	(e.g. LT_A)
Particle size	Categories	(e.g. $CP_{63/190}$, $LM_{A5/40}$, $HM_{B300/1000}$)
Particle density	Declared value	(M_q/m^3)
Resistance to fragmentation/crushing	} Category	(e.g. CS_{90})
Resistance to breakage		
Resistance to attrition		
Release of dangerous substances	e.g. Substance X: $0.2 \mu m^3$	
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 6-1 Requirements for CE marking.

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 is exactly described what are the requirements for the sieve curve. 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. In Figure The category 80/180 mm is especially for the use in gabions.

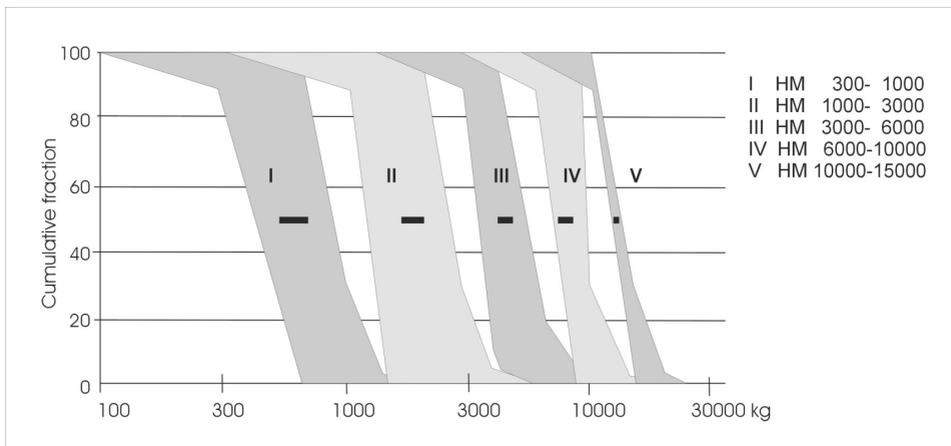


Figure 6-2 Weight distribution for Heavy Grading (EN13383).

In fact this means that 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 in Figure 6-2. When a class A is delivered, it also means that 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/m³. Other parameters on the label are the resistance to wear and resistance to breakage. For more details on all parameters 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.

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 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.

All the above leads to the main question of whether locally available equipment will be used or that the required heavy equipment is 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, 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 Chapter 7, attention will be paid to the stability of individual stones on a sloping surface under wave attack. 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 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.

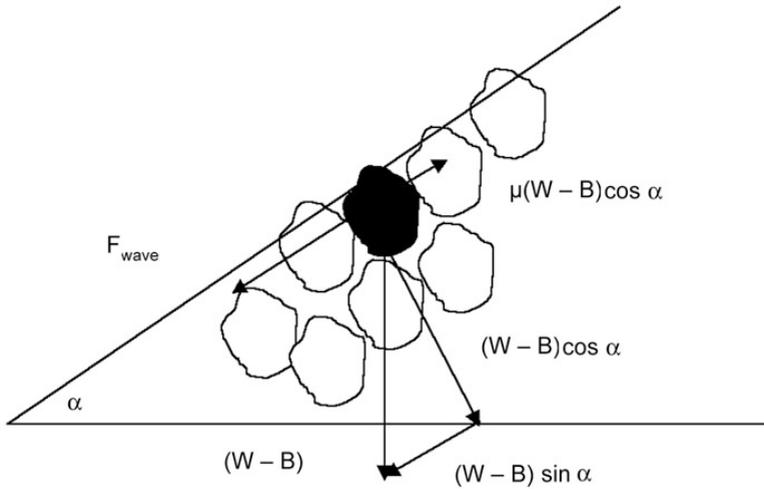


Figure 7-1 Equilibrium after Iribarren (downrush).

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:

$$F_{wave} = \rho_w g d_n^2 H \quad (7.1)$$

Note: For an element in permanent flow the force $F = C_v \mu^2 d_n^2$ and also the maximum velocity at the bed in a wave is proportionally to the wave height; compare also the formula of Isbash, developed in the same period (see SCHIERECK [2000], p51).

$$W - B = (\rho_r - \rho_w) g d_n^3 \quad (7.2)$$

and

$$W = \rho_s g d_n^3 \quad (7.3)$$

In case of uprush for stability the following condition is required:

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 uprush for stability the following condition is required:

$$\begin{aligned} F_{wave} &< N \{ \mu(W - B) \cos \alpha - (W - B) \sin \alpha \} \\ \rho_w g d_n^2 H &< N \{ (W - B)(\mu \cos \alpha - \sin \alpha) \} \\ \rho_w g d_n^2 H &< N \{ (\rho_s - \rho_w) g d_n (\mu \cos \alpha - \sin \alpha) \} \\ \frac{H}{\Delta d_n} &< N (\mu \cos \alpha - \sin \alpha) \end{aligned} \quad (7.4)$$

In case of downrush a similar equation can be derived

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

Nevertheless, considering the equilibrium for downrush along the slope, this leads to a requirement for the block weight:

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

For uprush, the formula changes into:

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

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.

Basically Iribarren concluded on a theoretical basis that the stability of rock is (amongst others) a function of the slope, described as $(\mu \cos \alpha - \sin \alpha)^3$. 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 Modern stability formulae

In this chapter the backgrounds of modern stability formulae are discussed. First a rather general formula will be presented, later some simplifications will be presented which leads to different formulae frequently used in literature. So 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 number is, the higher waves can be accommodated by the same stone size.

This number has to be a function of slope, as discussed before. An other important parameter is the Iribarren Number $\xi = \tan \alpha / \sqrt{s}$ where s is the wave steepness. Also the permeability of the structure is important (because much internal reflection inside the breakwater will result in a heavier load on the rocks. Furthermore one should use a period which gives more weight to the longer periods in the spectrum than 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 .

This leads to the following formulae:

$$\frac{H_{2\%}}{\Delta d_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} (s_{m-1,0})^{0.25} \sqrt{\cot \alpha} \quad \text{for plunging waves} \quad (7.8)$$

$$\frac{H_{2\%}}{\Delta d_{n50}} = c_s P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} (s_{m-1,0})^{-0.25} (\xi_{s-1,0})^{P-0.5} \quad \text{for surging waves} \quad (7.9)$$

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

$$\xi_{cr} = \left[\frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan \alpha} \right]^{\frac{1}{P+0.5}} \quad (7.10)$$

For $\xi_{s-1,0} < \xi_{cr}$ waves are plunging and equation (7.6) applies;

For $\xi_{s-1,0} \geq \xi_{cr}$ waves are surging and equation (7.7) applies.

In this 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. (5.40))

$s_{m-1,0}$ = fictitious wave steepness: $2\pi H_s / (gT_{m-1,0})$

$\xi_{s-1,0}$ = 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 $c_p = 8.4$ and that $c_s = 1.35$; this implies a ratio $c_p/c_s = 6.2$.

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, therefore it is called fictitious.

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

$$\begin{array}{lll}
 H_s = 2\text{ m} & H_{2\%} = 2.8\text{ m} & T_{m-1.0} = 6\text{ s} \\
 \cot \alpha = 3 & N = 3000 & \Delta = 1.65 \\
 P = 0.5 & d_{n50} = 0.6\text{ m (300-1000 kg)} & S = 2
 \end{array}$$

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

Wave steepness

When the increases, the stability of the construction also increases. With other words, when the Stability Number is higher, the same stone is stable in case of a higher wave. From the equations 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 fictive 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 height 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.

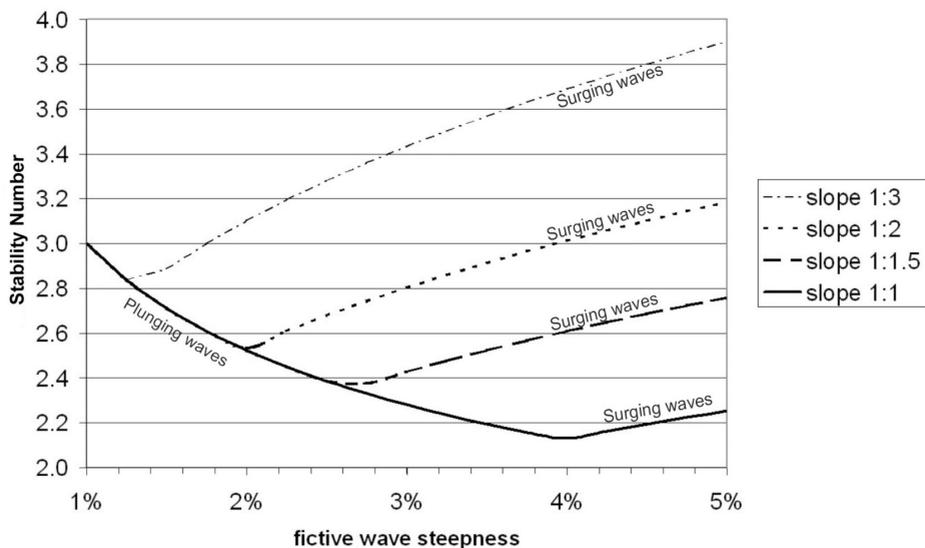


Figure 7-2 Example of the relation between Stability Number and fictive wave steepness.

Permeability

In case of a low permeability of the sub layer, the waves are reflected against this 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.

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 (7.6) and (7.7), one cannot easily transfer the expression of Van Gent into equations (7.6) and (7.7).

The advantage of the permeability expression of Van Gent is that it is a value to be computed from values, while the P 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.

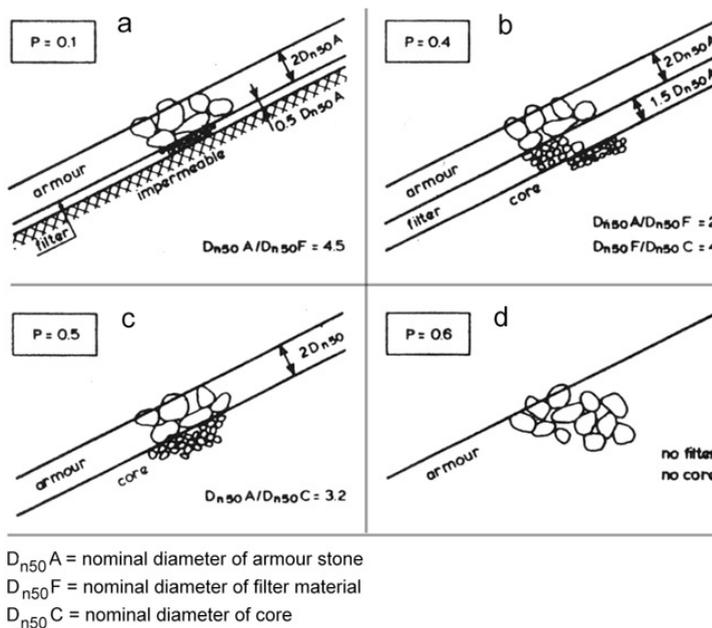


Figure 7-3 Permeability coefficients for various structures.

Damage level

The level of damage is expressed by the parameter:

$$S = \frac{A}{d_{n50}^2} \quad (7.11)$$

in which:

A = the erosion area in a cross-section in [m²]

$$d_{n50} = \left(\frac{W_{50}}{g\rho_s} \right)^{1/3} = \left(\frac{M_{50}}{\rho_s} \right)^{1/3} \quad [\text{m}]$$

W_{50} = mean weight of armour stones [N]

ρ_s = density of armour stone [kg/m³]

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.

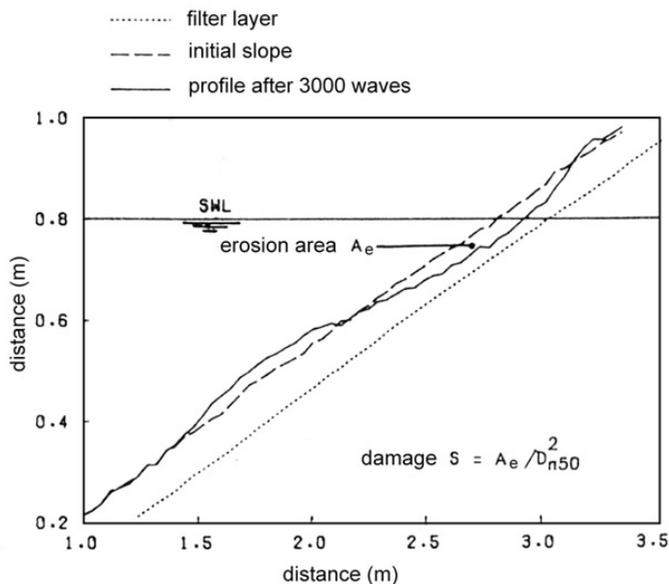


Figure 7-4 Damage(S) based on erosion area (A).

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 armour stone, 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.

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.

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:

$$N_{od} = G(1 - n_v)S \quad (7.12)$$

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 equations (7.6) and (7.7) 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 (7.6) and (7.7) can be reduced to:

$$\frac{H_s}{\Delta d_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5} \quad \text{for plunging waves} \quad (7.13)$$

$$\frac{H_s}{\Delta d_{n50}} = c_s P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^P \sqrt{\cot a} \quad \text{for surging waves} \quad (7.14)$$

In these equations one should use $c_{pl} = 6.2$ and $c_s = 1.0$. The transition between plunging and surging waves remains the same (eq. (7.8)). Note that now $\sqrt{\tan \alpha}$

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 (on deep water) and ξ_m is based on the mean period ($\xi_m = \tan \alpha / \sqrt{(2\pi H_s / g T_m^2)}$).

Note: When applying equations (7.6) and (7.7) with deep water data, the found values of the Stability Number are somewhat smaller than using equations (7.11) and (7.12). This difference has not yet completely be explained in a physical way. However it seems that the wave asymmetry also plays a role in the stability in shallow water conditions. But because the wave asymmetry is not described by a wave spectrum these effects have to be included in the “dust-bin” coefficients c_{pl} and c_s .

Equations (7.11) and (7.14) 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:

$$\frac{H_{2\%}}{\Delta d_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} s_{m-1,0}^{0.25} (\cot \alpha)^{0.5} \quad (7.15)$$

For slopes between 1:1 and 1:2 ($\cot \alpha)^{0.5} \approx 1.08 (\cot \alpha)^{1/3}$. Further 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 eq. (7.13) leads to:

$$\frac{H_{2\%}}{\Delta d_{n50}} = 1.25 (\cot \alpha)^{1/3} \quad (7.16)$$

This can be rewritten as

$$\frac{H_{2\%}}{\Delta d_{n50}} = \frac{(K_D \cot \alpha)^{1/3}}{1.27} \quad (7.17)$$

where $K_D = 4$. Because $H_{1/10} = 1.27 H_{2\%}$ one can write this as:

$$\frac{H_{1/10}}{\Delta d_{n50}} = (K_D \cot \alpha)^{1/3} \quad (7.18)$$

or:

$$W_{50} = \frac{\rho_s g H_{1/10}^3}{K_D \Delta^3 \cot \alpha} \quad (7.19)$$

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 was originally derived by HUDSON[1953,1959, and 1961] based on test with regular waves performed at the Waterways Experiment Station in Vicksburg, USA. In this formula the coefficient K_D obviously depends on many factors. For rock this factor is between 3 and 6, depending on the storm type, acceptable damage, etc.

For concrete armour units also this formula is used, 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.

7.1.4 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 in fact 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 minor difference between the formulae only. 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 .

The dashed lines indicates 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 1:1.5 and 1:3) there is not much difference between the formulae. For very gently slopes probably the approach of Iribarren is much better; but this part is not very interesting for breakwater design.

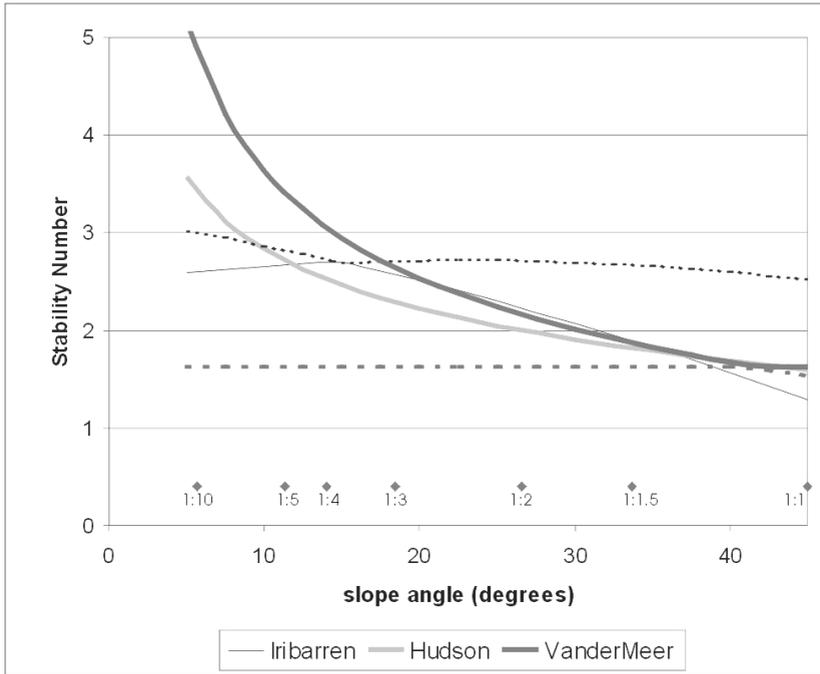


Figure 7-5 Variation of the Stability Number as function of the slope for the example.

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. Then it becomes interesting to use artificial blocks made of concrete. Also one may use special shapes with have also some interlock. One may distinguish three approaches:

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

The last group are pattern placed concrete blocks and columns, mainly used in block revetments. These blocks will not be discussed here, referred is to SCHIERECK, (2001).

Blocks mainly based on weight are the randomly placed cubes, or the grooved cube, also called Antifer cube. 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 gives a much mass per running meter, and this is therefore more economic. The most effective block shapes are the very slender blocks like Dolos. Because of breakage, their use is limited to smaller sizes. This reduces their applicability. It must be realized that special block shapes are costly because 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 quite popular. There was a period during which no self-respecting laboratory or consultant could do without his own armour unit. This started with the development of Tetrapods by Sogreah; it was 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 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 S. Africa attempted to maintain the basic shape of the Akmon, but to increase the permeability/porosity by making the legs more slender. This work led to (unpatented) Dolos. Initially, this was very promising, yielding K_D values of 20 and higher. At the same time, the sizes of vessels were increasing, requiring longer breakwaters, extending into deeper water with higher waves. This resulted in the design of some 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 appeared 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, and if tests had been carried out the results would have been of no use, because of scale effects.

In the meantime, it has been 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 development was to avoid the use of slender units and to rely upon simple cubes. Although the required weight 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 has resulted in the Core-Loc.

More recently, 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. As an alternative to the Accropode at this moment also the Xbloc of Delta Marine Consultants is available.

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. This makes using the Iribarren number ξ in a formula less realistic, since this expresses the influence of both, wavelength or period and slope. All formulae for concrete units, except the Accropode are based on a slope of 1:1.5. (Accropodes are placed on a slope 1:1.33)

Since the mechanical strength of the concrete blocks may play a role, it is useful to distinguish damage due to actually 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 N_{or}).

The total number of moving units 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 compatible with the value of S , (compare equation (7.10)). S is about double the value of N_{od} .

Van der Meer [1988] gives the stability for various frequently used blocks. He makes a distinction between displaced blocks and moving blocks. The difference appears to be a reduction of the N 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 $d_n = 0.65 d$ if d is the height of the unit

Dolos $d_n = 0.54 d$ if d is the height of the unit (waist ratio 0.32)

Accropode $d_n = 0.7 d$ if d is the height of the unit

Xbloc $d_n = 1/\sqrt[3]{3} d$ 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			>2

Table 7-3 Classification of damage levels N_{od} and N_{omov} for concrete multi-layer elements (slope 1:1.5).

Cubes

$$\frac{H_s}{\Delta d_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0 \right) s_{om}^{-0.1} \quad \frac{H_s}{\Delta d_n} = \left(6.7 \frac{N_{omov}^{0.4}}{N^{0.3}} + 1.0 \right) s_{om}^{-0.1} - 0.5 \quad (7.20)$$

Tetrapods

$$\frac{H_s}{\Delta d_n} = \left(3.75 \frac{N_{od}^{0.5}}{N^{0.25}} + 0.85 \right) s_{om}^{-0.2} \quad \frac{H_s}{\Delta d_n} = \left(3.75 \frac{N_{omov}^{0.5}}{N^{0.25}} + 0.85 \right) s_{om}^{-0.2} - 0.5 \quad (7.21)$$

Dolos

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

$$N_{od} = 6250 \left[\frac{H_s}{\Delta^{0.74} d_n} \right]^{5.26} s_{op}^3 w_r^{20s_{op}^{0.45}} + E \quad (7.22)$$

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 $\sigma(E)$:

$$\sigma(E) = 0.01936 \left[\frac{H_s}{\Delta^{0.74} d_n} \right]^{3.32} \quad (7.23)$$

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:

$$\begin{aligned} \frac{H_s}{\Delta d_n} &= 3.7 \quad (\text{start of damage; } N_{od} = 0) \\ \frac{H_s}{\Delta d_n} &= 4.1 \quad (\text{failure; } N_{od} > 0.5) \end{aligned} \quad (7.24)$$

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 1:1.33. 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, the placing method for Accropodes 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 1:1.333 (3:4), 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 7-7).

	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 7-7 s ($H_s/\Delta d_n$) for single layer elements (slope 1:1.33).

The values for "start of damage" and "failure" as given in table 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 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.

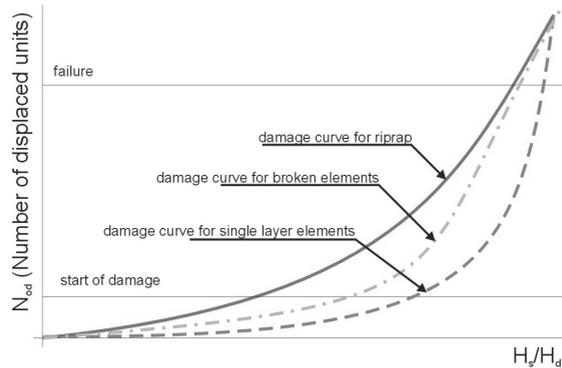


Figure 7-6 Damage progression for different type of armour units (from DE ROVER (2007)).

7.3 Stability calculation

Basically, the computation for the stability of armour units can be made by substituting the significant wave height of the design storm in the stability equations like (7.11) or (7.14). In the computation, one can use a deterministic approach, an approach based on partial safety coefficients, or one can use a full probabilistic approach. In case of a design storm one determines the H_{ss} 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_{ss} 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 part, and one for the load part. The values to be used follow from international guidelines, like 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 this last case one has to know especially the distribution of H_{ss} .

These methods are worked out in the 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”. As compared with “standard” quarry stone, the coefficient in the (deep water) Van der Meer formula changes slightly as shown in Table 7-4.

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

Table 7-4 Effect of stone shape on stability.

For a visual impression of block shapes, one is referred to Figure 7-7.

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

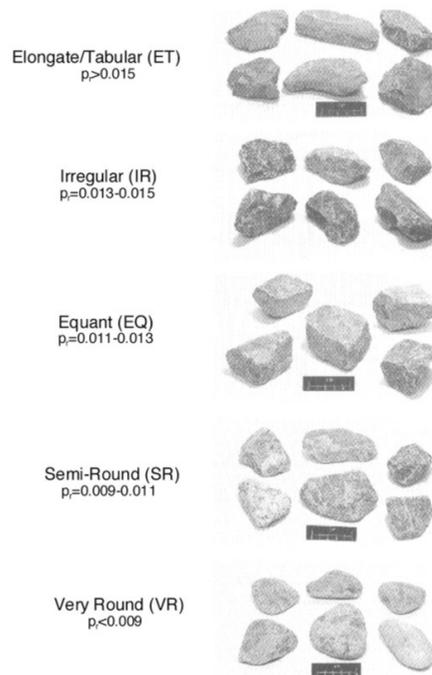


Figure 7-7 Visual comparison of block shapes (from Rock Manual, 2007).

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. Con-

verting mass into diameter is done by using the well-known d_n (nominal diameter) method:

$$d_n = \sqrt[3]{\frac{M}{\rho}} \quad (7.25)$$

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:

$$d_n = 0.8 d_s \quad (7.26)$$

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 grades. 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 there be supported by a toe. Of course some optimisation is possible. The higher the toe is placed, the less armour is needed. But 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]) did 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 $h/H_s < 2$):

$$\frac{H_s}{\Delta d_{n50}} = \left(0.24 \frac{h_t}{d_{n50}} + 1.6 \right) N_{od}^{0.15} \quad 3 < h_t / d_{n50} < 25 \quad (7.27)$$

$$\frac{H_s}{\Delta d_{n50}} = \left(6.2 \left(\frac{h_t}{h} \right)^{2.7} + 2 \right) N_{od}^{0.15} \quad 0.4 < h_t / h < 0.9 \quad (7.28)$$

Critical values for N_{od} are:

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

These values apply for a standard toe, with a height of 2 to 3 d and a width of 3-5 d . A definition sketch is given in Figure 7-8

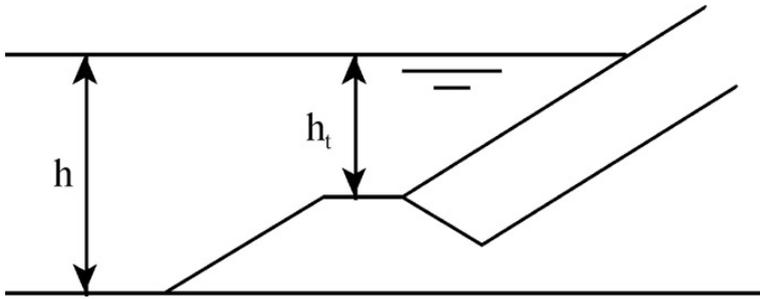


Figure 7-8 Definition sketch toe stability.

7.4.4 Breakwater head

The head of a breakwater is relatively vulnerable since, owing 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.

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; 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 with allows the production of concrete with a ρ_s up to 4000 kg/m³.

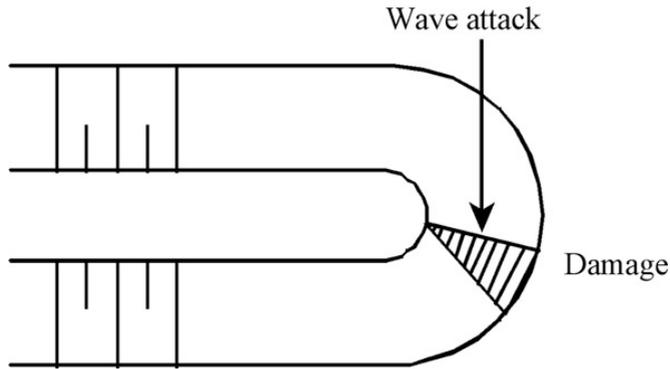


Figure 7-9 Typical damage pattern breakwater head.

7.4.5 Stability of crest and rear armour

As long as the crest of the structure is so high that it prevents 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.

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 one take into account waves penetrating through the entrance.

In many cases, however, the crest will not be so high, and in the design conditions considerable overtopping will take place. 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 the inner slope. The reduction of the attack on low crested breakwaters is discussed in Section 7.4.6.

In a limited study for a breakwater with Tetrapod armour layer, DE JONG [1996] concludes that the worst condition for the rear 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 $< -4 R_c / d_{n50}$), 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 is referred to the Rock Manual (2005).

7.4.6 Stability of low and submerged breakwaters

Essentially two cases can be discerned: a crest above still water level and a submerged crest. The European project Delos has lead to much understanding of these 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:

$$\frac{1}{1.25 - 4.8R_p^*}, \text{ for } 0 < R_p^* < 0.052 \quad (7.29)$$

With:

$$R_p^* = \frac{R_c}{H_s} \sqrt{\frac{s_{op}}{2\pi}} \quad (7.30)$$

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 above equations can be used as a first approach.

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 he Delos project followed that the Stability Number of the crest mainly depends on the ratio of the submergence and the rock size:

$$\frac{H_s}{\Delta d_{n50}} = A + B \frac{R_c}{d_{n50}} + C \left[\frac{R_c}{d_{n50}} \right]^2 \quad (7.31)$$

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:

$$d_{50} \geq 0.3d \quad \text{for} \quad \frac{H_s}{h} = 0.6, \cot \alpha_s \geq 100 \quad \text{and} \quad \Delta \approx 1.6 \quad (7.32)$$

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 breaking does not have a significant influence. 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. 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: :

$$\frac{u^2}{g\Delta d_{n50}} = \left(5 \frac{S}{\sqrt{N}} \right)^{1/3} \quad (7.33)$$

$$u = u_0 = \frac{\pi H_s}{T_m} \frac{1}{\sinh(k(-R_c))}$$

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

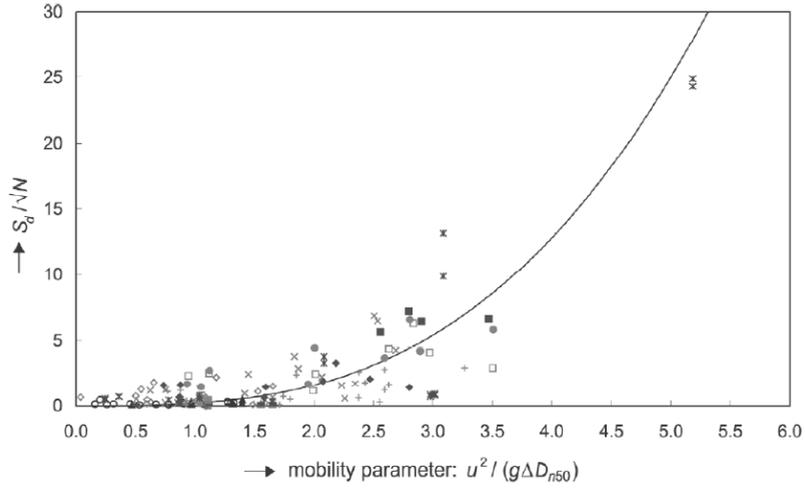


Figure 7-10 Illustration of the spreading around equation (7.31) for the stability of near-bed structures.

As can be seen from Figure 7-10 there is a large spreading of the results around this design line.

8 BERM BREAKWATERS AND DYNAMIC STABILITY

Design of breakwaters is gaining more and more attention. The term berm breakwater refers to the presence of an extended quantity of stone on the seaward side of the breakwater. This volume may be sufficient to allow for a certain deformation without endangering the integrity of the structure as a whole. But also the berm may have other functions like decreasing wave load on the higher slopes. This chapter gives some empirical data on berm breakwaters. It is good to remind that nearly all dynamically stable breakwaters are berm breakwaters but not all berm breakwaters are dynamically stable

8.1 Introduction

In Chapter 7, the stability of blocks 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 blocks. It is not always possible to obtain such heavy blocks of quarry stone because of geological limitations of the quarry. Casting the blocks in concrete is complicated because very large units are rather sensitive to breaking.

Whether it would be possible to allow slight movements of the armour stone, 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 will tend to become gentler. To maintain the overall function of the breakwater, including the required crest level, one has to provide extra material. 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 added advantage of this type of breakwaters is the fact that a wider gradation of material can be used. This eliminates the need for expensive sorting operations at the quarry. The wider gradation, plus the fact that the maximum stone size is limited, makes it easier to bring the stone demand for the design into accordance with the 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 cost. However, one must ascertain that a sufficient volume of material is used per running metre of cross section.

Breakwaters with a berm may be divided into three types [PIANC/MARCOM 40, 2003]:

- Statically stable non-reshaping structures. In this condition only some few 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 breakwater is discussed. At the end of this chapter attention is paid to the statically non-reshaping 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 graded can be used in specified places inside the structure. Instead of a wide stone gradation of the dynamically stable breakwater the statically stable non-reshaping

breakwater has a narrow stone gradation which means higher permeability and increase 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 size-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.

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). 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, the go to wider and more voluminous berm breakwaters (statically stable reshaped type).
4. If 1) to 3) are not possible options, then check out 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 shore, but is not suitable for a head section.

Traditionally a breakwater design starts with the design wave height, which leads to a set of required rock sizes. As long as 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 to the output yield of the quarry.

8.2 Seaward profiles

Model tests on slopes that are not statically stable have indicated that a typical profile is formed according to Figure 8-1.

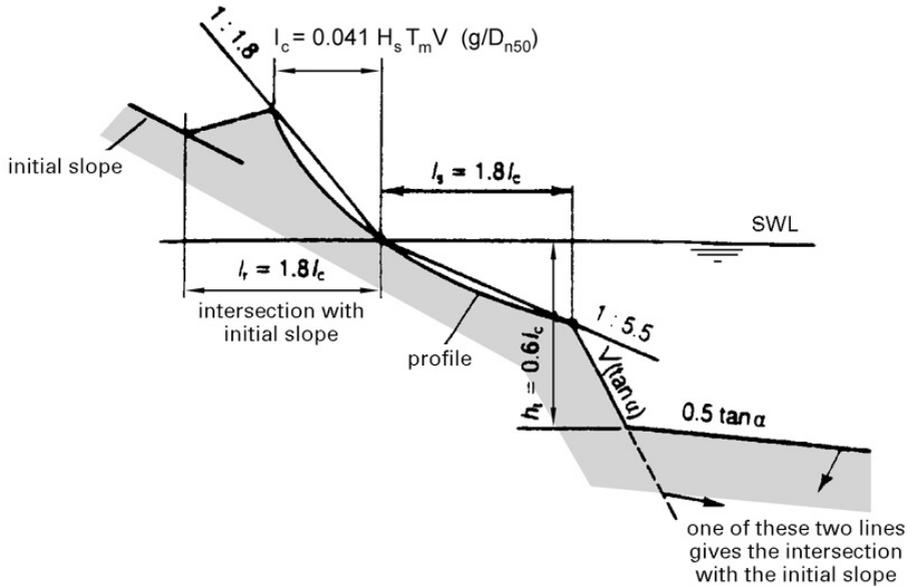


Figure 8-1 Schematised profile for sand gravel beaches

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

$$\ell_c = 0.041 H_s T_m \sqrt{\frac{g}{d_{n50}}} \quad (8.1)$$

$$\ell_s = \ell_r = 1.8 \ell_c \quad (8.2)$$

$$h_t = 0.6 \ell_c \quad (8.3)$$

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 1:1.8 and a lower slope at 1:5.5. 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.

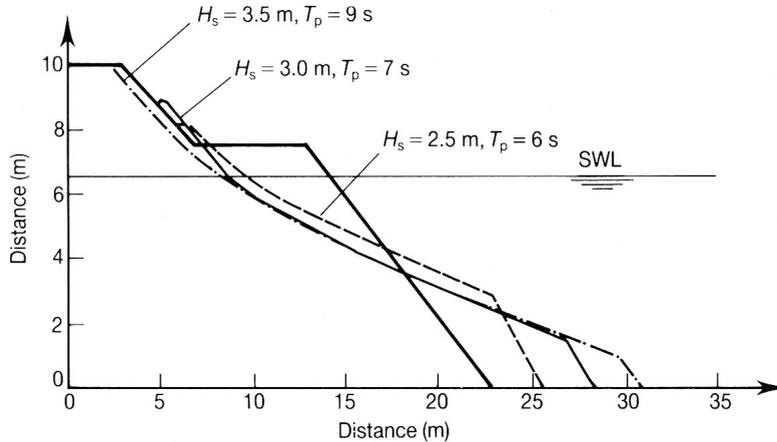


Figure 8-2 Influence of wave climate on a berm breakwater profile.

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-section 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.

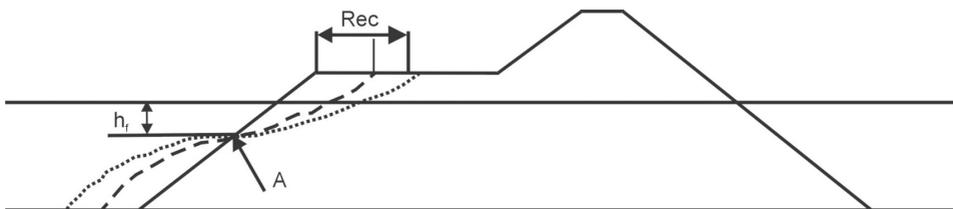


Figure 8-3 Recession on a reshaping breakwater.

Usually the recession length is made dimensionless by dividing by $d_{n,50}$. Apart from the above iteration procedure, several researchers have developed more direct approximation methods, using curve fitting. All these equations have the form:

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

Note: the notation *HoTo* may cause some confusion; both *Ho* and *To* are dimensionless parameters, *Ho* is equal to the stability number. Because of this potential confusion the parameters *Ho* and *To* 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 formula is the formula developed by Tørum [PIANC-MARCOM 2003]:

$$\begin{aligned} \frac{Rec}{d_{n50}} &= A(HoTo)^3 + B(HoTo)^2 + C(HoTo) - f(f_g) - f\left(\frac{h}{d_{n50}}\right) \\ f(f_g) &= -9.9f_g^2 + 23.9f_g - 10.5 \\ f\left(\frac{h}{d_{n50}}\right) &= -0.16\left(\frac{h}{d_{n50}}\right) + 4.0 \quad (12.5 \frac{h}{d_{n50}} < 25) \\ f_g &= \frac{d_{n85}}{d_{n15}} \quad (1.3 < f_g < 1.8) \end{aligned} \quad (8.5)$$

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

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 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 state 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:

$$\frac{H_s}{\Delta d_{n50}} \leq 4.5 \quad (8.6)$$

Trunks exposed to oblique waves:

$$\frac{H_s}{\Delta d_{n50}} \leq 3.5 \quad (8.7)$$

Breakwater Heads:

$$\frac{H_s}{\Delta d_{n50}} \leq 3 \quad (8.8)$$

Van der Meer has carried out similar tests and concludes that the number of stones transported per wave along the breakwater $S(x)$ is at its maximum for wave angles between 15 and 35 degrees. According to VAN DER MEER [1992]) the transport is:

$$S(x) = 0 \quad \text{for } HoTo < 105 \quad (8.9)$$

and for $HoTo > 105$:

$$S(x) = 0.00005 (HoTo - 105)^2 \quad (8.10)$$

8.4 Crest, rear slope and head

One of the design principles of a berm breakwater is the simplification of the cross-section. Therefore, the armour on the crest and the rear slope is the same as on the front slope. Since we have seen in the previous sections that $H_s/\Delta d_{n50}$ should be in the order of 3 to 3.5, this applies for the armour on the rear slope as well.

From model investigations by VAN DER MEER AND VELDMAN [1992], it appeared that the crest level (R_c) determines the damage to the inner slope, with a slight influence of the wave steepness as well.

Start of damage:

$$\frac{R_c}{H_s} s_{op}^{1/3} = 0.25 \quad (8.11)$$

Moderate damage:

$$\frac{R_c}{H_s} s_{op}^{1/3} = 0.21 \quad (8.12)$$

Severe damage:

$$\frac{R_c}{H_s} s_{op}^{1/3} = 0.17 \quad (8.13)$$

It is evident that wave attack on the round head of a berm breakwater shows similarities to wave attack by oblique waves. As stated in Section 8.3, it is recommended that the value of $H/\Delta d$ should be limited to less than 3.

Furthermore, it is a wise measure to supply an extra buffer of armour stone at the round head when longshore transport is expected. The extra quantity in the buffer can be created by increasing either the height or the width of the berm. However, the buffer must not become an obstruction to safe navigation.

8.5 The Icelandic breakwater

A modification of the originally dynamic breakwaters has evolved into the statically stable berm breakwater. This structure is more stable than the original dynamic breakwater but at the same time less voluminous. This type is also called “Icelandic Breakwater”, because this type is developed by Sigurður Sigurðarson of the Icelandic Maritime Administration. It is built up 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 for the above reasons extremely important for the planning and economics of a successful breakwater project that information on the specific quarry is available at an early stage. So one should first analyse a promising quarry, and determine the potential yield curve for this quarry. By using appropriate blasting techniques (see Appendix 2.8) one may produce a relatively large percentage of big stones.

Then the cross section of the breakwater is designed given the yield of the quarry. One can define a number of stone classes and calculate the amount of stone available in each class. For example for the Sirevåg breakwater the quarry was able to provide the following classes:

Stone class	W_{\min} - W_{\max} (tonnes)	$-W_{\text{mean}}$ (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 [SIGURDARSON, 2004].

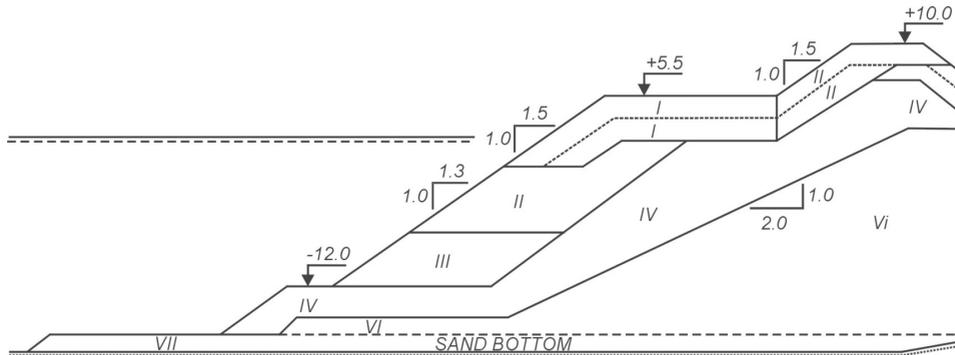


Figure 8-4 Multi-layer or non-homogeneous berm breakwater (Sirevåg in Norway)[PIANC-MARCOM, 2003].

A cross section of the outer part of the 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 for the three quarries for a 640,000 m³ breakwater. The armourstone material is anorthosite gabbro rock of good quality ($\rho_s = 2700 \text{ kg/m}^3$). 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.

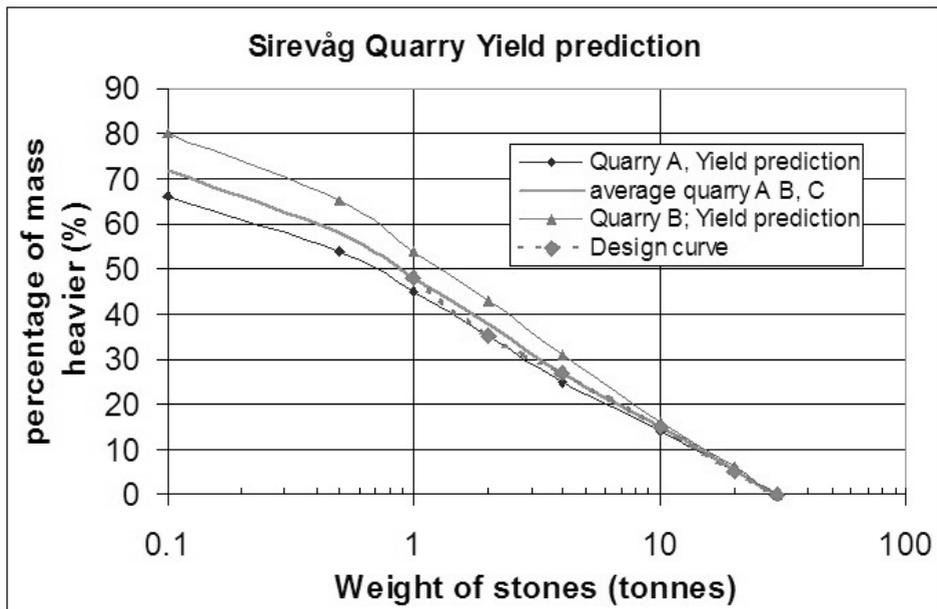


Figure 8-5 Quarry yield prediction curve and real production curve for Sirevåg breakwater [SIGURDARSON, 2004].

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 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.

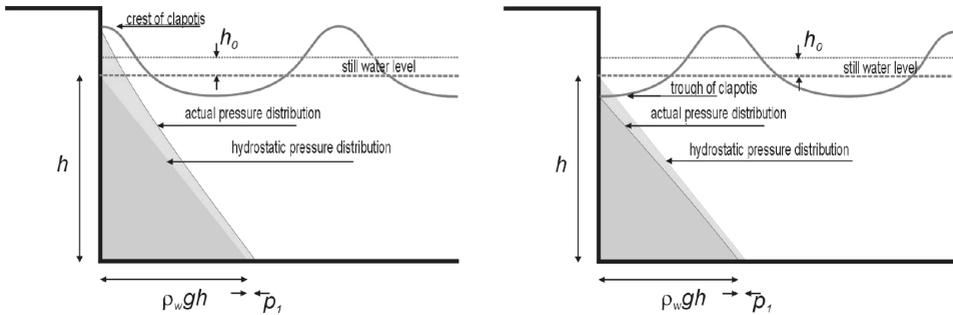


Figure 9-1 Schematic pressure distribution for non-breaking waves.

In this figure p_1 is given by:

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

and H_i is the (undisturbed) incoming wave height and K_r the reflection coefficient. Design manual may 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 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.

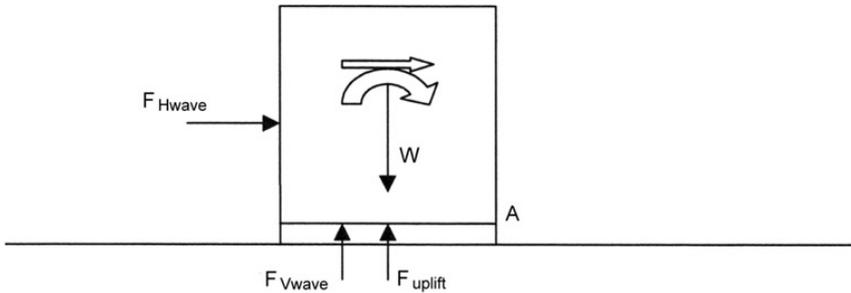


Figure 9-2 Load and equilibrium diagram.

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 9.2.1, 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 ρgH , 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 the 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.

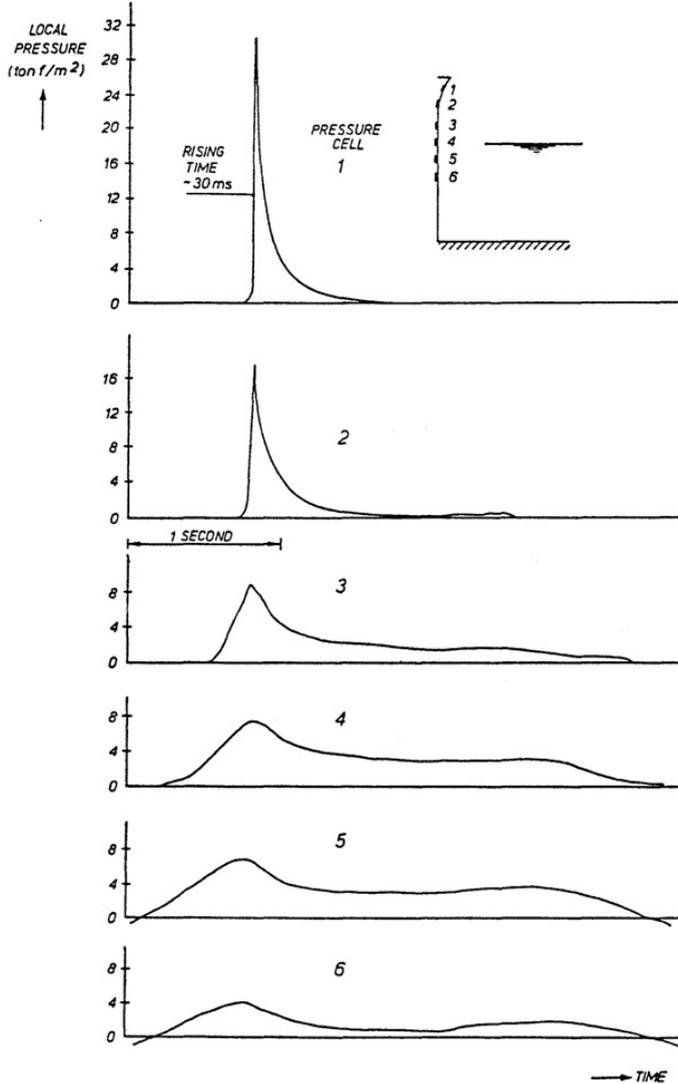


Figure 9-3 Example of a pressure record.

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:

$$p = \frac{1}{2} \rho u^2 \quad (u \text{ is the water velocity in the jet})$$

- The water hammer effect, resulting in:

$$p = \rho u c$$

in which:

u = the water velocity in the conduct

c = the celerity of sound in water (1543 m/s)

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 in 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 be 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 would 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 placed on a high mound of stone (see Figure 9-4).

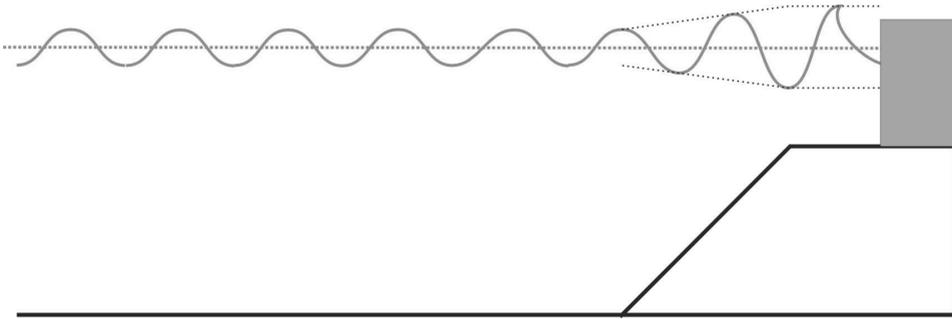


Figure 9-4 Changes to incoming wave front induced by high mound breakwater.

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).

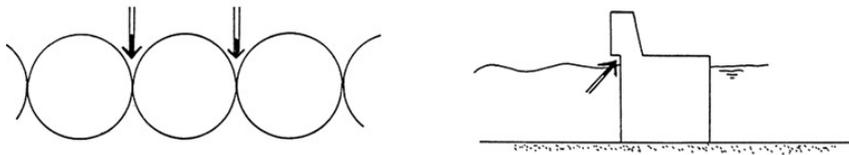


Figure 9-5 Risk of local impact forces.

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 satisfaction. Goda analysed many of the successful and unsuccessful structures and came up with a practical formula that can be used to analyze 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].

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 9-6).

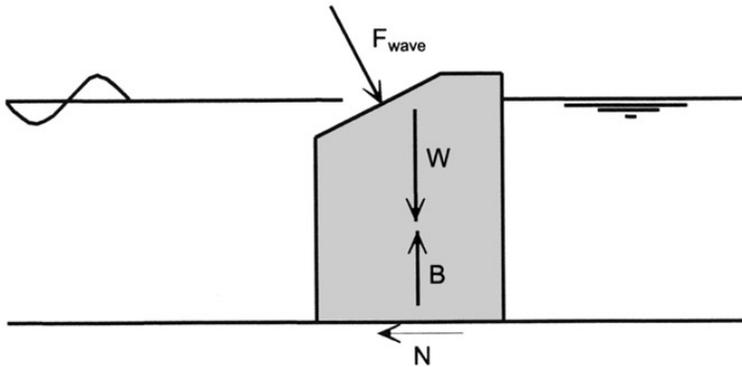


Figure 9-6 Hansthalm caisson.

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 9-7), partly to reduce forces, 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 9-8). Many of the designs have been described by TAKAHASHI [2002]. Two examples of their typical designs are given in Figure 9-9 and in Figure 9-10.

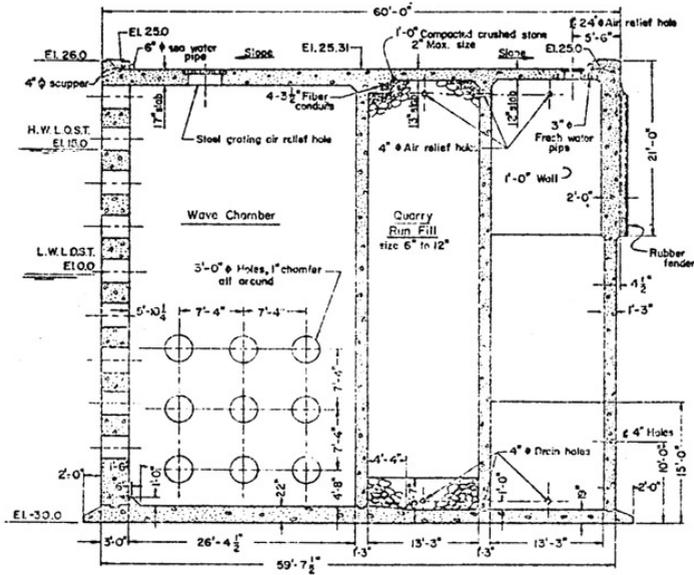


Figure 9-7 Jarlan caisson.

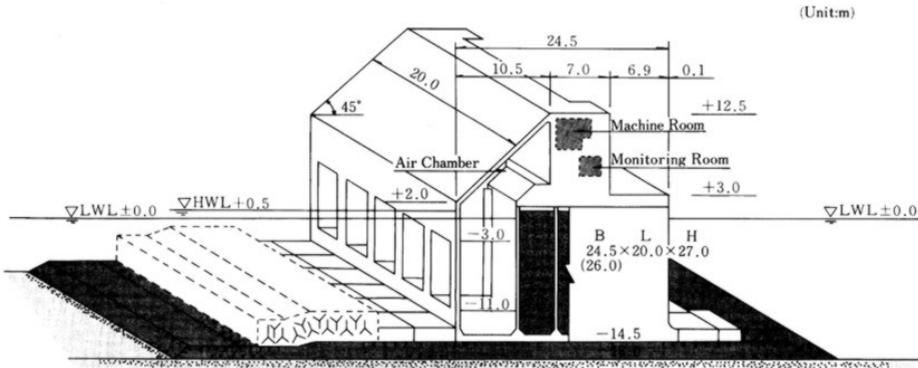


Figure 9-8 Breakwater with a wave power generating system.

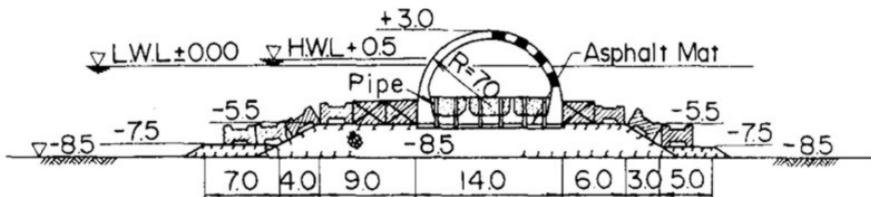


Figure 9-9 Possible cross-section of semi-circular caisson breakwater for extremely high breakers.

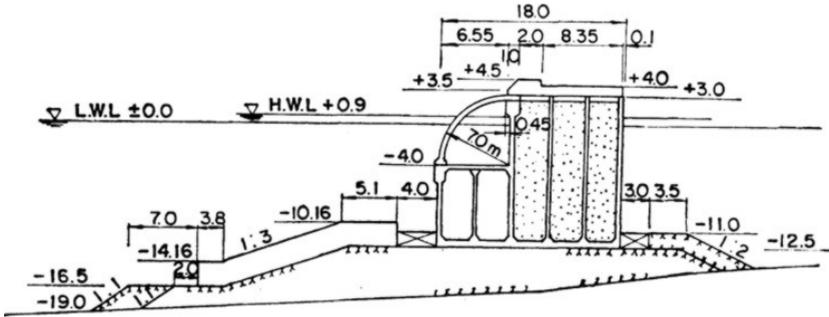


Figure 9-10 Cross section of curved-slit breakwater at Funakawa Port.

Many other designers followed the ideas of Jarlan. One of these designs is given in Figure 9-11. 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 effectively for a given wave period, but when the period is only a few percent longer, the damping is considerably less.

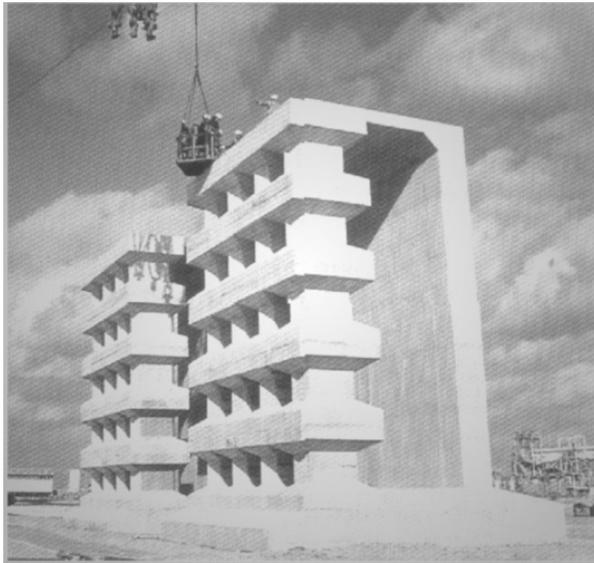


Figure 9-11 Honey wall breakwater.

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
- 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 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 ($\frac{1}{4}L \pm \frac{1}{2}L$) 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 [MEI, 1989]. The sediment on the bed will respond to these circulations. When the bed consists of coarse 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.

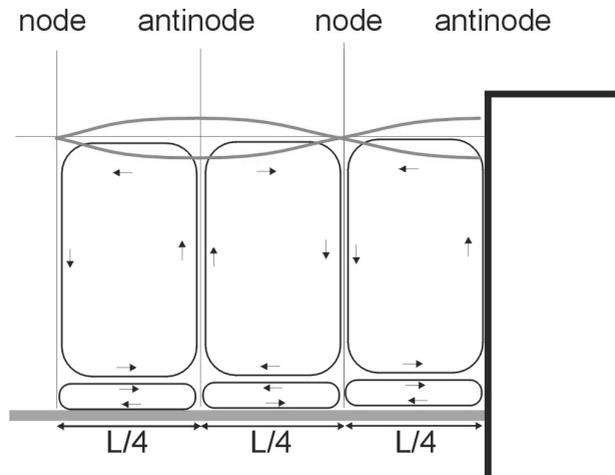


Figure 9-12 Steady streaming in the vertical plane in front of a breakwater.

This means that for fine material (suspension mode) accretion can be found near the antinodes and for coarse material (non-suspension mode) accretion can be found near the nodes. See also Figure 9-13, 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 such blocks.

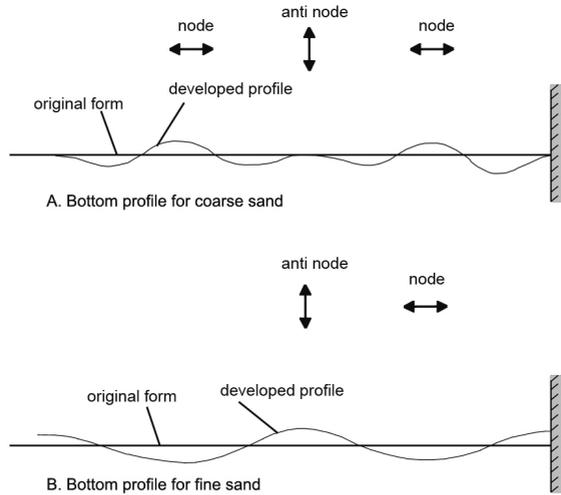


Figure 9-13 Erosion pattern depending on the grain size.

9.6 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 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. In first instance 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:

$$\sigma = p + \sigma' \quad (9.2)$$

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 like 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 interaction between the structure and the wave field near the structure. We discern various phenomena that lead to different wave patterns in the vicinity of the structure:

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

Wave reflection plays a role 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. Before using any of the expressions given in this chapter, it is useful to analyze which phenomenon influences the design problem in question. 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 to assess if on 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.

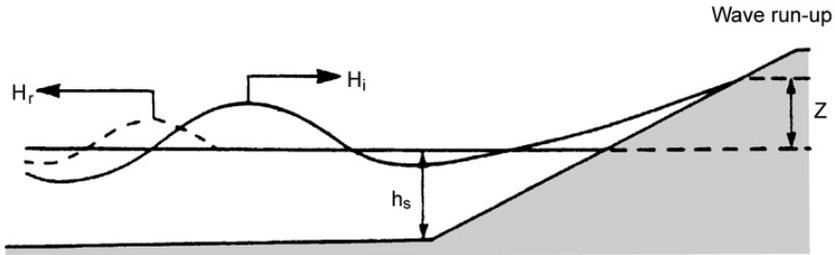


Figure 10-1 Definition of wave run-up.

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.

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.

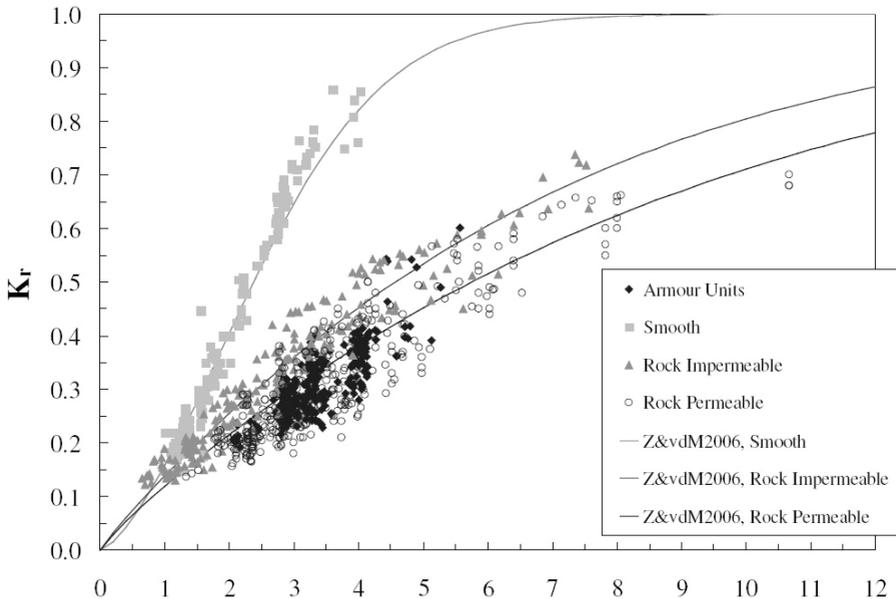


Figure 10-2 Reflection data for straight slopes [ZANUTTIGH AND VAN DER MEER, 2007].

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

$$\begin{aligned}
 K_r &= \tanh \left(a \xi_{m-1,0}^b \right) \\
 a &= 0.167 \left[1 - \exp(-3.2\gamma_f) \right] \\
 b &= 1.49 (\gamma_f - 0.38)^2 + 0.86
 \end{aligned}
 \tag{10.1}$$

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

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 10-1). 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 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 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 given: 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:

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

with a maximum of $\gamma_f\gamma_\beta \left(B - C / \sqrt{\xi_{m-1,0}} \right)$, 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, the other set is the parameter to be used in designs. This is in fact the average value plus one times the standard deviation. The average value can be used in probabilistic designs, while the second value can be used in deterministic designs.

Type of paramter	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. Waves approaching the structure at an angle will also lead to reduced run-up levels. 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 10-1 gives some values. For more impermeable slopes some values can be found in Table 10-3.

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 is referred to the Overtopping Manual [PULLEN *ET.AL.*, 2007].

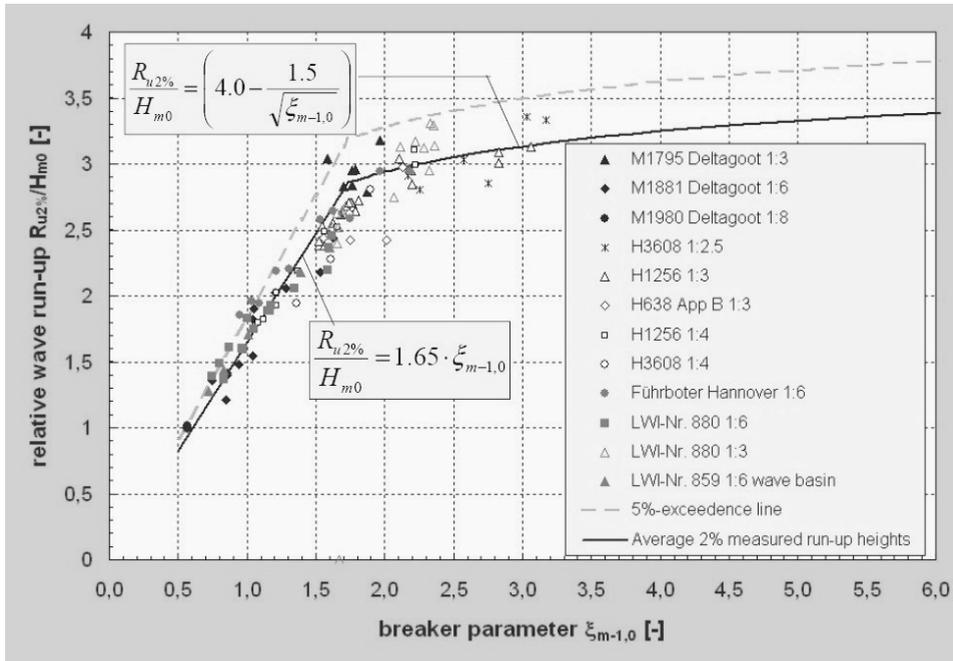


Figure 10-3 Wave run-up on a smooth impermeable slope [PULLEN *ET.AL.*, 2007].

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.

For a permeable rubble construction basically the same equation can be used. However as can be seen from Figure 10-4 for high Iribarren numbers there are some differences compared to smooth slopes. For two layers of rock on an 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).

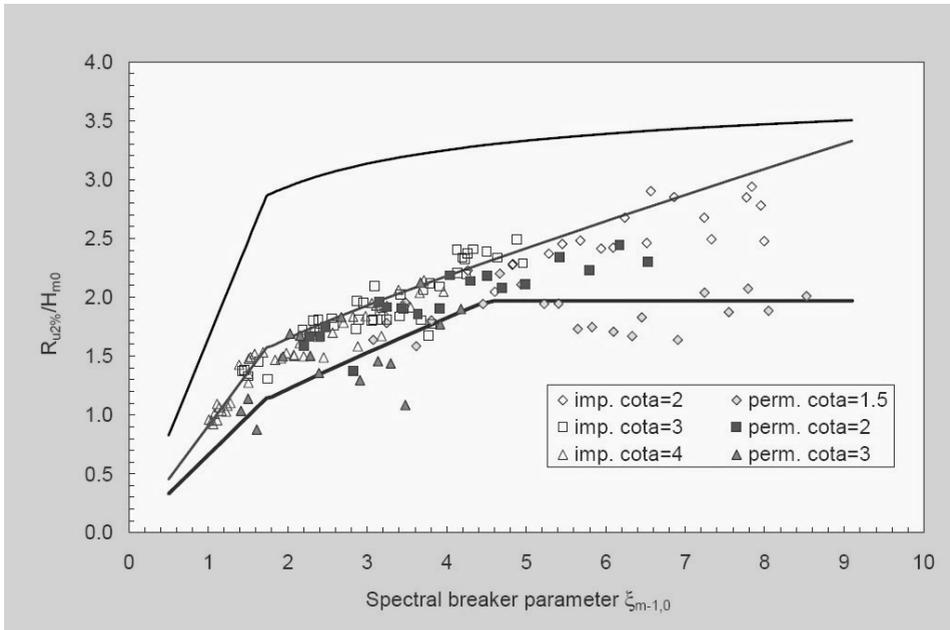


Figure 10-4 Relative 2% run-up on rock slopes [PULLEN *ET.AL.*, 2007].

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:

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

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 as the actual height (A_c) of the breakwaters. For various armour units one may combine several formulas, leading to one expression including the wave height and the unit dimension.

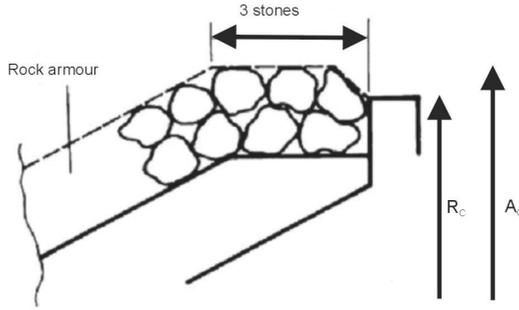


Figure 10-5 Difference between R_c and A_c

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

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

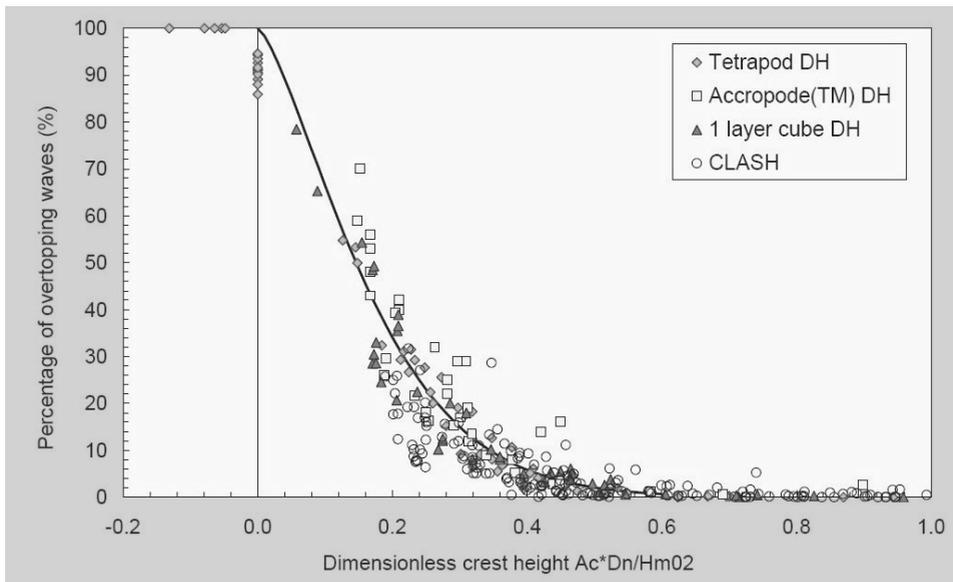


Figure 10-6 Percentage of overtopping waves for rubble mound breakwaters as a function of relative crest height and armour size ($R_c < A_c$) [PULLEN ET.AL., 2007].

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 [$\text{m}^3/\text{s}/\text{m}$].

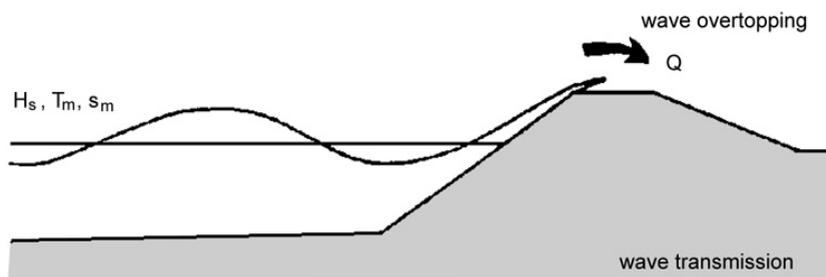


Figure 10-7 Typical wave overtopping.

The study of overtopping quantities is again a subject that was initiated in relation to the stability of the inner slopes of grass covered dikes. 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.

The Eurotop manual [PULLEN *ET AL*, 2007] 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 is referred to SCHIERECK [2001]

Systematic research on overtopping for smooth and impermeable slopes had lead 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:

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

See also Figure 10-8. The roughness factors from Table 10-1 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] gives some empirical formulae for special cases.

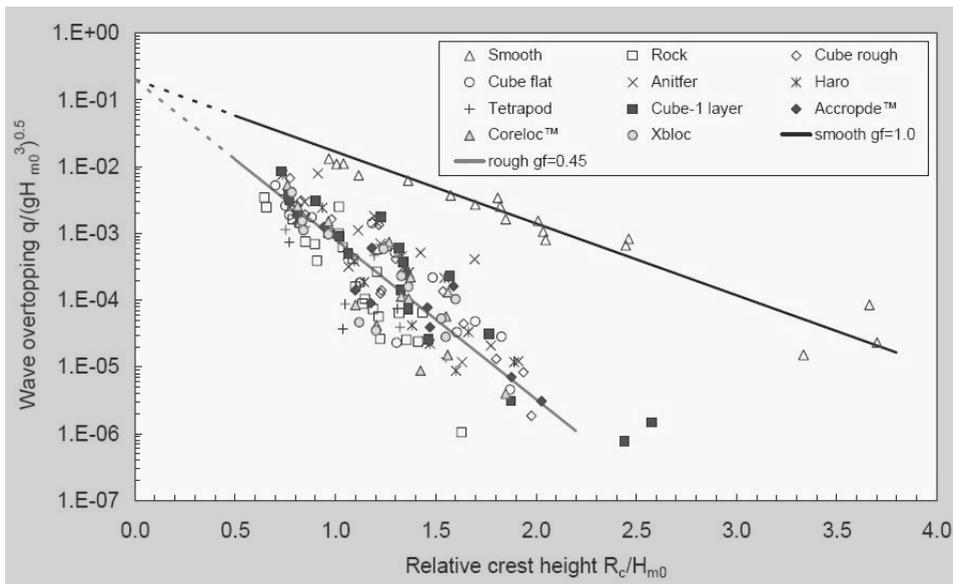


Figure 10-8 Mean overtopping for 1:1.5 slopes (smooth and rough slopes).

Spatial distribution of overtopping

Part of the water which overtops the outer crest line will infiltrate in the crest, part 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 in 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:

$$\frac{q_2}{q_1} = \begin{cases} \left(\frac{Q_{tot}^* - Q_d^*}{Q_{tot}^* + 0.07} \right) & \text{for } Q_{tot}^* > Q_d^* \\ 0 & \text{for } Q_{tot}^* < Q_d^* \end{cases} \quad (10.7)$$

in which:

q_1, q_2 overtopping in m^3/s per meter breakwater, as defined in the figure
 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

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:

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

in which:

q = unit discharge ($\text{m}^3/\text{m}/\text{s}$)

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 transmission is much more, and is given by:

$$\frac{q}{h_*^2 \sqrt{gh_s^2}} = a \left(h_* \frac{R_C}{H_{m0}} \right)^{-3.1} \quad (10.9)$$

in which:

$$h_* = 1.35 \frac{h_s}{H_{m0}} \frac{2\pi h_s}{gT_{m-10}^2}$$

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 $a = 1.5 \cdot 10^{-4}$.

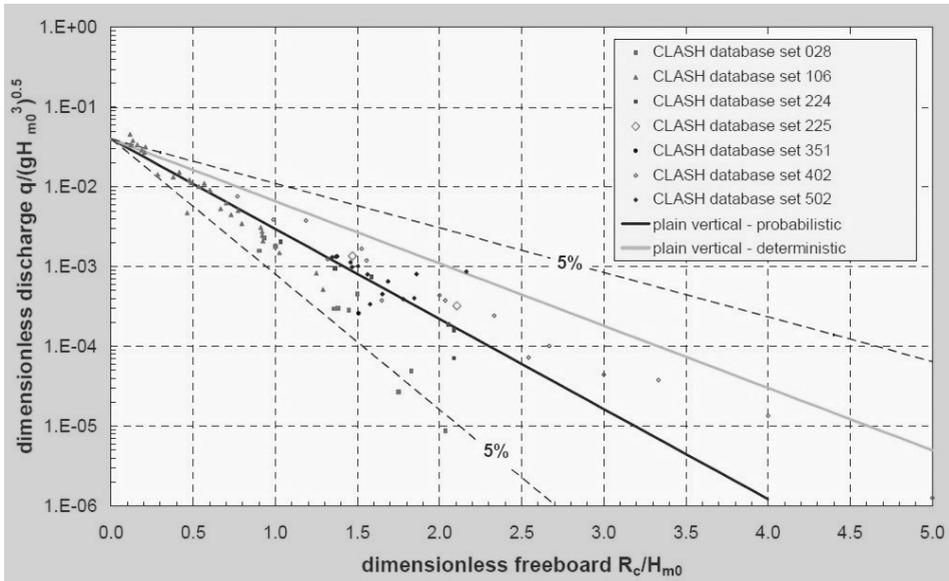


Figure 10-9 Wave transmission for a vertical breakwater.

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 is referred to the Eurotop manual [PULLEN, 2007].

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.

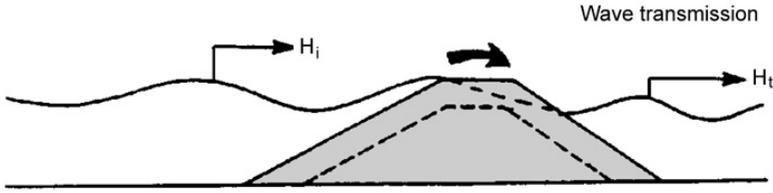


Figure 10-10 Typical wave transmission

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. It must be noted that the transmission coefficient can never be smaller than 0 or larger than 1. In practice, limits of about 0.1 and 0.9 are found (Figure 10-11).

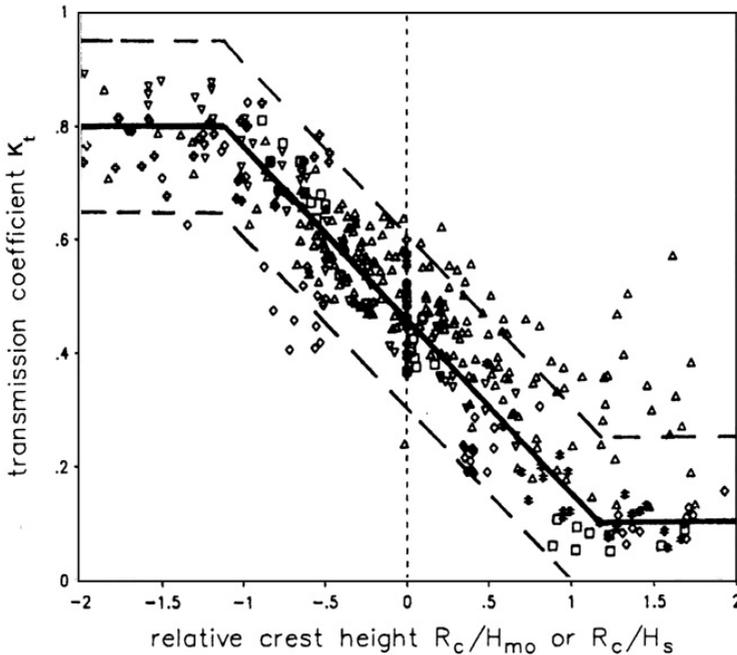


Figure 10-11 Wave transmission for low crested structures [D'ANGREMOND *ET AL.*, 1996].

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 armour stone diameter. This eliminates much of the scatter that was present in previous approaches. For traditional low crested breakwaters the Daemen formula reads as follows:

$$K_t = a \frac{R_c}{d_{n50}} + b \quad (10.10)$$

with:

$$a = 0.031 \frac{H_i}{d_{n50}} - 0.24 \quad (10.13)$$

$$b = -5.42s_{op} + 0.0323 \frac{H_i}{d_{n50}} - 0.0017 \left(\frac{B}{d_{n50}} \right)^{1.84} + 0.51 \quad (10.11)$$

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 armour stone

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 armour stones with asphalt into a solid mass. Therefore, DE JONG [1996] (see also D'ANGREMOND *ET AL.* [1996]), reanalyzed the data and came up with a different expression. He choose to make the freeboard dimensionless in relation with the incoming wave height:

$$K_t = 0.4 \frac{R_c}{H_{si}} + 0.64 \left(\frac{B}{H_{si}} \right)^{-0.31} \left(1 - e^{-0.5\xi} \right) \quad (10.12)$$

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 10-11.

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 (10.12). For wider crests this formula is not valid. For wide crested breakwaters ($B/H_s > 12$) one may use

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

The lower boundary is $K_t = 0.05$, the upper boundary is $K_t = 0.93 - 0.006 B/H_s$. Interpolation between the two equations is done for $8 < B/H_s < 12$. See also Figure 10-12.

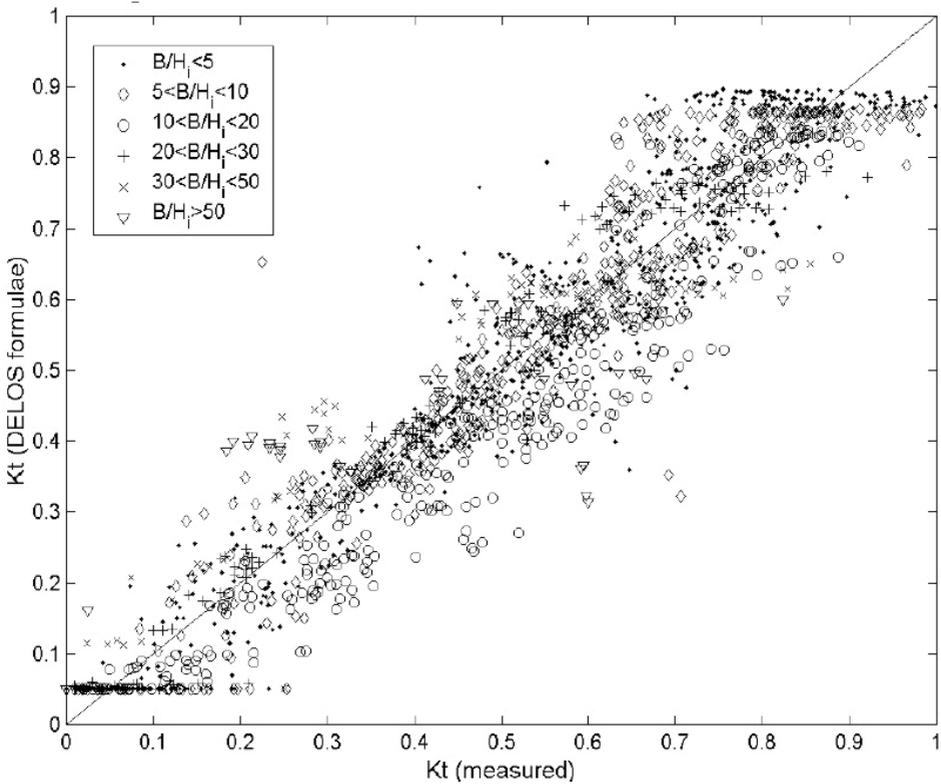


Figure 10-12 Comparison of computed and observed transmission coefficients with equations (10.11) and (10.12) (from DELOS database).

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 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. Using such a database a Neural Network can be trained. With a trained Neural Network anyone can make calculations in 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.

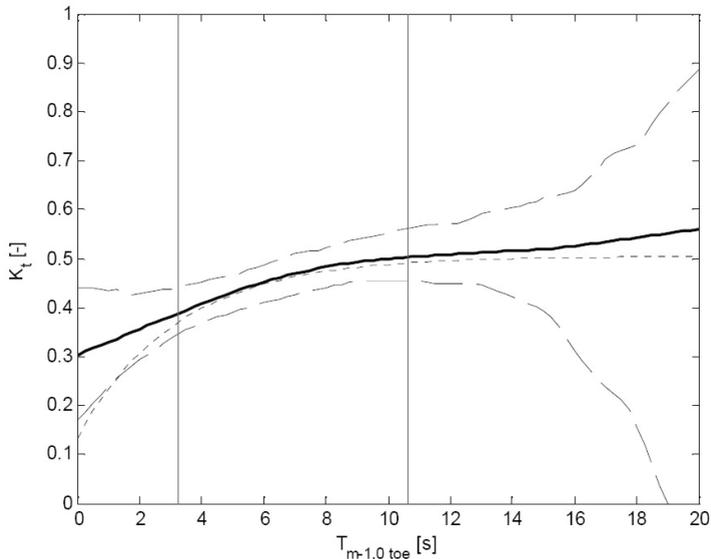


Figure 10-13 Result of a Neural Network calculation [VAN OOSTEN and PEIXÓ MARCO, 2005].

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 dashed line in the figure is the result of equations (10.11) and (10.12). The thick line is the prediction of the Neural Network. But also the reliability is given. It is clear that for periods larger than 11 s one should be very careful with the found results. Because the Neural Network and eqs. (10.11) and (10.12) are based on the same data, the unreliability for periods longer than 11 s in this particular case is also applicable for eqs. (10.11) and (10.12).

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 a design problem 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 analyzes the examples at the same time and wonders why the specific designs have been followed.

11.1 Introduction

In the foregoing 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.

- The first one is to 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).
- The second option includes the 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 the 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, in this case also 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 determines at least part of the hydraulic response of the structure, and it influences the stability (P in the Van der Meer formula). During construction, it is important because the porosity determines the bulk density. Quarry stone is often paid for per ton of material to the quarry operator. When the contractor is paid per m^3 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:

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

in which:

ρ_b = bulk density as laid

ρ_r = density of rock

Determination of the bulk density is not simple 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 11-2, 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 11-2. 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 is too low, and the client still wants the given crest level, this means that extra armour stone has to be used. Since armour

stone is generally more expensive than core material, this again will cause a financial loss to the contractor. See also Appendix 7.

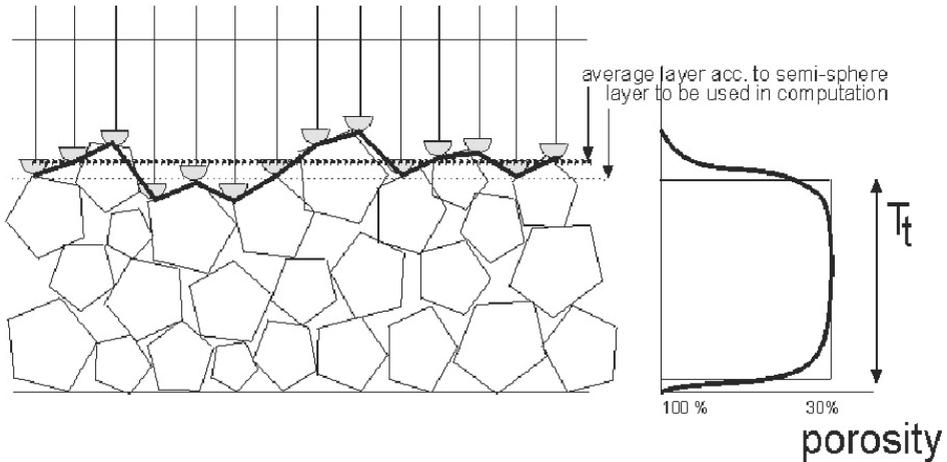


Figure 11-1 measuring the thickness of a rock layer.

For relatively thin layers, there is a variation in n_v at the boundaries of the various layers. First of all the boundary is not defined easily. Usually one measures the toplayer with the so-called hemisphere method or spherical foot method. See Figure 11-1. From the work of BOSMA [2001] 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:

$$t = n k_t d_{n50} \quad (11.2)$$

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:

$$N = n k_t A (1 - n_v) d_{n50}^{-2} \quad (11.3)$$

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, 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 envisaged 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.

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.

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 armour stone 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 anyway, it makes no sense to construct the cross-section to a particular slope. It is generally accepted that the material should be left 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 11-2 and Figure 11-3.

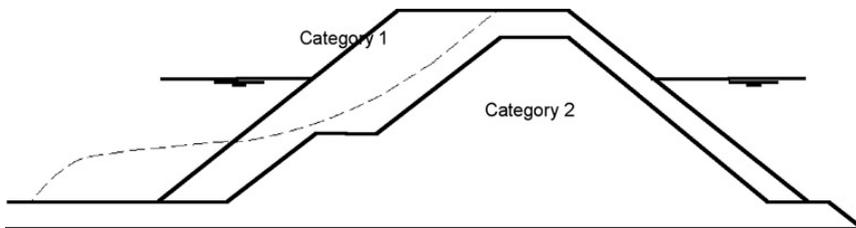


Figure 11-2 Berm breakwater with high berm.

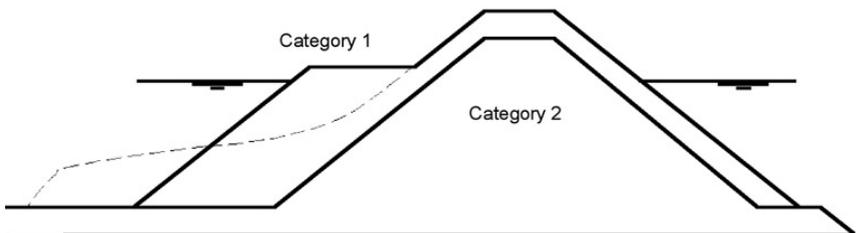


Figure 11-3 Berm breakwater with berm at MSL.

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, and a third type refers to the kind of armour units.

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 tranquility on the lee side). It must be clear that 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 also have serious consequences for the construction method and the maintenance. A low crest may 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 say $1:1\frac{1}{2}$ (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 11-4. It indicates the main elements of such a breakwater and their respective names.

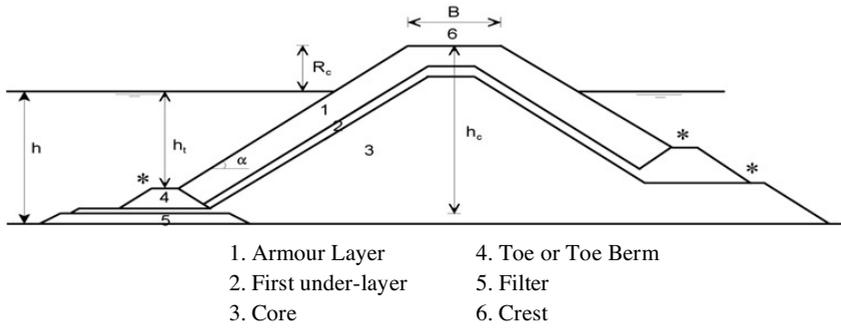


Figure 11-4 Definition sketch of cross-section.

Tolerances

Regardless of the construction element under consideration, it must be clear that considerable 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 1 or 2 m 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 11-4 through Figure 11-9.

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:

$$B = n k_t d_{n50} \quad (11.4)$$

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 11-5).

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 the literature, 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. 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. It is generally a narrow grading.

Toe berm

The toe berm is the lower support for the armour layer. In traditional literature, one finds a weight recommended that is equal to the weight of the first under-layer. With the most recent data that are 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. This means that 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 usually 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 an important threat to the stability of the armour layer. There are ample examples in the literature that 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 other 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 (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 presence of that particular 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 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 11-5, to Figure 11-9, examples of standard cross-sections based on the considerations given in Section 11.4.2 are given. 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.

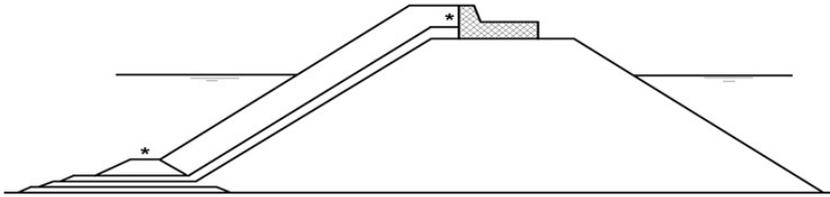


Figure 11-5 Rubble mound breakwater – light overtopping (with cap block).

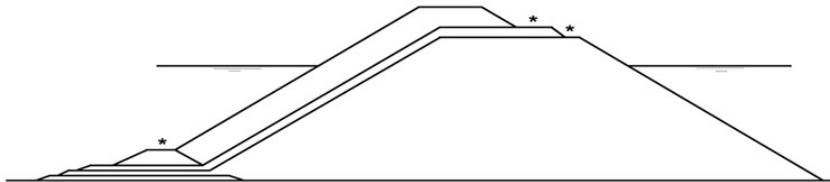


Figure 11-6 Rubble mound breakwater - light overtopping.

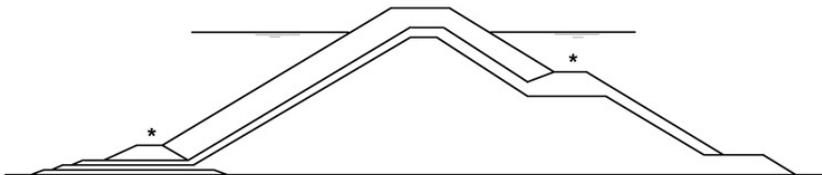


Figure 11-7 Rubble mound breakwater - moderate overtopping.

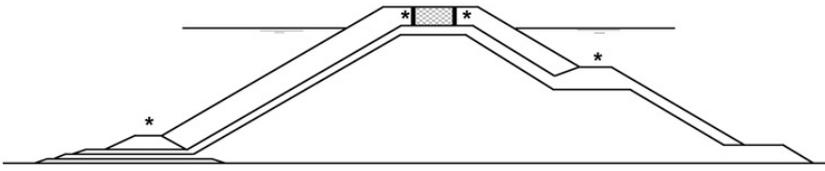


Figure 11-8 Rubble mound breakwater - moderate overtopping (with cap block).

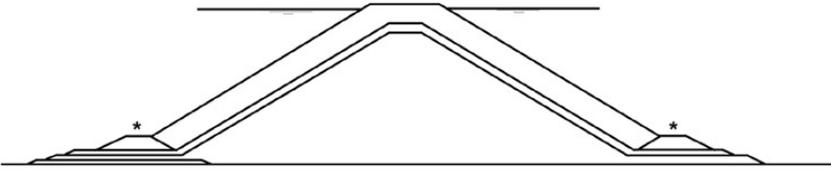


Figure 11-9 Rubble mound breakwater - severe overtopping.

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 11-10, Figure 11-11, Figure 11-12 and Figure 11-13.

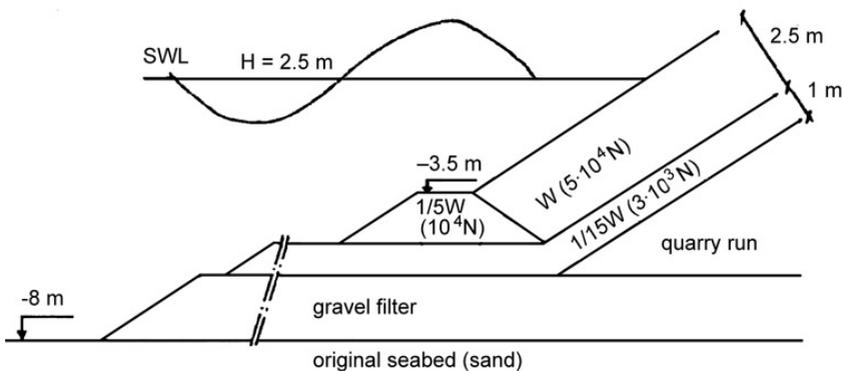


Figure 11-10 Shallow water, dredged trench.

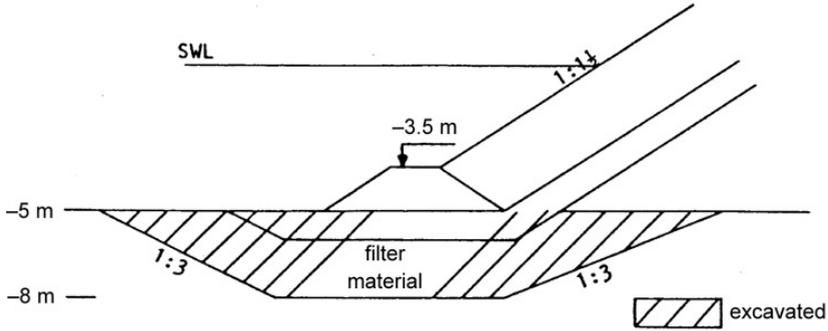


Figure 11-11 Shallow Water dredged trench gravel filter.

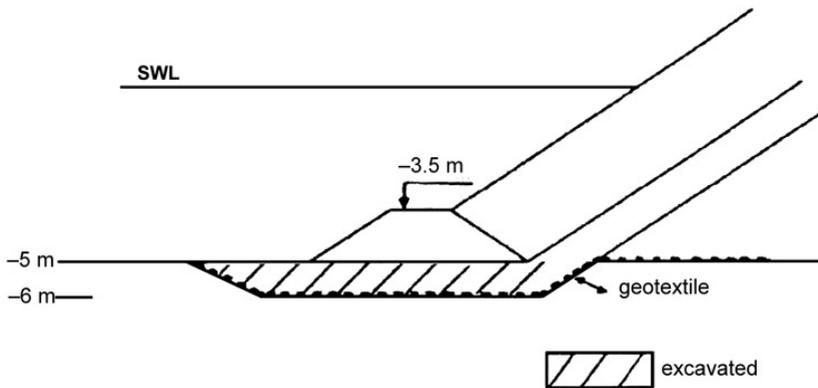


Figure 11-12 Shallow water, dredged trench geotextile.

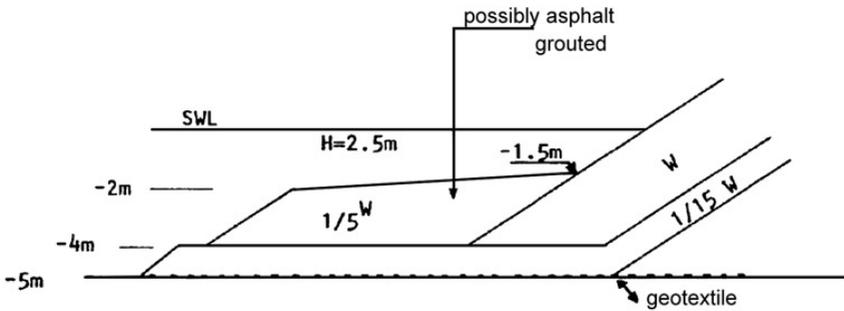


Figure 11-13 Shallow water (no excavation, geotextile and increased berm).

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 that safety levels for monolithic breakwaters have to be different. For details 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.

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 11-14). This means that one has to define two design wave heights: a value H_r with is the design wave height for the ULS condition and a value H_u which is the design wave height for the SLS condition.

- Select a wall that presents a free height 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 $1.72 H_r$ 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 $0.75 H_u$
- 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 11-14. 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].

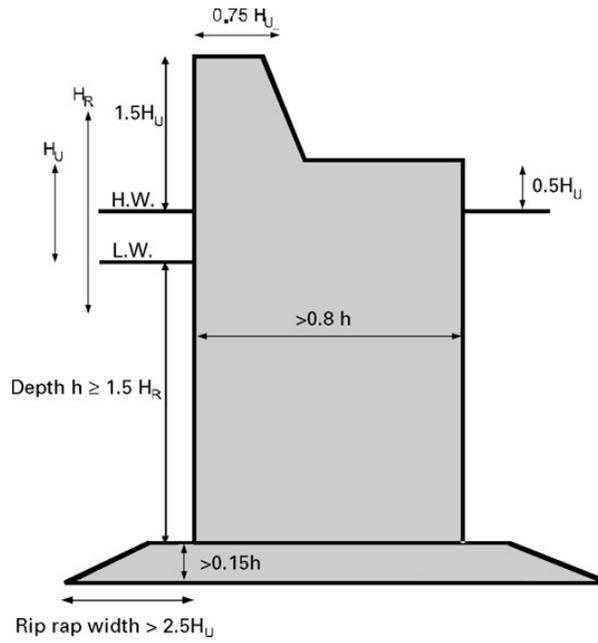


Figure 11-14 Recommended cross-section for a preliminary design.

12 DESIGN PRACTICE FOR CLOSURE DAMS

Chapter 12 treats the design of closure dams. The focus is on closures of short basins using the storage/area approach. Contrary to Chapter 11, little attention is given to the cross-section of the closing dam. This is not so relevant because the shape of the dam is not relevant for the closing operation. The main issue in this chapter is the closing operation itself. After closure the dam is completed as a normal dam or dike. The design of the final cross-section depends on future use and is not discussed in this book. Important is the understanding of the relation between narrowing the cross-sectional profile and the tidal motion.

12.1 Basics of the storage area approach

The storage area approach is a very easy one to determine the local boundary conditions for a closing structure. In this section this method will be explained in more detail. The relations between the global hydraulic boundary conditions, the geometry of the closure area and the local hydraulic boundary conditions will also be presented for the various closing methods.

The storage area approach schematises the water movement in the estuary without any friction or inertia. The equation for the discharge is:

$$Q = B \frac{dh_3}{dt} \quad (12.1)$$

In Figure 12-1 a storage area system is indicated.

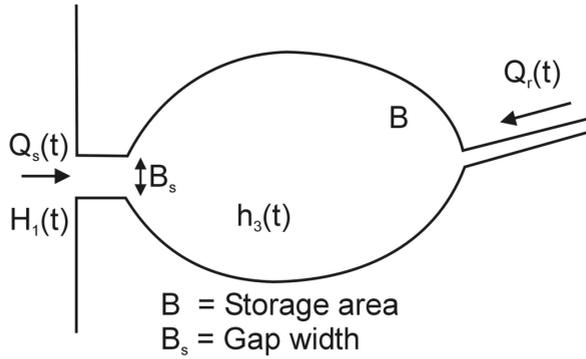


Figure 12-1 A storage area system.

This storage area system has the following boundary conditions:

- A river discharge as function of time [$Q_r(t)$], in which the inflow is defined as positive.
- A discharge through the closure gap [$Q_s(t)$], completely determined by the difference in energy-head upstream and water level downstream of the closure gap [$H_1(t)$ and $h_2(t)$] and the discharge properties of the closure gap, like width, sill height and discharge coefficient. As discussed in Section 5.2, two flow conditions may occur (sub-critical and critical), each with their own discharge relations.

Combining equation (12.1) with the discharge equations from Chapter 5 leads to:

$$\mu A_g \sqrt{2g(H_1 - h_2)} = B \frac{dh_3}{dt} - Q_r(t) \quad (12.2)$$

in which A_g is the cross-sectional area ($= h_2 \cdot B_s$) the entrance and

$$\begin{aligned} h_2 = h_3 & \quad \text{for } h_3 > \frac{2}{3} H_1 \\ h_2 = \frac{2}{3} H_1 & \quad \text{for } h_3 < \frac{2}{3} H_1 \end{aligned} \quad (12.3)$$

When the boundary conditions $H_1(t)$ and $Q_r(t)$ are known the water level inside the basin $h_3(t)$ can be calculated with the formula mentioned above. Also the velocities and the head-differences can be determined in this way.

From the last equation follows that only:

- the water level in the sea,
- the river discharge,
- the ratio between storage area and width of the closure gap,

- the sill height and
- the discharge coefficients of the gap

determine the local hydraulic boundary conditions.

When all parameters are known, the water levels and discharges can easily be computed as a function of time with equation (12.2). Simple software is available, but also a spreadsheet will do. One can start with any initial boundary condition, and after some tidal cycles a stable situation will occur. This provides the maximum flow velocities. When assuming the discharge coefficients m and n to be 1, the output is in fact the reference velocity. In empirical stability relations often not the real velocity, but the reference velocity is used.

In many cases the river discharge is negligible. Also one may combine the storage area and the gap with to one parameter, B/B_s . After doing so, the maximum velocity in the closure gap is only a function of the tidal amplitude, the depth of the closure gap and the parameter B/B_s .

This makes it possible for a given project (usually with a fixed given tidal amplitude) to construct a design graph where the velocities can be read directly. Figure 12-2 shows an example. The value on the vertical axis (d') is the water depth over the sill.

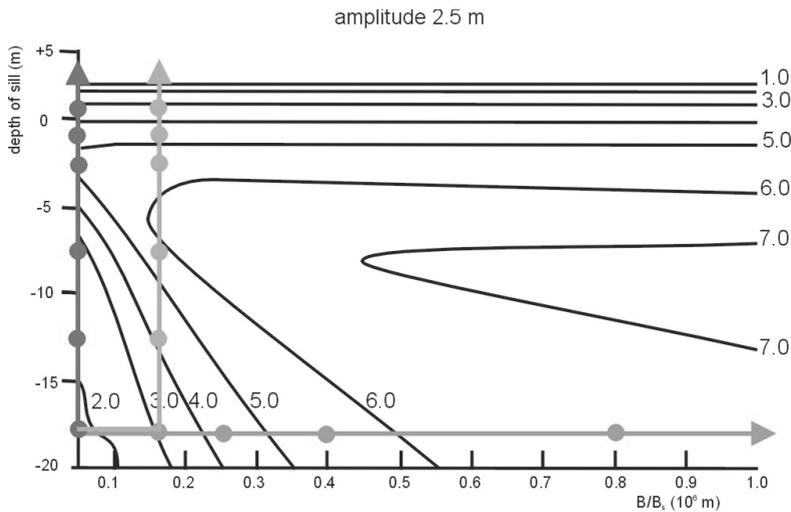


Figure 12-2 Example of a velocity design graph (lines indicate the max velocity (m/s) in the closure gap).

Such a graph can be used for the determination of the optimal closure method. A basic condition during a closure operation is that a certain value of u_0 should not be exceeded (to guarantee stone stability, for example). Given this maximum u_0 and the tidal amplitude a closing strategy can be developed. Such a strategy may consist of a horizontal closure, a vertical closure or various combinations. Horizontal closures

can be plotted in these figures as horizontal lines, vertical closures as vertical lines. Combined closures are a combination of horizontal and vertical lines.

This can be illustrated with the following example:

Given is an estuary with an area of $B = 200 \text{ km}^2$, an entrance width $B_s = 4000 \text{ m}$ and a depth of 17.5 m . The tidal difference at that location is 5 m , so the amplitude of the tide is 2.5 m . Investigated are the following strategies:

A: Horizontal closure

B: Vertical closure

C: Combined closure with first a horizontal closure until a gap-width of 1250 m , followed by a vertical closure of the remaining gap.

These three strategies can be plotted in a velocity design graph for an amplitude of 2.5 m . The result is presented in Figure 12-2. In order to determine the lines, first the value of B/S_s has to be determined for the original opening and for the moment when is shifted from a horizontal to a vertical closure.

For the original situation B/B_s is $200/4000 = 0.05 \cdot 10^6 \text{ m}$. For the shift from horizontal closure to vertical closure B/B_s is $200/1250 = 0.16 \cdot 10^6 \text{ m}$. Given these points, the lines can easily be drawn in the figure.

On the basis of such design graphs one can also plot the resulting velocities as a function of the percentage of closing. Such graphs are very useful in the further design steps. In the following box the previous example is also worked out in this way for a horizontal closure.

Horizontal closure: Depth is constant ($d = 17.5$)

Point	B_s (m)	B/B_s (10^6)	% Opening	% Closed	u_0
0	4000 m	0.05	100 %	0 %	1.0 m/s
1	1250 m	0.16	$1250/4000 = 30 \%$	70 %	3.1 m/s
2	800 m	0.25	$800/4000 = 20 \%$	80 %	4.4 m/s
3	500 m	0.40	$500/4000 = 12.5 \%$	87.5 %	5.7 m/s
4	250 m	0.80	$250/4000 = 6.25 \%$	93.75 %	6.8 m/s

For a vertical closure one may follow the same procedures.

In case of a vertical closure B/B_s is constant (0.05). Closure has to continue until a level of 2.5 m above MSL (otherwise the dam will be flooded during high tide). That means that for the computation of the percentage of closure one has to divide the height of the sill by the water depth + amplitude. So for point 1 (sill height is 5 m , $d' = 12.5 \text{ m}$, the percentage of closure is $5/(17.5 + 2.5) = 25 \%$. In this case the percentage of opening is calculated as $(12.5 + 2.5)/20 = 75 \%$. The reading of the U_0 has to be done at a value $d' = 12.5 \text{ m}$.

Data for vertical closure

point nr.	d'	% opening	% closure	u_0
0	17.5 m	100 %	0 %	1.0 m/s
1	12.5 m	15/20 = 75 %	5/20 = 25 %	1.4 m/s
2	7.5 m	10/20 = 50 %	10/20 = 50 %	2.5 m/s
3	2.5 m	5/20 = 25 %	15/20 = 75 %	5.3 m/s
4	1.5 m	4/20 = 20 %	16/20 = 80 %	5.0 m/s
5	-0.5 m	2/20 = 10 %	18/20 = 90 %	3.6 m/s

The combined closure is somewhat more complicated but follows the same lines. Below this is worked out in more detail:

Until point 1, the same line is followed as for the horizontal closure. At point 1, 70 % of the dam is closed. The remaining gap is closed vertically. For this vertical closure one may follow the same method as described above. However, one should take into account that this only covers the remaining 30 % of the gap.

For example, in point 2, $d' = 12.5$ m, the percentage of opening of the remaining gap is $(12.5 + 2.5) / (17.5 + 2.5) = 75$ %. However the total percentage of opening at that moment is only 75 % of 30 % = 22.5 %.

Point	d'	% opening	% closure	u_0
0	17.5	100 %	0 %	1.0 m/s
1	17.5	30 %	70 %	3.1 m/s
2	12.5	$0.3 \cdot (12.5+2.5)/20 = 22.5$ %	77.5 %	3.8 m/s
3	7.5	$0.3 \cdot (7.5+2.5)/20 = 15$ %	85 %	5.7 m/s
4	2.5	$0.3 \cdot (2.5+2.5)/20 = 7.5$ %	92.5 %	5.7 m/s
5	1.5	$0.3 \cdot (1.5+2.5)/20 = 6$ %	94 %	4.7 m/s
6	-0.5	$0.3 \cdot (-0.5+2.5)/20 = 3$ %	97 %	3.3 m/s

From Figure 12-2 one may conclude that the velocities remain relatively small or the vertical closure, and that especially the horizontal closure leads to excessive high velocities just before closing. This indicates that it may be useful to switch from horizontal to vertical at a certain moment and to continue with a vertical closure (= combined closure).

Figure 12-2 and Figure 12-3 can be used to investigate the necessary rock size the closure dam, but in case of a sand closure one can also use these graphs to determine whether a sand-closure is possible. For a sand closure the critical reference velocity is much lower - it is in the order of 2.5 m/s. In this example it is clear that a sand-closure is completely impossible - Every closure strategy will lead to a velocity which is higher than 2.5 m/s.

When a choice has been made for a closure with stones, then comparable figures have to be made. In these figures the maximum stone diameter has to be used instead of the reference velocity u_0 as a output parameter. On the basis of these plots one can select the optimal closure strategy.

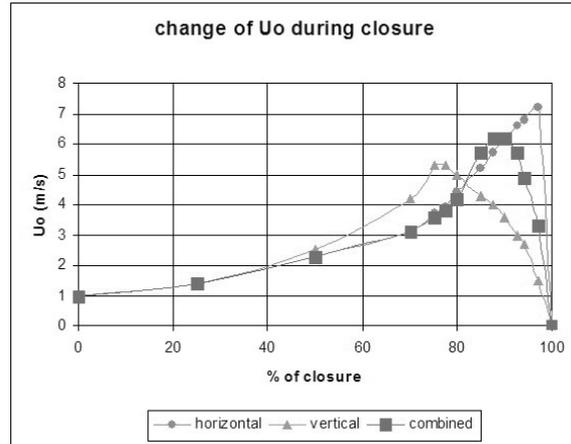


Figure 12-3 Maximum velocities in the closure gap (example).

The change of the reference velocity u_0 as a function of the percentage of closure for the various strategies in the previous example have been plotted in. It then follows from Figure 12-3 that the reference velocities are the highest with a complete horizontal closure. These velocities occur everywhere above remaining part of the closing gap. Because heavy current attack can only take place in the remaining closure gap, only a part of the bottom protection and the dam-heads will be attacked. With a complete vertical closure the reference velocities are limited and decrease when the dam is increased in height.

12.2 The design methodology for closures

In all cases first a preliminary design will be made. During the preliminary design the overall feasibility of the project is investigated. An other purpose of the preliminary design is to decide which type of closure is the most appropriate. This implies that there is no need to make very detailed calculations in this phase. Because usually the amount of money available in this phase is limited, money should not be spend on to detailed work (with the risk that this work is not used in a later stage).

Of course it is very wise to start already collecting data for the final design during this phase, especially data which require long term observations, like wave data, water level exceedance data, extreme current data, etc.

In the design of closures one may distinguish two steps:

- Step 1: Translation of the characteristics of the water movement and the geometry of the closure area to relevant local current parameters in the closure gap;
- Step 2: Translation of these local parameters to the constructional requirements of the final construction.

These two steps are illustrated in Figure 12-4.

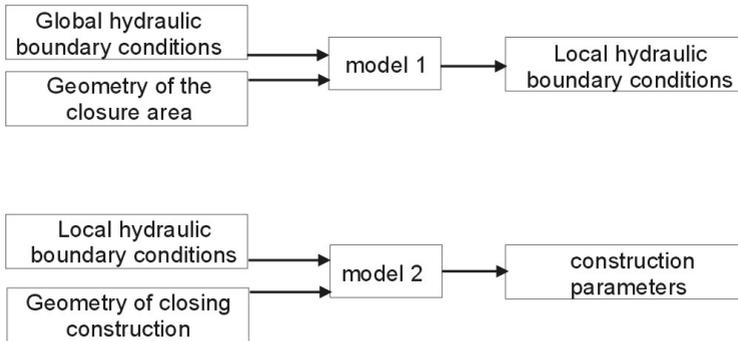


Figure 12-4 Steps in the design process.

In the Figure it is seen that with Model 1, the local hydraulic parameters are calculated on basis of large scale water movement and the geometry of the closure area. This approach is universal for all closure methods. In the second scheme is indicated that with Model 2, construction parameters (load on the construction) are calculated on the basis of the local hydraulic boundary conditions and the geometry of the closing construction. So, Model 2 is different for each type of closure.

In most cases (a short basin with only one entrance), one may apply the storage-area approach. In this case Model 1 is only solving the Mass Balance equation (as explained in Section 12.1 above). For more complex closures and in case of longer basins, the velocities should be calculated with a one-dimensional model. It is certainly not recommended to use two-dimensional models in the preliminary design.

In the next sections this will be worked out for simple cases (only one channel, short basin, semi-diurnal tide). In Appendix 6 a more complicated closure with two channels will be discussed.

12.3 Stone closures

Stability analysis of stones under attack of currents have indicated that a general relation is:

$$\Delta d_n = Au_0 \quad (12.4)$$

The value of A is different for all phases of the construction. It depends on many parameters, as is explained in SCHIERECK [2001].

After completion of the Deltaworks, the Ministry of Public works has evaluated all tests and prototype measurements done during the design and execution of these

works. These results were summarised in KONTER *ET AL.* [1992]. The conclusion was that one may define A as follows:

$$A = \frac{K^2}{\Psi C^2} \quad (12.5)$$

$$C = 18 \log \left(\frac{6h}{d_n} \right)$$

in which K is a correction and calibration factor. To be consistent the Chezy-value has to be determined using the above equation. For Ψ a value of 0.04 is used, because during closure operations some movement of the stones is certainly allowed.

Re-analysis of all data from prototypes, large scale model tests and small scale model tests revealed that in all cases the value of K is in the order of 1. Surprisingly this is the case for both vertical and horizontal closures.

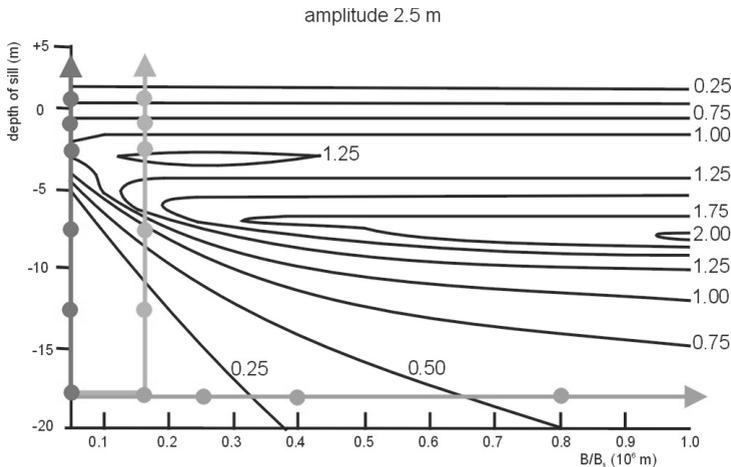


Figure 12-5 Example of a rock closure design graph (the lines indicate the value of Δd_n).

The consequence of this is that it is relatively easy to combine equation (12.5) with the velocity calculations as presented in Section 12.1. Doing so, one may construct design graphs where Δd_n is plotted a function of water depth and ratio between gap width and storage area. These graphs have an identical shape as the once presented in Section 12.1. The advantage of this approach is that one can directly read in a graph whether the available rock is sufficient. This results in graphs like Figure 12-5.

The following example will explain this:

Given is a closure in a channel of 4000 m wide, with a storage area of 200 km², a depth of 17.5 m and a tidal amplitude of 2.5 m. The question is now which stones are

required. In the figure the three closure strategies are indicated, as was also done in Chapter 5. Now an identical table can be made for the stone weights:

Point	Horizontal ($d'=17.5$)			Vertical				Combined			
	%close	u_0	d_{n50} cm	%close	d' m	u_0 m/s	d_{n50} cm	%close	d m	u_0 m/s	d_{n50} cm
0	0 %	1.0	<1	0 %	17.5	1.0	<1	0 %	17.5	1.0	<1
1	70 %	3.1	5	25 %	12.5	1.4	1	70 %	17.5	3.1	5
2	80 %	4.4	13	50 %	7.5	2.5	3	77 %	12.5	3.8	13
3	87 %	5.7	27	75 %	2.5	5.3	73	85 %	7.5	5.7	50
4	93 %	6.8	44	80 %	1.5	5.0	53	92 %	2.5	5.7	80
5				90 %	-0.5	3.6	20	94 %	1.5	4.7	53
6								97 %	-0.5	3.3	20

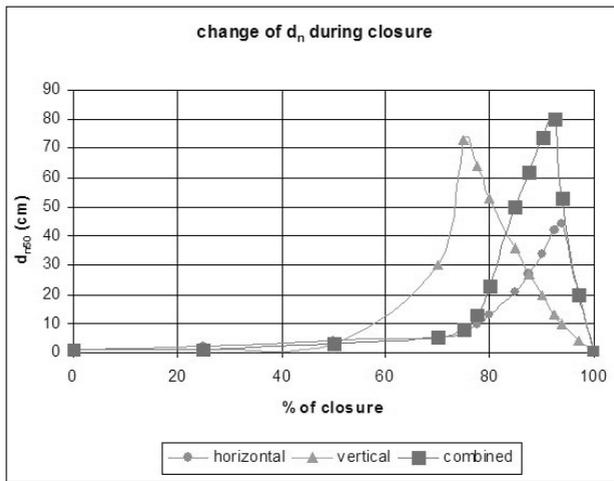


Figure 12-6 Required stone size as a function of percentage of closure.

From this example follows:

Strategy	Required d_n
Vertical	0.73 m
Combined	0.98 m
Horizontal	0.57 m

In this case, the horizontal closure is to be preferred. However, the difference in size with the vertical and the combined is small.

From Section 6.5 follows that in case of a horizontal closure a stone 300/1000 is necessary. The diameter will not exceed the 65 cm. The line for $\Delta d_n = 1$ coincides with the change from 300/1000 kg stones to the 1/3 ton stones. It is important not to use a strategy which crosses this line.

12.4 Caisson closures

When the stability guidelines of Chapter 9 are followed, a caisson will be stable during transport and sinking. It is always possible to design a stable caisson, so for the design of closures this is usually not a point which is relevant in the decision if a caisson closure is possible or not. Of course, the stability requirements may be as such that the size of the caisson becomes so large that this type of closure becomes too expensive.

Decisive for an alternative with caissons is usually if it is possible to place the caissons in the available tidal window. For placing the caisson velocities have to be rather low. For the real sinking procedure the velocities should be less than 0.30 m/s. In general the caissons will be parked in the neighbourhood of the closure gap on a place with low velocities. When the tide approaches slack water, and the velocities are low enough the caisson is sailed into the closure gap and sunken down.

In general placement during low water slack is preferable, because during low water slack the required sinking time is less. During high water slack sinking is also possible, for example when the water depth above the sill is too small or when every slack water has to be used to speed up the total closure period. The required time for the placement of a caisson depends on the time needed for every step in the process. An overview of the timing is given in Table 12-1.

In Figure 12-7 the different steps are indicated. One has to start connecting the caisson to the already placed ones at 30 minutes before slack water. However, before starting with this operation the velocity has to be less than 0.75 m/s. For sinking down approx. 13 min is required, the max. velocity for the sinking process is 0.30 m/s.

time before	velocity	slack water	above sill
- sailing in the caisson	-70 min		
- positioning caisson above sill	-55 min		
- connect caisson to already placed ones	-30 min		< 0.75 m/s
- sinking down of caisson	-15 min		< 0.30 m/s
- caisson on sill	- 5 min		
- moment of slack water	0 min		
- removal of wooden floating planks	+10 min		
- dumping of extra stone for ballast	+60 min		

Table 12-1 Minimum time required for caisson placement

For a sinusoidal M2-tide (12 hours and 20 minutes) one can deduce that the maximum velocity of the tide cannot be higher than $u = 2.5$ m/s. Otherwise the time available for sinking down is less than that which is required. In other words a caisson closure is only possible if:

$$u_0 < 2.5 / \mu \tag{12.6}$$

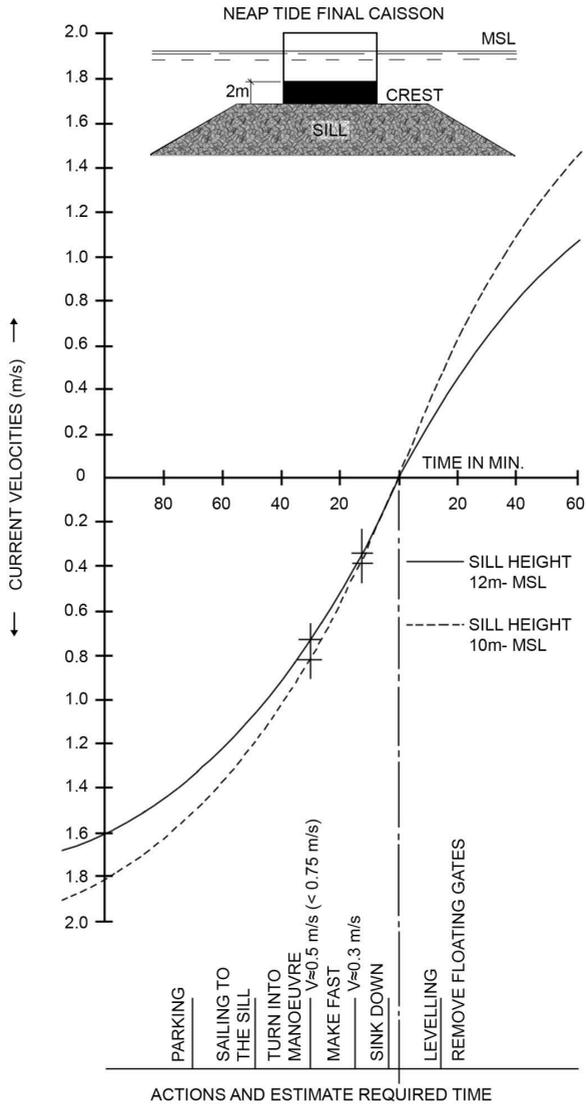


Figure 12-7 Water level variation during caisson closures.

Using the figures of Section 12.1 one can determine whether a caisson-closure is possible or not. One has to take into account that the value of μ varies between 0.6 and 1.0 (depends on sill level).

For example:

Assume a sill-level of -10 m and a μ of 0.8. The allowable reference velocity above the sill is $2.5 / 0.8 = 3.1$ m/s. Final sill-level $-10 + 2$ (for thickness of caisson-bottom) = -8 m.

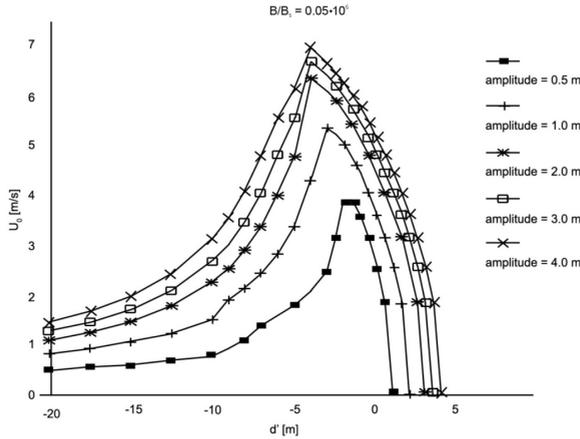


Figure 12-8 Maximum velocity as function of amplitude and depth for a given closure gap width.

From the graphs from Section 12.1 one can derive two other figures. Figure 12-8 is a design graph to be used for a given (final) gap width. This graph is for a final gap width with a B/B_s ratio of $0.05 \cdot 10^6$. From this figure one can derive that the last caisson can still be placed at a maximum tidal amplitude of 3 m (6 m difference between high and low water).

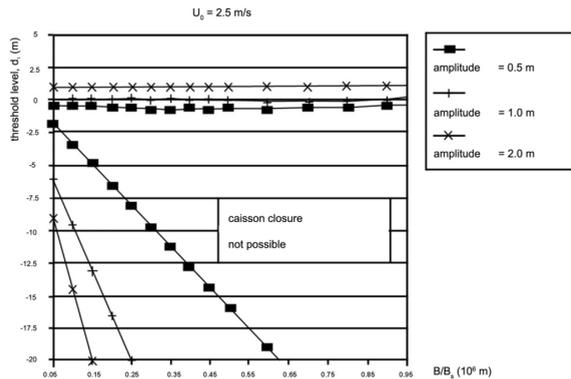


Figure 12-9 Placing boundary conditions for caissons.

Figure 12-9 shows the relation between tidal amplitude, water depth and gap width for the critical reference velocity ($u_0 = 2.5$) as function of the sill level and the parameter B/B_s . From this figure it follows that caisson closures are only possible for B/B_s -values of less than 100 km and for relatively deep tidal channels (sill level has to be lower than -10 m).

With the given tidal amplitude the maximum level of the storm-season gap can be derived.

For example:

Given a tidal amplitude of 1 m and $u_0 = u = 2.5\text{ m/s}$ ($\mu = 1.0$); $B/B_s = 0.05$.

From the figure it follows we need a sill level of -7.0 m. Because the bottom of the caisson also has to be accounted for, the top of the storm-season gap has to be designed on -9.0 m. This is the maximum level to allow a caisson-closure.

In the final design the level of the storm-season gap has to be optimised. In this optimisation the following aspects are important:

- From a viewpoint of scour-holes and construction costs of the sill, it can be desirable to lower the level of the sill.
- Placement of the last caisson becomes less critical with a lower sill level.
- The discharge coefficient becomes higher for a lower sill.
- Sailing-in the caissons is easier with a lower sill.
- With a lower sill the caissons become higher, and consequently more expensive.
- Higher caissons require thicker walls, which reduce the cross-sectional flow area.

In general a low sill level seems to be desirable. But each situation requires its own optimisation

12.5 Sand closure

Basically in a sand closure the amount of sand to be washed away should be less than the amount of sand to be pumped in.

Characteristic for a sand-closure is the movement and the loss of the construction material. A sand closure is based upon the simple fact that more sand is produced than gets lost. The sand loss occurs, depending on the current conditions, every hour and every day, even during average current through the closure gap.

In terms of “strength and load”, the “strength” of a sand closure is the production capacity and the “load” is the occurring loss. As long as the production is more than the loss, the gap becomes smaller and will the closure be successful.

The investigation into whether a (complete) sand closure is possible should first focus on the phase of the maximum losses. When in the neighbourhood of the closure gap sufficient sand production capacity can be realized to pass through this phase, then a sand closure is possible. A criteria for this is that the tidal average sand loss should be less than the average production. The big uncertainties, both in loss-computation as well as in production computation, have to be taken into account especially.

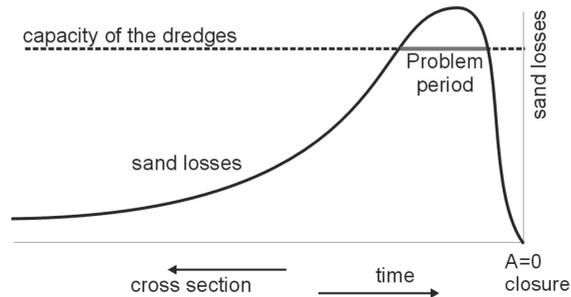


Figure 12-10 Dredge capacity vs. sand losses.

The losses as a function of the cross-sectional area of the closure gap are described by a curve with one maximum (see Figure 12-10). Nearly always the maximum can be found at a value of the cross-sectional area of 0 - 30 % of the original value of the cross-sectional area. In first instance the computation of losses can be limited to this size of the closure gap. In case the capacity of the dredges is less than the sand loss during this maximum, the gap will never be closed. The only solution in that case is to mobilize more dredging capacity. However, in case the problem period is very short (order couple of hours), one may try to overcome this period during slack water.

The maximum sand loss will not occur as the gap is nearly closed. The current velocities are then high, but the width of the eroding part of the closure gap is so small that the total sand-losses are quite limited. For the determination of the hydraulic boundary conditions, one is referred to Section 12.1; the sand-losses will be discussed below.

In general a sand closure is feasible until a maximum reference velocity of 2.0 to 2.5 m/s occurs. At higher velocities a sand closure is nearly impossible. The current velocities which occur are determined by the reference velocity u_0 and the discharge coefficient μ .

The magnitude of the discharge coefficient μ is determined both by the friction losses as by the slow-down losses in the closure gap. Because of the big size of the sand dam in the closure gap, friction losses play a relative important role. The discharge coefficient is therefore strongly influenced by the choice of the section over

which the head-loss is determined. The measured values of the discharge coefficient show a widespread. In the last, predominant phase of the closure the spreading of the coefficient decreases. For this phase it is advised to use a value of 0.9 as an reasonable upper limit.

The design rules given for a sand closure use the velocity u . This velocity u is the average velocity in the closure gap. In order to use the hydraulic boundary data from Section 12.1, it is necessary to convert the reference velocity to the average velocity, using

$$u_0 < u / \mu \quad (12.7)$$

If it is clear that a sand closure is feasible, one can start with dimensioning the sand closure. The process and the time required for the construction phase of the closure can be determined with

$$\frac{dV}{dt} = P - L \quad (12.8)$$

in which

- $V =$ volume of the dam (m^3)
- $P =$ sand production (m^3/s)
- $L =$ (tidal average) sand loss (m^3/s)
- $t =$ time (s)

The build-up of the dam body (parameter V) is discussed in Section 12.5.1, while the sand loss (parameter L) is discussed in Section 12.5.2.

12.5.1 Build up of a dam body

There are a number of methods of performing a sand closure. The choice of the execution method has a big influence of the design, because both the cross-sectional profile as well as the loss are influenced by the execution method. The possibilities for executing a closure (horizontal, vertical and combined), as sketched in Figure 2.1, are in principle also possible for sand closures. The specific executional aspects of sand closures will not be discussed here. One is referred is to CUR-157 [1992]

The horizontal build-up of the sand body is the standard method for realisation of a full closure. The vertical method is only applicable in the first phase of the closure, in which a sill is constructed. On the sill one may start with effecting of the closure. This can be done with sand, stones or caissons. Some aspects regarding the design will be discussed in more detail in the next sections.

The crest width in the horizontal construction method depends mainly on the number of production pipelines to be placed on the crest. In general there are 2 pipelines required per dredge (one for production while the other one is extended). In the table a relation is given between the number of dredges and the required crest width. From the table it is clear that one dredge with a high capacity is preferred above two smaller ones.

number of dredges	crest width of the dam
1	approx. 40 m
2	40 - 55 m
3	65 - 75 m
4	75 - 100 m

In a vertical construction method, the width of the crest is determined by the accuracy of the execution method. Also the crest has to be wide enough so that in the next phase the material (stones or caissons) can be placed on the crest without going “over the edge”.

Apart from the execution method there is a strong influence of the grain size of the sand to be used on the slopes of the dam and consequently on the volume of the dam. Measurements on executed works have indicated that for sand between 0.150 and 0.250 mm the following slopes can be found:

- under water slope 1:15 to 1:30
- intertidal area 1:50 to 1:100

For a vertical construction method (from pontoons) slopes are 1:10 to 1:25.

Especially for vertical construction methods the slope depends very much on the way the sand is placed (discharged just above the bottom, etc) and the hydraulic conditions.

12.5.2 Sand losses

Sand losses cannot be avoided, they are inherent to the construction method. The sand losses can be classified according to the underlying processes.

erosion:

Sand can get lost due to erosion. Both sand placed in the closure gap as well as the original bottom material can be transported by the currents outside the dam profile.

process losses:

Also during the placement of the sand, sand can get lost. Before the material gets the opportunity to settle down it is washed away outside the dam profile.

geotechnical processes:

Sand can flow outside the dam profile by sliding and liquefaction; both are geotechnical failures of the dam body. These type of failures frequently occur during the construction phase. However, most of the sand, transported by these processes will remain inside the final profile of the dam, and are thus not losses. The amount transported in this way outside the final profile is quite limited. Geotechnical sand losses are therefore neglected in the following computations.

The sand loss model describes the losses due to erosion. In this loss model a distinction is made between the main current and the vortex-street. Separate sediment transport formulas are used for both conditions. Consequently the sand loss formulas for horizontal and vertical closure are different.

The loss calculation is as follows:

First the dimensionless current parameter Ψ and the transport parameter Φ are defined as:

$$\Psi = \frac{u_0}{C^2 \Delta d_{50}}$$

$$C = 18 \log \left(\frac{12h}{k_s} \right) \quad (12.9)$$

$$\Phi = \frac{s}{\sqrt{g \Delta d_{50}^3}}$$

in which:

- u_0 = average current velocity (m/s)
- Δ = relative density of sand under water $[(\rho_s - \rho_0) / \rho]$ (-)
- d_{50} = grain size (m)
- C = Chezy-parameter ($m^{0.5}/s$)
- h = water depth (m)
- k_s = roughness (m)
- s = sediment transport (m^3/ms)

For sand the relative density Δ is approx. equal to 1.65. For all calculations the d_{50} of the local material is used. For sand closures a roughness value $k_s = 0.1$ is recommended. Because of the measured ripple heights, this value is very plausible.

Starting point for determining a regression between Ψ and Φ is the transport formula derived by Engelund-Hansen, because of the good results and the simple formulation.

with

$$\frac{\Phi g}{C^2} = a\Psi^b \quad (12.10)$$

In this equation a and b are regression coefficients.

For the calculation of the transport and the sediment loss the average current velocity along the axis is used:

$$u_0 = \frac{Q}{A_g} \quad (12.11)$$

in which

$$\begin{aligned} Q &= \text{discharge through the closure gap (m}^3\text{/s)} \\ A_g &= \text{cross-sectional profile (m}^2\text{)} \end{aligned}$$

Only in those cases where the current distribution strongly deviates from the uniform distribution, one has to use the local velocity. As a criterion for this one may use:

$$\frac{u_{d\max}}{u_0} > 1.2 \quad (12.12)$$

in which

$$u_{d\max} \quad \text{minimum depth-averaged current velocity (m/s)}$$

As a result of a regression analysis of executed sandy closures, it has been found that for vortex-streets a value $b = 1.75$. This means that transport is proportional to velocity to the power 3.5. For the main current a value $b = 2.5$ has been found. This means a power of 5. This is also the value which was determined by Engelund and Hansen for transport in rivers.

The empirical sand transport formula to be used in the design is obtained by correcting the loss in several cases with a calibration factor. The sand loss per meter of closure gap is then given with the following formula:

$$Loss = \frac{s}{1-n} = \frac{aC^2 \sqrt{\Delta d_{50}^3}}{(1-n)\sqrt{g}} \Psi^b \quad (12.13)$$

in which

n = porosity (-)

From curve fitting with the measured sediment transport the factor b can be found. This factor takes care of a good description of the sediment transport. The factor a is a calibration factor and depends on the relation between measured and computed losses.

The loss calculated with equation (12.13) is an instantaneous loss per meter length of the closure. In order to determine the total loss per unit of time one has to multiply this value with the width of the gap (B_s); however on both sides of the closure gap the transport (and consequently the loss) is somewhat different. Because of extra turbulence and flow contraction the calibration coefficients at the edges are different than in the central section.

The integration over the width of the gap gives the total loss per unit of time. It is important to realise that this loss is not constant, but a function of the tide and a function of the progress of the closing works.

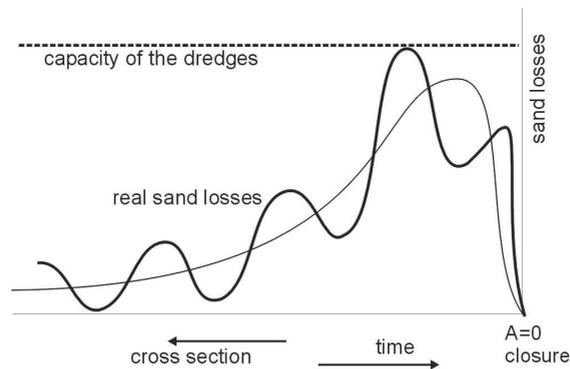


Figure 12-11 Dredge capacity vs. sand losses (including the effect of the tide).

From this calculation one can determine the required dredging capacity for the closure operation.

12.6 Cross-section of closure dams

In damming activities sometimes a choice can be made between a rather permeable initial closure dam or a profile provided with an impermeable (which in practice generally means with very low permeability) core or structure. The choice will

depend on various circumstances. The main consideration is that in a permeable profile the phreatic pressure decreases gradually but the flow in the full profile has to remain within limits to ensure micro stability. A watertight core leads to a high pressure difference over the core, which endangers macro stability. Moreover, locally along the boundary of the core or structure, the high pressure difference may lead to micro instability.

In several closure designs made for the Delta scheme in The Netherlands, an asphalt mat was used to protect the bottom. Although permeable mats composed of graded stone with an asphalt binder can be made, these mats were practically impermeable. Between the sea and the basin the phreatic level in the soil underneath the mat gradually diminished according to an extended flow net.

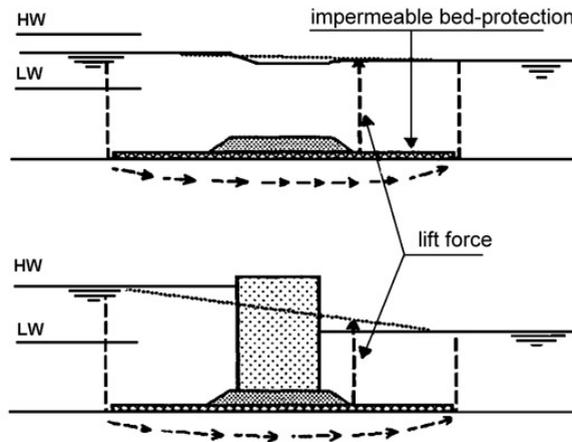


Figure 12-12 Uplift of impermeable bed protection.

Since the protected length (L) is large, i is small and so is u (Darcy). After construction of the dam profile in the centre of the protected area, the difference in the water level increases and thus u also increases. As shown in Figure 12-12, the phreatic level underneath the mat just downstream of the dam results in a lift force. The relatively thin mat, meant to prevent scour, suddenly gets another task: to counteract the upward pressure by its own weight. If lifting should occur, the mat might tear and the water would break out. The seepage length (L) would then be about half the original and in a two-dimensional case u would double. In practice there would be one location which would give way first and the flow would converge on this point. The flow would start a piping process and after that the whole dam profile would collapse. Consequently, the mat had to be much thicker than in cases in which a permeable mat has been designed.

In the design process the various combinations of water levels and soil parameters for the subsoil and the structural details have to be considered. Permeability coefficients have to be tested or assumed. Determining conditions in the various

construction phases have to be concluded. Next, groundwater flow nets can be drawn and analysed. In three-dimensional situations (beside a caisson or at the end of a sheet pile wall), the flow net is difficult to draw and great care must be given to these details. A problem may arise if infill of fines gradually plugs the soil. If this occurs the permeability changes and the pressures start building up. The reverse, (erosion of fines), may lead to higher flow velocities that worsen the erosion and finally lead to a scour pipe in which seepage changes into piping.

Generally, the most critical stage is the closure of the final gap. This is a situation in which a bottom protection against scour has been laid on which in a short period of time an initial dam or structure is made. Differential water levels attack the freshly-made profile. With caissons in particular, the stability of the foundation bed and of the side connections is important.

A typical example of piping actually occurred during the closure of the River Feni in Bangladesh. During preparatory operations, stockpiles of jute bags filled with clay had been stored along the initial closure alignment. These stockpiles had been built on geotextile sheets strengthened with bamboo grating. During the closure operation sufficient bags would be carried from the stockpiles into the longitudinal initial dam profile to block the flow. Immediately afterwards this profile would be strengthened by a heavy clay profile placed behind it. The procedure required the removal of the bamboo gratings but time did not allow the complete removal and the clay profile covered several remnants of these bamboo canes. During high water, the groundwater then found its way underneath the clay along these bamboo canes and several wells gave artesian water. The problem was solved by dumping some extra clay on top of the wells. This appeared to be sufficient to block the flow and prevent the escalation of the piping.

If piping occurs, several measures can be taken to prevent escalation. The best measure is to block the inflow on the upstream side. However, although the outflow point is known where the water enters is generally unknown. The next measure is to ballast the well as in the above example. But sometimes, water may break out time and again. In such cases, the well can be covered by a permeable sheet that is sufficiently ballasted with cobbles. Owing to the permeability, the upward pressure will remain low and further ballasting is possible. The flow will carry soil particles that become stuck in the sheet and gradually the flow is blocked while the pressure increases. Ballast and soil on top will ultimately stop the flow. In shallow water and above water, a method is to construct a ring-shaped wall of sufficient height around the well. Within this ring the water level will rise until the high level at the entrance of the pipe is reached. Then the flow stops and any sort of material can be dropped onto the well and block it.

12.7 Final remarks

The calculations and considerations relating to the closure options for the above example demonstrate that the change in conditions during the closure operations depends on the method adopted. Furthermore, it corroborates the statement, made in the preface, that there is no single correct solution to this design problem. Without further details about availability and costs of materials and equipment no conclusions can be drawn on the basis of the above calculations.

Moreover, various other considerations may influence the decisions, such as: "Is it possible to build a large dry-dock for the construction, immersion and float-out of the caissons? Is there any social-economic reason why labour intensive low-level technology is preferred above a high-skilled approach? How skilled is the available labour force and in what sort of operations are they experienced? Are there any restrictions on the import and use of equipment or materials from outside the country and what is the taxation situation?"

But even the technical arguments have not all been considered fully. What about operational conditions and time periods? Are there seasonal changes in sea level or tidal amplitudes, periods with storm surges or cyclones, monsoons restricting operations because of waves and swell, limiting the execution of part of the works to specific work-windows? What happens when, due to unforeseen set-backs, the critical operation will exceed that time window?

No attention has yet been paid to the impact of extreme conditions on the design. Is sufficient data available to make an analysis of the probability of the occurrence of such an event? In the considerations included in the above example, not even the tide has been varied for springs and neaps.

Some operations are more vulnerable to extreme conditions than others and so consequently the measures to be incorporated in the design may take different proportions of the costs. These have to be included in the assessment of the various options in order to make a proper choice. In the event of a failure, the options for repair depend largely on the closure method used. These considerations are difficult or sometimes impossible to assess, and yet it is all part of the design process.

It is impossible to provide a single recipe for the closure of all dams. The solution is always site specific. The objective is to illustrate the basic technical problems likely to be encountered and the application of scientific principles to their solution, substantiated by the wealth of practical experience involved.

13 CONSTRUCTION METHODS FOR GRANULAR MATERIAL

Chapter 13 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.

13.1 Introduction

Although for a first approach theoretical and analytical considerations are indispensable to designers of closure dams and 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 (13.3), the provision and handling of quarry stone (13.5), the use of rolling and floating equipment in various conditions (13.6) and finally some very specific techniques and ancillary equipment (13.7) are given in the next sections. Some details of construction equipment are given in Appendix 7.

13.2 Scour prevention by mattresses

Every closure of a watercourse leads to a situation in which the flow accelerates, circles around dam heads or crosses over materials with different hydraulic

roughness. Each of these results in 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:

- i. the flow velocity increases in course of time
- ii. the flow distribution over the vertical changes
- iii. the flow is not saturated with sediment
- iv. the turbulence intensity increases

These aspects occur in closure processes for instance:

- when diminishing the gaps profile (item i)
- at the end of a stone protection as consequence of change in roughness (item ii)
- due to reduction of the discharge quantity in the approach gully (item iii)
- around dam heads, structures and obstacles (item iv)

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 13-1).

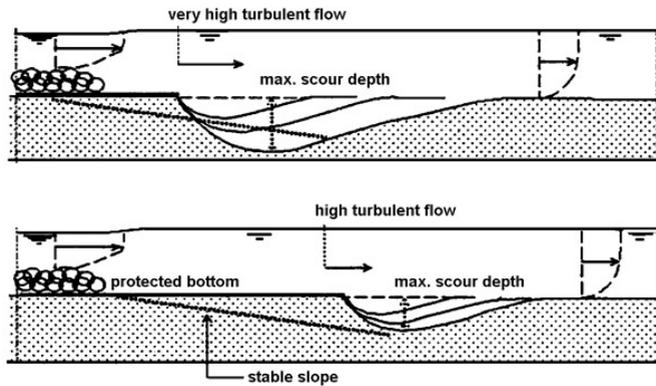


Figure 13-1 Development of scour hole.

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. 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).

13.3 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)

13.4 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

13.5 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 armour stone. 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 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 13-2.

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.



Figure 13-2 Stockpile area with wide gradings.

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.

13.6 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 transshipment 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 armour-stone 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.

13.6.1 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 13-3). A picture of such breakwater under construction is given in Figure 13-4.

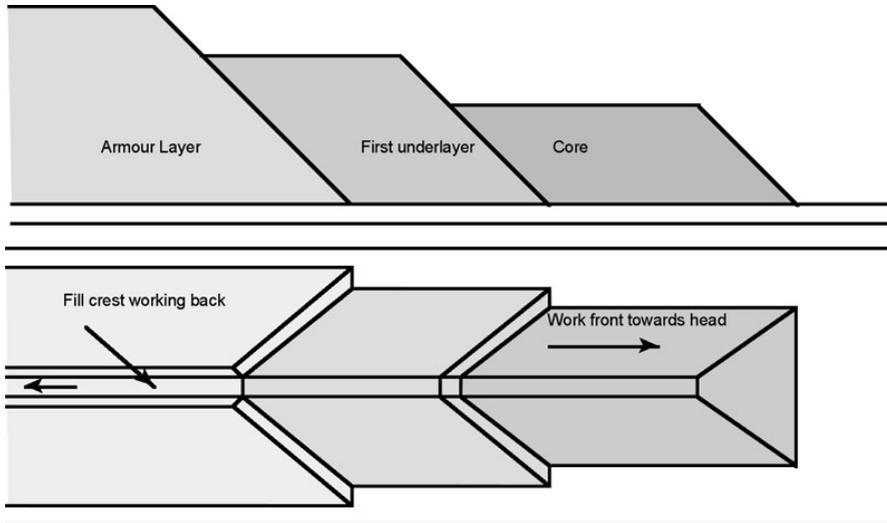


Figure 13-3 Subsequent work fronts.

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 then 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 powerforks 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



Figure 13-4 A breakwater under construction.

.Guidance of achievable vertical placing tolerances with land-based rolling equipment (cranes) is presented in Table A13-1.

Depth of placing relative to LW	Bulk-placed armourstone		Armour layers and individually placed stones, with $M_{cm} > 300$ kg	
	$M_{cm} < 300$ kg	$M_{cm} > 300$ kg (not armour)		$M_{cm} < 300$ kg
Above LW = dry	± 0.2 m	+0.4 m to -0.2 m	$\pm 0.3D_{n50}$	+0.35 to $-0.25D_{n50}$
0 to -5 m	+0.5 m to -0.3m	+0.8 m to -0.3 m	$\pm 0.5D_{n50}$	+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 A13-1 Practical, achievable vertical placing tolerances with land based equipment [Rock Manual, 2007].

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 13-5). 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.



Figure 13-5 Truck waiting on the breakwater.



Figure 13-6 Use of cheap local equipment.

The major advantage of the dry construction method is the potential use of cheap local equipment (see Figure 13-6) 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 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.

13.6.2 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.

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 A13-2 Practical, achievable vertical placing tolerances with floating equipment [Rock Manual, 2007].

Guidance of achievable vertical placing tolerances with floating equipment (side-dumping vessels and crane barges) is presented Table A13-2.

For closure dams, floating cranes can be used for horizontal closure by positioning them at the downstream side of the closure gap, in the area protected from the flow behind the dam head (see 13-7).

For vertical closure, floating cranes are not very adequate. Dump barges are the best tools but they can only provide the lower portion of the dam profile. The top section, crossing the waterline, has to be made in a different manner. This is either a continuation of the horizontal system, making the total closure method a “combined method” (see Section 13.6.3). or a vertical continuation, using very special equipment like a cableway or a bridge (see 13.7.2) Dumping creates a layered build-up. Depending on the way the dumping is executed, this results in a triangular or a trapezoidal profile (Figure 13-8 and Figure 13-9).



Figure 13-7 Positioning the floating cranes.

In closure dam design, the creation of a sill by dump vessels not only has a financial advantage but also the possibility to create extra stability in the flow attack. Instead of tipping from a centre line, vessels can dump horizontally layer by layer in a trapezium-shaped profile. The side slopes thus depend on the individual dimension of every successive layer (Figure 13-9). 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.

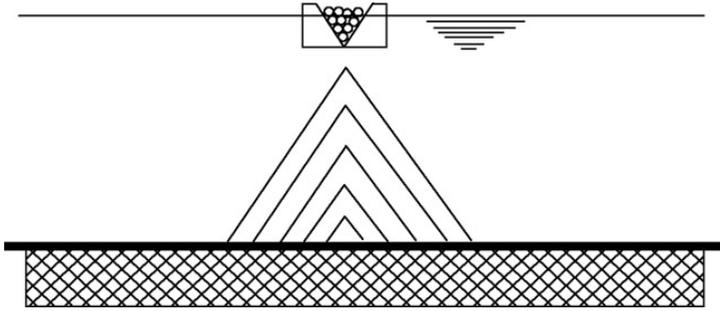


Figure 13-8 Profiles in a line dump.

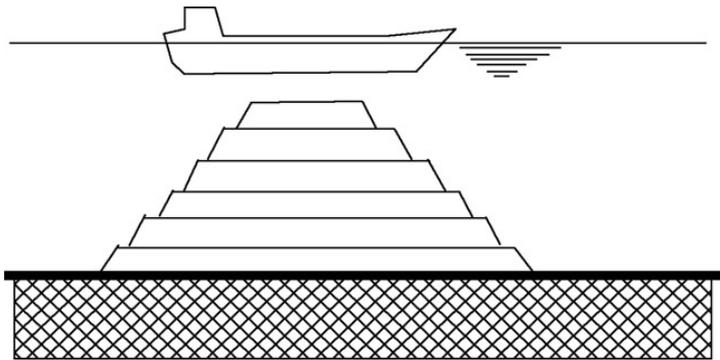


Figure 13-9 Profiles in horizontal layers.

13.6.3 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 13-10).

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.

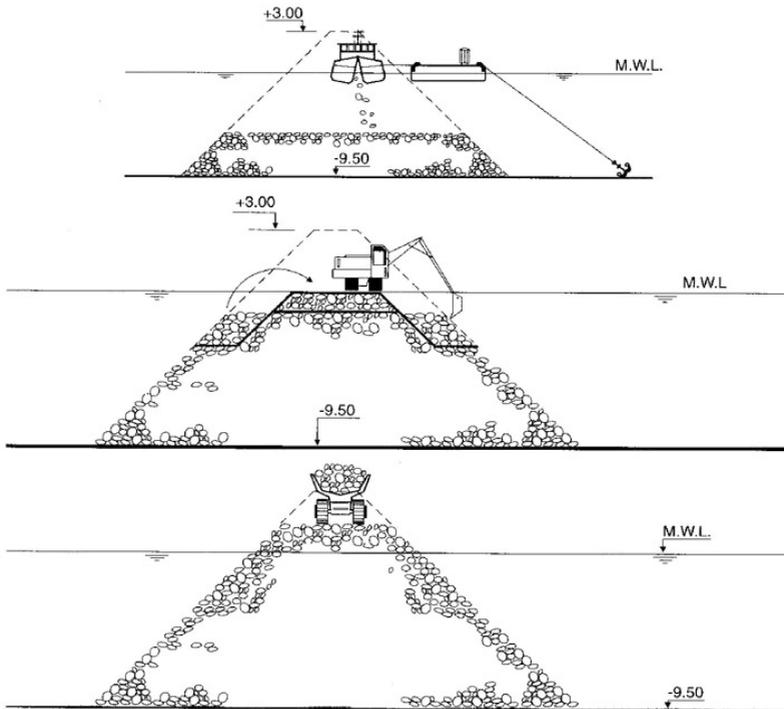


Figure 13-10 Construction of a closure dam by successively using waterborne equipment, grab and dump truck.

13.7 Very specific techniques and ancillary equipment

13.7.1 Closure by hydraulic filling with sand only

In a number of cases, a tidal basin can be closed just by pumping sand into the gap. The principle is simple. As long as more sand is pumped into the closure gap than the flow erodes away, the gap narrows. Due to the development of dredgers with very high capacities (5000 to 8000 m³ sand per hour), this has become a realistic option. Bulldozers are needed to spread the sand-water mixture over the fill and shape the desired profile of the dam. In addition, they prevent the erosion of gullies on the fill slope and compact the deposited sand. The equipment, used to control the fill process has improved likewise.

Thus the main question is how much capacity is needed. A very high capacity can be attained by using many delivery lines from various dredgers. However, to keep the fill under control, for these operations every delivery line needs a specific width.

More capacity means a much wider fill-profile, which does not improve progress. So there is a practical limit to the supply. Of course, the gap can be approached from two sides.

A sand closure is a horizontal closure, with a progressively narrowing gap, in which the flow increases until the very last gap can be blocked. The flow in the gap has a sand transporting capacity that is related to the flow velocity to the power of the order of four. This means that when the flow increases from 2 m/s to 2.5 m/s, the transport of sand is multiplied by a factor 2.5 and when it reaches 3 m/s it is multiplied by a factor 5.

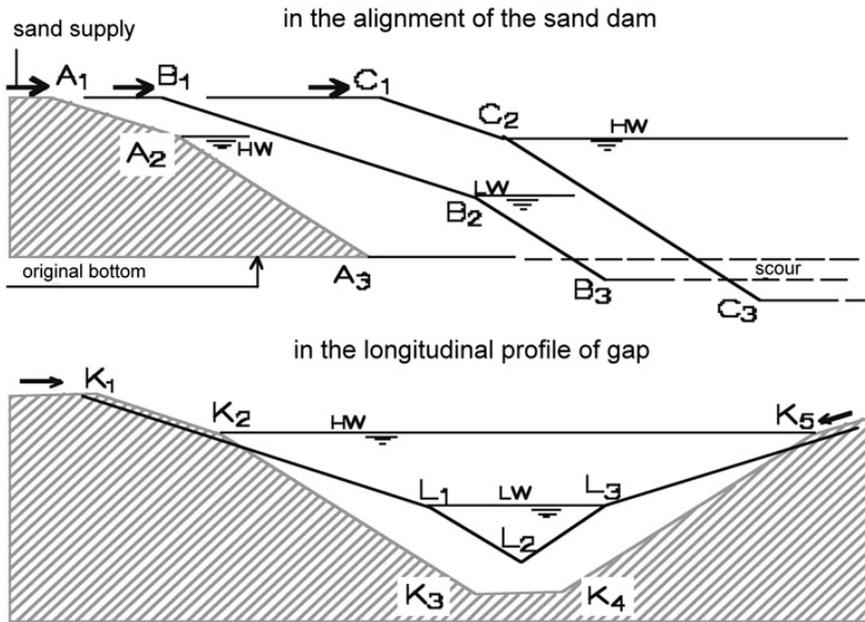


Figure 13-11 Closure by pumping sand.

The normal process of building a sand fill dam is as follows. A sand water mixture is pumped by the dredger via a delivery line to the fill to a sufficient height above HW (point A1 in the figure). The mixture runs down the slope to the waterline (A2) and into the water where the sand settles. The sand creates a slope above water (A1-A2) which is much less steep than the submerged slope (A2-A3). In the ebb period, going from HW to LW, the progression, as shown in the figure, is from A1-A2-A3 to B1-B2-B3. Progress seems small, as the delivery point moves very little (A1-B1).

All the sand goes into the toe circle. During flood however, the line B1-B2-B3 shifts to C1-C2-C3. The nett progress per tidal cycle is A1-C1. Scour will erode the original bottom in front of the dam and enlarge the profile to be made. In addition, part of the supplied sand will also be taken by the flow and carried outside the

alignment area. The further the dam extends the higher the rate of flow in front of it and the smaller the nett progression per tidal cycle.

When hardly any progress is being made, the final blocking has to be enforced. This is a special operation, which starts at the moment of HW, which is shown in the figure by the line K1-K2-K3-K4-K5 for two fills approaching the gap from both sides. Progress during the ebb may even seem to be negative, as the line K1-K2 retreats but the size of the gap at LW is very small (L1-L2-L3). Then, just before slack water, the profile has to be blocked. This requires the temporary interruption of hydraulic transport by taking recourse to earth moving plant, thus obviating the erosive action of the hydraulic transport by water. Bulldozers and cranes will have to shift as much sand as possible into this LW-gap to shape a tiny ridge in the triangular profile (L1-L2-L3). This sand is taken from the slope above water (K2-L1). The ridge being ready, the flow is blocked and all the sand supplied by resumed pumping will settle in the profile. The ridge, protruding above LW has to be heightened and widened to stay ahead of the rising outer water. The volume required increases tremendously with the rising level, as the length of the ridge also increases. To create a stable profile over the full length (K2-K5), that is able to stand the head difference of the full tidal range, in a few hours, requires a very skilled fill procedure and a sufficient sand delivery capacity as well.

Thus the principal questions are; how large a gap can be closed in the last tidal cycle and what will the sand transporting capacity of the flow in that gap be. The last operation in the gap takes place in one tidal cycle, so for a semi-diurnal tide this is about twelve hours. The twelve hours before that is the last tidal cycle in the normal fill procedure in which some narrowing progress is still to be made. Consequently, the possibility of closing is determined by the operations during the last day. The capacity required thus is determined by:

- the normal process followed to attain the size of gap that can be closed in one tidal cycle
- the ebb-phase during which the tiny ridge can be shaped in the LW-slack period
- the flood-phase in which stable profile must be built before the next HW

With that capacity laid down, the progress in the days or weeks before the closure can be calculated by phase-wise determination of the nett progress per cycle. Summing-up these phases gives a reasonable approximation of the total time needed for the construction of the dam.

Several sand closures have been constructed. In these cases it appeared that, depending on the grading of the sand, the maximum flow velocity that could be accommodated was in the order of 2 to 2.5 m/s. According to the flow formulae in gaps, these velocities occur for a head difference of about 0.30 m, for rounded sand

dam heads. The gap size for which this boundary condition exists can be calculated by using a mathematical model of the closure procedure.

For instance, in the mathematical example "case 1" of Section 16.2 (Figure 16-5) the flow reaches about 6 m/s in the final stages (4 and 5), when the head difference is slightly more than half of the tidal range (1.5 m). The 2.5 m/s is already reached at stage 2, which is for a gap dimension of 4000 m². This gap size is far too much for the final day's operation. Nevertheless, the sand-closure of the "Krammer" (in the rear of the Eastern Scheldt basin) in 1987 closed a basin of about 55 km², which had a tidal range of 2.70 m. However, by that time, the storm surge barrier in the entrance of the estuary was operational and the tidal range was artificially reduced to 0.60 m during the closure. Thus, to enable the closure flow velocities were kept under 2.5 m/s. This shows that if the tidal range is smaller than 0.6 m, even very large basins can be closed by pumping sand.

Basins with larger tidal ranges can only be closed by sand pumping if the storage area of the basin is much smaller. An example of that is the sand closure of the Wohrdener Loch in northern Germany near Meldorf in 1978 (Figure 13-12). The tidal range during the neap tide on closure day was 3.20 m. The storage area was 10 km², the grading of the sand about 350 μ m, the total installed dredge capacity 8000 m³ per hour and 14 bulldozers and 8 hydraulic excavating machines were busy at the fills. The length of the gap at the waterline during HW (K2-K5) (Figure 13-11) in that case was 120 m. The capacity to strengthen the ridge during the flood phase appeared to be the determining factor.



Figure 13-12 Final closure using sand only.

A few conditions determine the possibility to close with sand only:

- the tidal range or the storage area of the basin has to be sufficiently small

- large quantities of good quality sand must be available nearby
- high-capacity dredgers have to be used
- A well-organized fill-procedure using cranes, backhoes and bulldozers is required

If the original bottom in the gap has a resistance against erosion comparable to the sand used for closing, protection against scour is not relevant. Scour is acceptable unless it endangers stability of structures in the proximity. A considerable volume of sand is carried by the flow outside the desired profile. This reduces progress but is part of the method. The lost sand is not considered a permanent loss. Actually, instead of providing an expensive bottom protection, scour is compensated by using an extra quantity of sand. The many machines operating at the fill require that there is good bearing subsoil. Sand closures using very fine or silty fine sand are impracticable or impossible. Therefore very fine or silty fine sand is not suitable for the construction of sand closures.

13.7.2 Use of a temporary bridge or a cable way

When closing vertically, the flow pattern will be completely different. Building up layer after layer, the flow will reach a stage in which critical flow starts. Stability of the profile depends on its shape, which in turn depends on the way the dam is built.

Dump vessels or dump trucks cannot create the layers to bring the sill above the waterline in the vertical closure system. Therefore different equipment is required. The most common method is to use a temporary bridge or a cableway. Of course this equipment has to be installed as the first part of the full closure operation. Generally, the method requires a high initial investment but it may be an alternative to a caisson closure in cases where the provision of a caisson construction dock is expensive. The advantage of vertical closure over combined or horizontal closure, which is obvious in the classical system of sinking mattresses, is much less determining when using modern equipment.

However, when a cable or a bridge is installed, it seems obvious that this should be used for the entire profile of the closure dam. This means that the profile is built-up in the line system (see Figure 13-8 and Figure 13-13). Using the return cable or by tipping at both sides of the bridge also makes a two-line dump possible, but these two tops are relatively close. The disadvantages of this type of operation are:

- Steep side slopes with a risk of sudden instabilities in the build-up. This is specifically true in the final stage for the downstream slope owing to seepage flow through and overflow over the top.
- Irregular very sharp crested top, which determines the critical flow.
- A low production rate and a high in-situ unit cost (manufacturing and handling)
- The largest portion of the profile must be placed in the final stage of closure

The advantage is that any place in the alignment can be reached at any moment, for instance for repair of an unstable section.

Nevertheless, it is always worthwhile to consider the construction of the lower portion by dump vessels.



Figure 13-13 Closure with a cable way.

13.8 Minimizing risks during construction

Coastal, maritime and estuarine construction is particularly hazardous because of the hostile and sometimes unpredictable nature of the environment. The subject of risk management has received increased attention from industry, academia and government, as it can help to:

- identify and question the assumptions that affect the success of the project
- concentrate the effort into controlling the risk through risk prioritisation
- balance the costs and benefits of the risk controlling measures
- protect the health and safety of the operatives and the public.

Important risk and hazard issues that relate to protecting the work during execution are (see also the Rock Manual [2007]):

- Stability of partly completed works and temporary works
- Methods of estimating site conditions for tender purposes, for real-time forecasting and site control during construction (marine environment, such as waves,

currents and wind, are often very volatile and unpredictable and may have a significant impact on the planning and execution of the works)

- Operational limits of plant
- Temporary works and partly completed permanent works
- Seabed and riverbed changes
- Timing of works
- Uncertainty about groundwater conditions
- Variations in quarry geology

Unreliable armourstone supply can create a high risk of escalations in time and costs of construction. The key issue is the yield of the quarry (eg the percentage of armourstone above 1 t). Since the quarry's products form a major part of the project cost, any change in the assumed yield value has a major effect on that cost and on the completion time.

Of course, much attention has to be paid as well to risk and safety aspects for personnel operators and the public.

As mentioned above, attention should be paid to the stability of partly completed works and temporary works, for instance the cross section during construction. For closure dams, this may be evident, because the closure dam itself is a temporary structure that will be replaced and protected by the final structure as soon as possible. Nevertheless, one must take into account that the closure dam must withstand the hydraulic loads that are expected during the period of exposure.

In a similar way, one must consider the various construction phases of a breakwater. Considering Figure 13-14, it will be clear that the work front cannot withstand the design storm. Therefore, one must consider what risks are threatening the structure during the construction phases and find ways to reduce these risks. Common methods are to:

- Select a specific construction period
- Reduce the exposed length of the vulnerable part of the structure
- Construct protective bunds

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 13-14 (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 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 13-14, the traditional method is compared with the alternative.

It is clear that the alternative method (on the right hand side of Figure 13-14) 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.

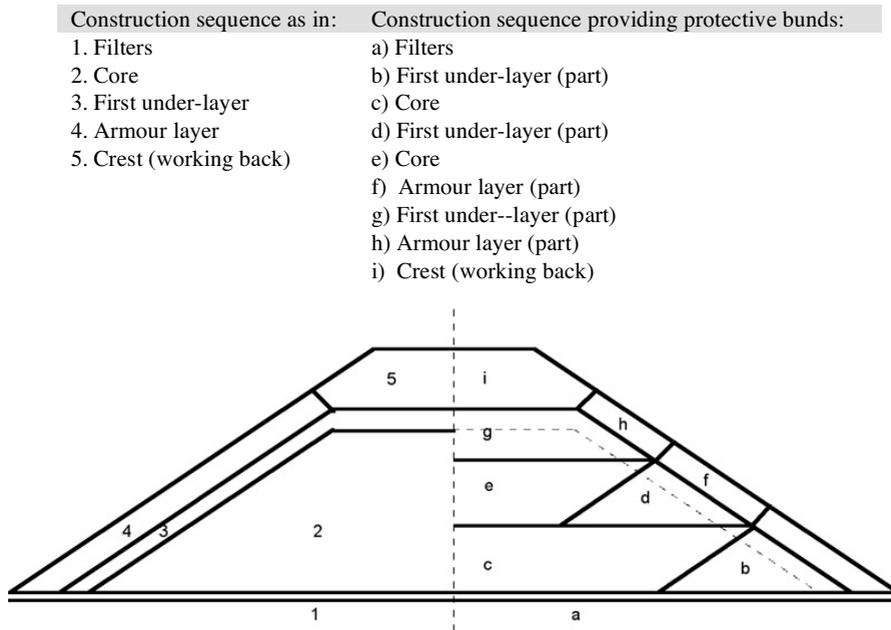


Figure 13-14 Varying construction sequence.

13.9 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 13-15):

- highest points
- spherical foot staff
- conventional staff.

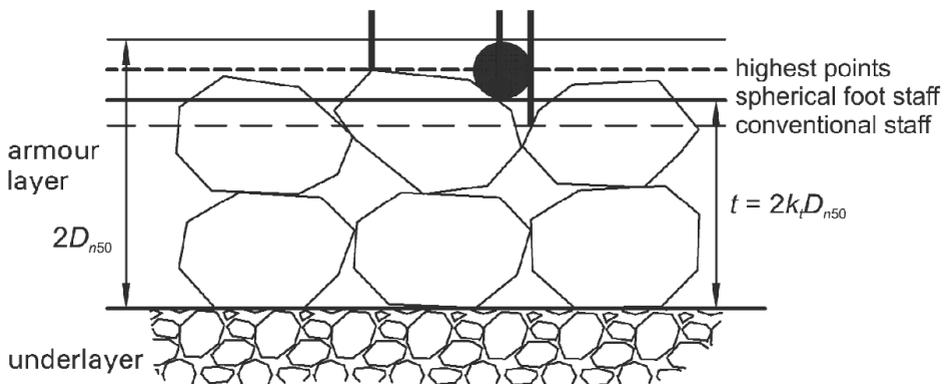


Figure 13-15 Effect of surveying methods on layer thickness for double armour layer.

As can be seen in Figure 13-15, 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.

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

Available systems

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 echosounders or side-scan sonar.

Echosounders 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, d_t (s), the water depth, h (m), can be calculated as:

$$h = 0.5v_s d_t$$

There are two main echosounder systems:

- single-beam
- multi-beam.

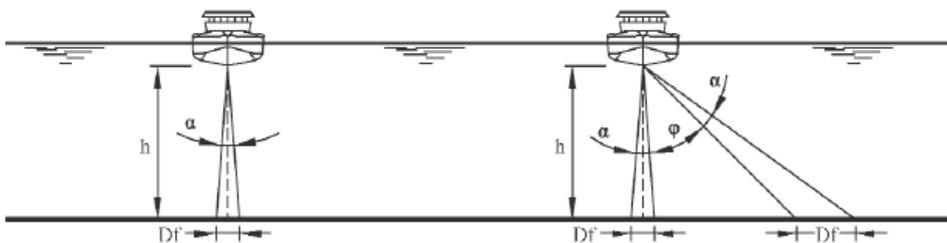


Figure 13-16 Footprint of single beam and multi-beam echosounders [Rotterdam PWED, 2001].

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 13-16). The diameter of the footprint, D_f (m), depends on the beam angle, α (deg), and the water depth, h (m), according to:

$$D_f = 2h \tan(0.5\alpha)$$

The beam angle (α) differs according to the system frequency but is in the range of 2.5 - 3.0°.

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 13-16). 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.

The diameter, D_f (m), of the oval-shaped footprint is given below:

$$D_f = 2h[\tan(\rho + 0.5\alpha) - \tan(\rho)]$$

where ρ 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 ρ can be varied in the range of -75° to $+75^\circ$ in steps of $\alpha = 0.5^\circ$ to 1.5° . 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 echosounding 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 echosounder will deliver continuous profiles these are still separated by the

distance between the survey lines. Measurements made with a multi-beam echosounder 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 echosounder)
- 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.

14 CONSTRUCTION METHODS FOR MONOLITHIC STRUCTURES

Like Chapter 13, this chapter gives a description of practical problems encountered when using caissons or building monolithic breakwaters. It should be read as an example rather than be considered as absolute facts.

14.1 Introduction

Both, in breakwater constructions and in closure dams, 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. For closures, the large elements are generally of the caisson-type.

Caissons have been widely used in both closure dams and monolithic breakwaters. Although there may be some structural and operational differences, the basic principles are identical. The structural differences may be due to the different load pattern or to the fact that in closure dams, the caissons are designed to allow discharge until gates are closed. The operational differences may be due to the fact that in closure dams the current is usually the main constraint during the placement of the caissons, while waves are the main constraint for breakwaters.

Because of the small differences, here no distinction is made when discussing the constructional aspects of caissons for the two types of application. The reader is referred to Chapter 9 for a discussion of wave loads on monolithic breakwaters.

14.1.1 Caissons, closed or provided with sluice gates

Caissons are used in breakwater construction in order to create a near vertical or vertical wall, which minimizes the cross-sectional profile. This is most effective in

deep water and is a choice requiring a small but expensive volume. The resulting reflection of the waves is the main disadvantage. The design will always consist of a rather large number of identical caissons, placed in line. The length of each depends on the handling and stability during transport and sinking and only very seldom on the desire to minimize the number of sinking operations.

In closure designs caissons are large, artificially made structures or vessels used to block a final closure gap in one major effort or in a minimum of major steps. Therefore, they are designed to be as large as is feasible from a constructional point of view. In emergencies existing ships and pontoons or the like have been used, sometimes after adaptation to fit the gap dimensions. In normal circumstances, caissons are specifically designed for the purpose. Generally they are made of concrete, have a box shape and are self-floating. Three different typical systems can be distinguished:

- The final gap is closed in a single operation by placing one or several caissons simultaneously
- Several identical units, which together fit into the gap, are made. They are placed one after the other in a period of several days, during which for each successive caisson the positioning conditions will be more severe
- Several units are used, a number (or all) of which are provided with sluice gates. Each unit is placed with its gates closed, but after stabilizing of the caisson, the gates are opened. As soon as all caissons are rigidly in place, all the gates are closed at a suitable moment.

Which system is used depends on circumstances and conditions. They are progressively more expensive.

In this chapter, the following subjects will be discussed:

- monolithic breakwaters assembled from small units
- monolithic breakwaters consisting of large units constructed in-situ
- monolithic breakwaters consisting of prefabricated large units, floating and non floating
- monolithic breakwaters consisting of prefabricated large floating units i.e. caissons

Specifically for the caissons, the identical aspects of the use in breakwater and closure design are given in detail. The construction, the transport, the foundation bed and abutments, the floating stability and sinking, and the stability after placement are described. A few aspects typical of the use of caissons in one or other of the applications are then discussed.

14.2 Monolithic breakwaters

14.2.1 Monolithic breakwaters constructed by assembling small units

The oldest vertical wall breakwaters were composed of rectangular blocks of natural stone. 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 14-1 and Figure 14-2)

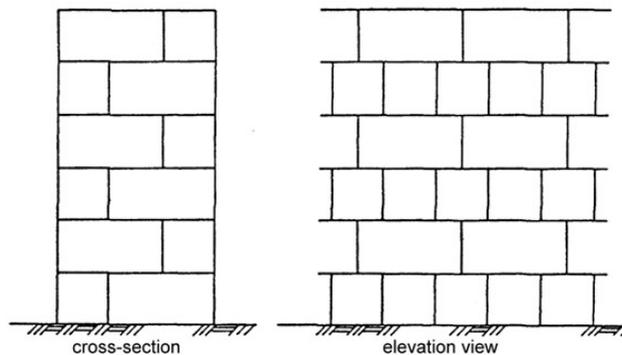


Figure 14-1 Typical block wall.

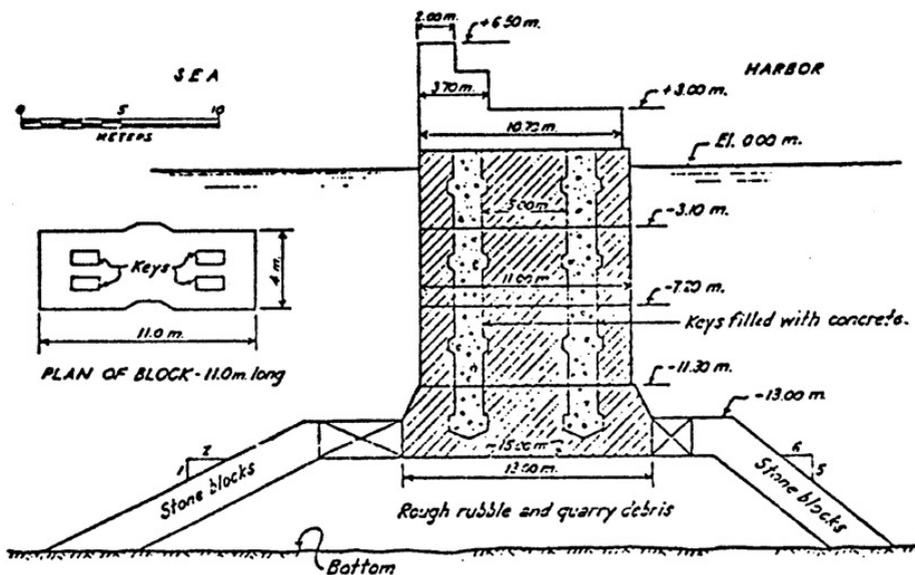


Figure 14-2 Breakwater Algiers, Morocco.

14.2.2 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.

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.

14.2.3 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 14.3.

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 14-3).

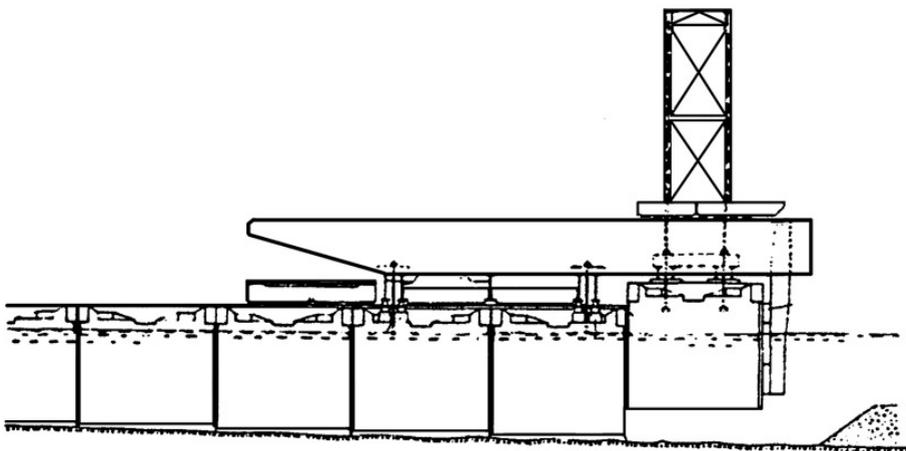


Figure 14-3 Construction method Brighton breakwater.

14.3 Caissons

14.3.1 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.

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 14-4).

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.

14.3.2 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).



Figure 14-4 Dredged dock for construction of caissons at Neeltje Jans construction island

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 14.3.4).

14.3.3 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.
- In closures, flow underneath the caisson will soon reach high values but piping (with scour of bed material) should be prevented.
- In closures, the gap size needs to be longer than the length of the caisson to allow for tolerances and diagonal length during the swing motion; however, outflanking of the flow along the sides has to be blocked immediately.
- 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. 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.

14.3.4 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/m³. 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 (<i>G</i>)	396 kN
centre of gravity above bottom (<i>g_b</i>)	4.80 m
moment of inertia ($I = \frac{1}{12}LB^3$)	60.75 m ⁴
displacement (<i>V</i>)	39.6 m ³
draught of caisson (<i>d_r</i>)	4.40 m
centre of buoyancy above bottom (<i>c_b</i>)	2.20 m
MC = <i>I/V</i>	1.53 m
metacentre height (<i>m_c</i>)	negative 1.07 m

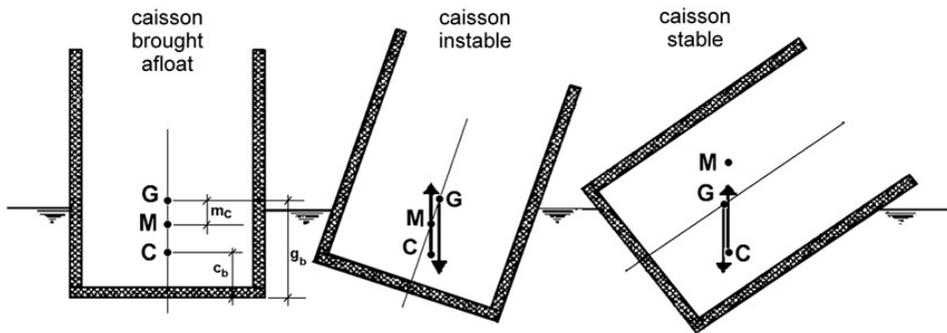


Figure 14-5 Stability of a floating caisson.

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 14-5 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° 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:

weight of concrete and water (<i>G</i>)	556 kN	increased by 160 kN
centre of gravity above bottom (<i>g_b</i>)	3.85 m	down by 0.85 m
moment of inertia (<i>I</i>)	60.75 m ⁴	unchanged

displacement (V)	55.6 m ³	increased by 16 m ³
draught of caisson (d_c)	6.18 m	increased by 1.78 m
centre of buoyancy above bottom (c_b)	3.09 m	raised by 0.89
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 14-6).

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.

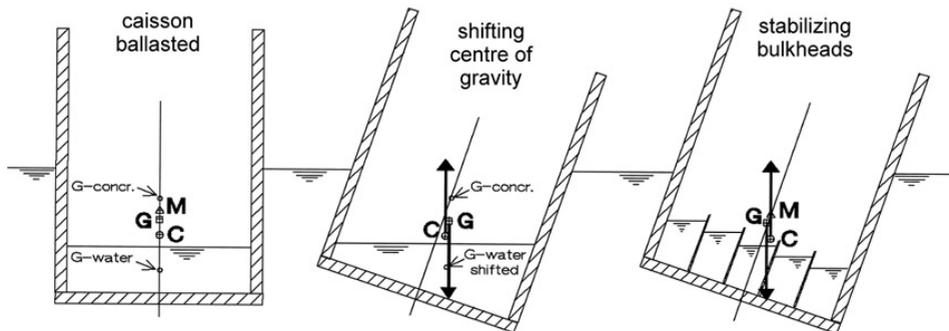


Figure 14-6 Stability of a caisson ballasted with water.

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.

14.3.5 The sinking operation

Placing

When on site, the caissons must be placed in the required position on the seabed. Adequate tugboat assistance is required to tow the units into position and to keep them in position during sinking. This is generally done at slack water. The question

of whether the placing operation should be carried out during HW slack or during LW slack depends on the draft of the caisson and the available water depth. In breakwater construction in many places the advantage of the LW slack is a reduction in the action that is required. For details see also Figure 12.7.

It is not easy to keep the unit in position during the sinking operation. It is recommended that at least one or two winches should be available to make a connection with the shore or with previously placed caissons. Shortly before it comes to rest on the foundation the unit has a tendency to move horizontally out of control. This is due to the overpressure in the thin layer of water between the seabed and the bottom of the caisson. The problem can be solved by increasing the permeability of the foundation layer or by fixing a skirt or some steel rods in the bottom of the caisson.

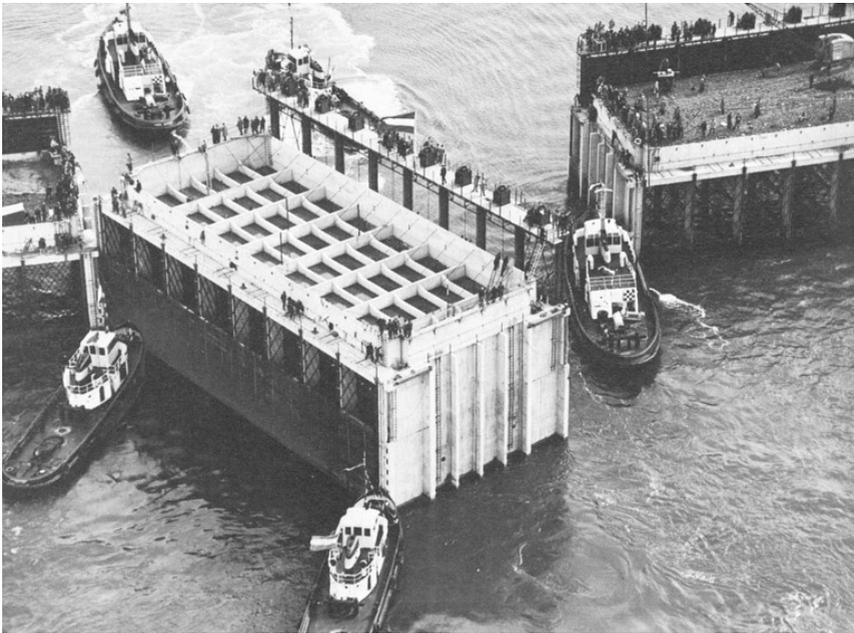


Figure 14-7 Placing the final caisson in "Het Veerse Gat".

In closures, the caisson is intended to block the gap or part of the gap and thus will be positioned transverse in the flow. Since dimensions are generally considerable, even small flow velocities will result in high forces during the manipulating and positioning of the caisson. Therefore placing will always be done during slack water when the tide turns. In practice, there is no instant when the flow velocity is zero. Generally, the tide starts turning near the bottom first and later at the surface and this does not usually occur over the full gap length at the same time. Therefore, slack water is the period (time window) in which velocities are smaller than about 0.5 m/s

in either direction. In a tidal cycle there are two such periods, these being during high-water when flood changes into ebb and during low-water with the change from ebb into flood.

14.3.6 Work-window of flow conditions during the sinking operation

In closure operations, the working window for the sinking of a caisson depends on the duration of the flow with velocities less than 0.5 m/s, called slack water. If a number of caissons have to be placed one after the other, the duration is different for each new caisson. As indicated before, in a tidal cycle, there are two periods of such slack water, one during high and one during low tide level. A number of considerations determine which slack water period is selected for the sinking operation:

- duration of the time window, which is not the same for the two slack periods.
- available keel clearance on the approach route of the caisson; sailing during high water may be preferred.
- draft and stability of the floating caisson
- stability during the ballasting and sinking operation
- the desired water level in the basin after closure (for the last caisson)
- the way of placing and the equipment used

The last item relates to the fact that caissons are preferably positioned by pushing (or pulling) them against the direction of the current. The advantage is that if something goes wrong, the caisson is pushed back by the flow into the free space, while in the opposite case the caisson may float into the gap and get damaged or cause damage. Thus the procedure starts by bringing the caisson into the gap against the diminishing flow well before the moment of still water. The most commonly used way to bring the caisson into the gap is to position it in advance, head-on on to one side of the gap. A corner is connected to a fixed point on the shore (as a hinge) and it can be swung around that point like a door into the gap. Then, by ballasting, the caisson is lowered and put down just before (or at) slack water. Further ballasting will stabilize its position while the direction of the flow underneath and around the caisson is reversed.

For an operation at high water slack, when the tide turns from flood into ebb, the caisson must be positioned opposite to the flood flow, thus from the basin side towards the open water. Therefore, the caisson has to be sailed into the basin via the gap during a preceding high water period (assuming the fabrication dock to be outside the basin). If the (last or the only) caisson is pushed into the gap by tugs, the tugs are trapped in the basin.

An example of an unusual method of closure that clearly demonstrates these aspects is the closure of the Miele in 1978. This was the main gully in the tidal flat area near

Meldorf in the north-west of Germany. The closing method that was originally planned failed and left a closure gap with limited possibilities to enforce a closure before the next winter. The gap was 320 m wide and the bottom elevation was 3.60 m below MSL. The tide had a range of 3.5 to 4 metres. It was decided to try a closure by caissons, by adapting five identical sand transport barges. A new bottom protection had to replace the distorted one.

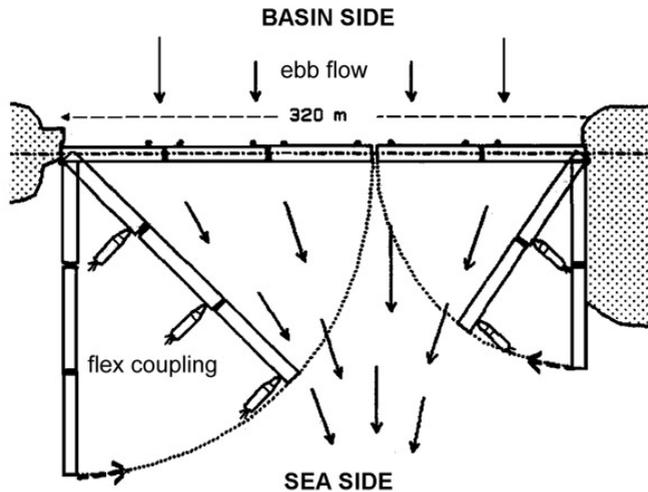


Figure 14-8 Caisson closure in the Miele (Meldorf).

A sill had to be created as foundation for the barges. The limited water depth did not allow the construction of a high sill, or even the use of large stones. Therefore flow velocities had to be kept low and the five barges could not be placed one after the other. The problem was how to put one composite caisson, consisting of five rather fragile steel barges onto a 320 m long sill without risk of breaking, piping underneath, or piping at the ends. The solution found was as follows:

The barges were provided with heavy steel H-profiles underneath, along both sides, to improve longitudinal strength, to improve penetration into the bed material and to reduce the formation of free-spans along uneven bed levels. For stability calculations, it was (pessimistically) assumed that the bed underneath the downstream H-profile would be less permeable than that on the upstream side and determine the upward water pressure. The barges were assembled into two composite caissons, one consisting of two and one of three barges. The connection between the barges was made by using flexible material that allowed each barge to settle independently (within reasonable limits). Much attention was paid to smoothing the sill to avoid high spots that would pierce the bottoms of the barges. The two sets of caissons were positioned near the gap at the two shore ends, where they were connected to a hinge (pole) by steel wire. Both sets of caissons were

swung into the gap simultaneously at low water slack. For ease of positioning (to prevent the "doors" from swinging too far) steel tubes had been piled in the alignment of the gap. Tugs on the seaward side had to gently push the caissons against these tubes. Being in line, the caissons had a wide slit where they met. This slit was closed by pushing from the shore-ends and releasing the wire hinges, while the space was divided over the two shore connections. Ballasting was done by pumping water and sand into the barges. Stones were dumped in the shore-end slits and to prevent piping sand was pumped at high capacity along the full length of the caissons via a floating pipeline. As soon as the rising water allowed dump-vessels to sail, stone was also dumped alongside the barges. (After a sand dike had been provided on the basin-side of the caissons, the barges were emptied, refloated, refitted and returned to normal operation. see Figure 14-8)

14.3.7 Number of caissons and/or sluice gate caissons

Generally, several caissons will be placed one after the other over a period of several days. Every caisson blocks part of the profile of the gap and the next caisson will be more difficult to position. Flow velocities increase, the time window diminishes and the turbulent eddies in front of the caisson will be more severe. Although the programme will be planned to work from spring tides to neaps, at least for the later caissons, the last caisson to be placed will determine the design and dimensions of the caissons and foundation bed material. The advantage of this method is that the operational phase in which flow velocities are very high is relatively short. This means that in areas with a limited workable period, for instance because of weather conditions or river discharges, the progress stays within schedule. Moreover, the duration of exposure to high flows with high eroding capacity is small.

For large closures, the last caisson may need unrealistic dimensions in order to achieve a sufficiently long workable period, so the use of caissons provided with sluice gates is a good option. In such cases, the structural components of every caisson (head-walls, diaphragm walls, bottom structure and ballast hold) block a small part of the gap profile. The gates will provide an opening of 80 to 85% of the submerged section of the caisson, which can be multiplied by a discharge coefficient in the hydraulic calculations. Again, the determining conditions are those during placing of the last caisson. These flow conditions determine the dimensions of the total opening provided by the gates. Multiplication by 1.3 gives the total gap size to be blocked by caissons with gates.

15 FAILURE MODES AND OPTIMIZATION

Chapter 15 looks in hindsight at the design and construction procedures for breakwaters and closure dams. An attempt is made to analyze 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.

15.1 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

15.2 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 15-1. BURCHARTH [1992] presents a slightly different review more focussed on breakwaters (Figure 15-2).

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 same applies to monolithic breakwaters. For this type of breakwater, some failure mechanisms have been assembled in Figure 15-3.

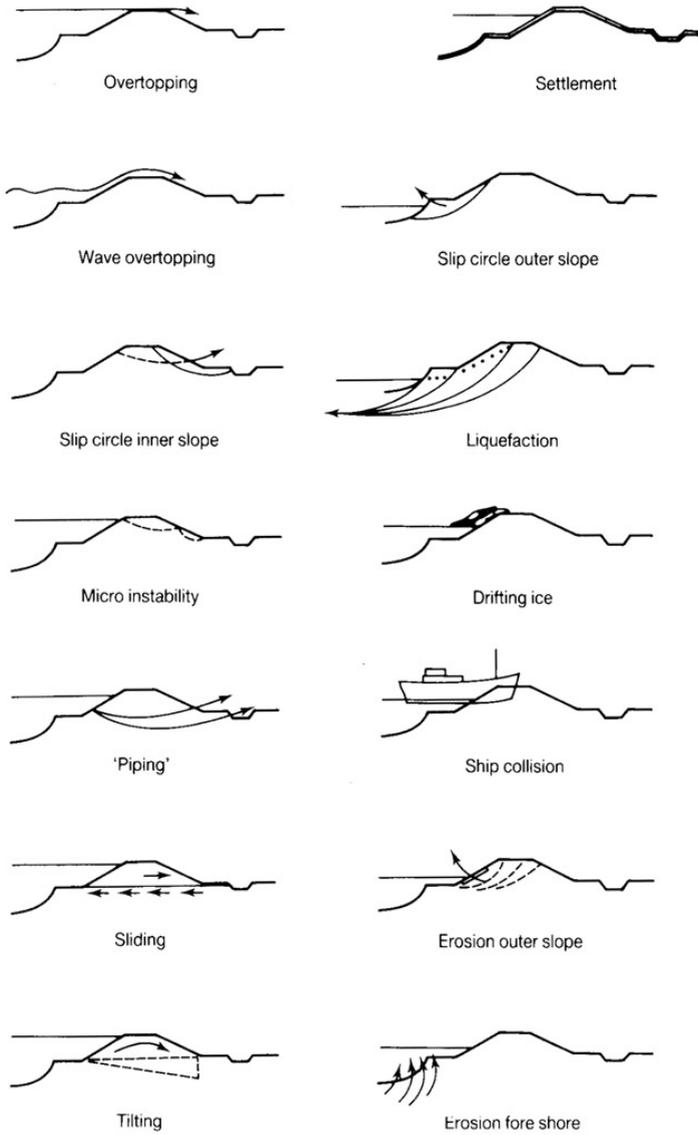


Figure 15-1 Failure modes of rock structures.

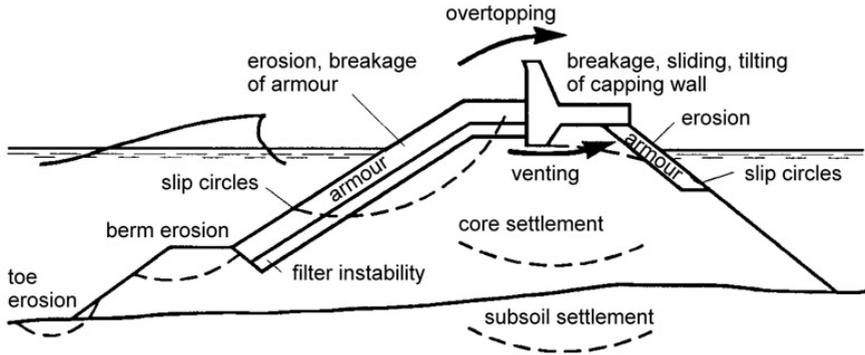


Figure 15-2 Failure modes for a rubble mound breakwater according to Burcharth [1992].

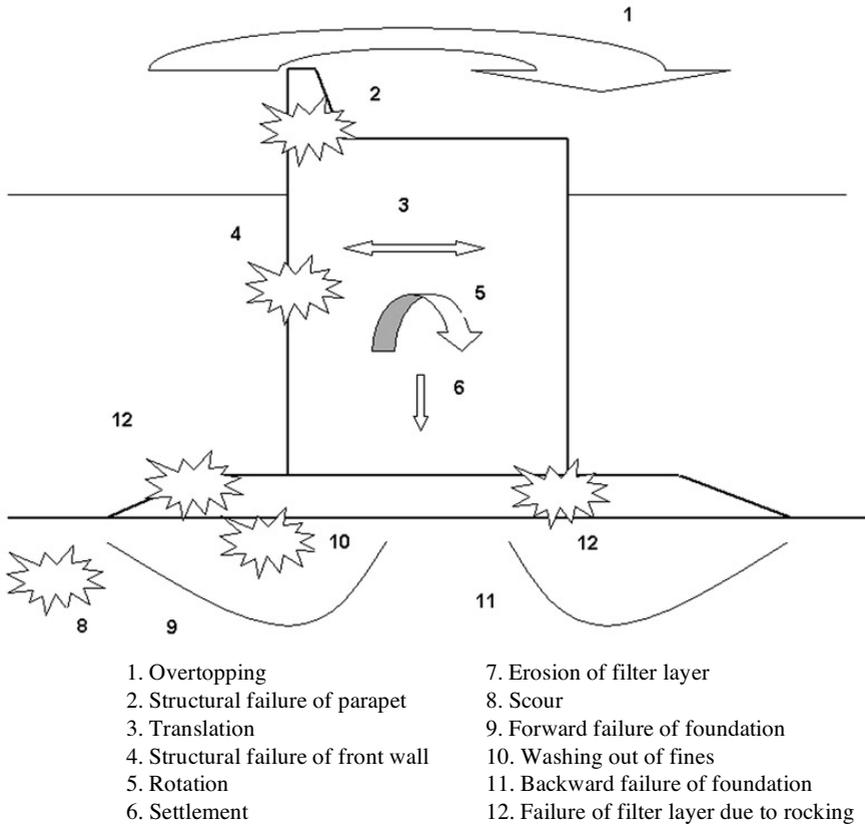


Figure 15-3 Some failure mechanisms for monolithic breakwaters.

An overview of failure modes of rock-fill overflow dams during their construction is shown in Figure 15-4. This list of failure modes is far from complete. In particular, failures caused by three-dimensional mechanisms are easily forgotten.

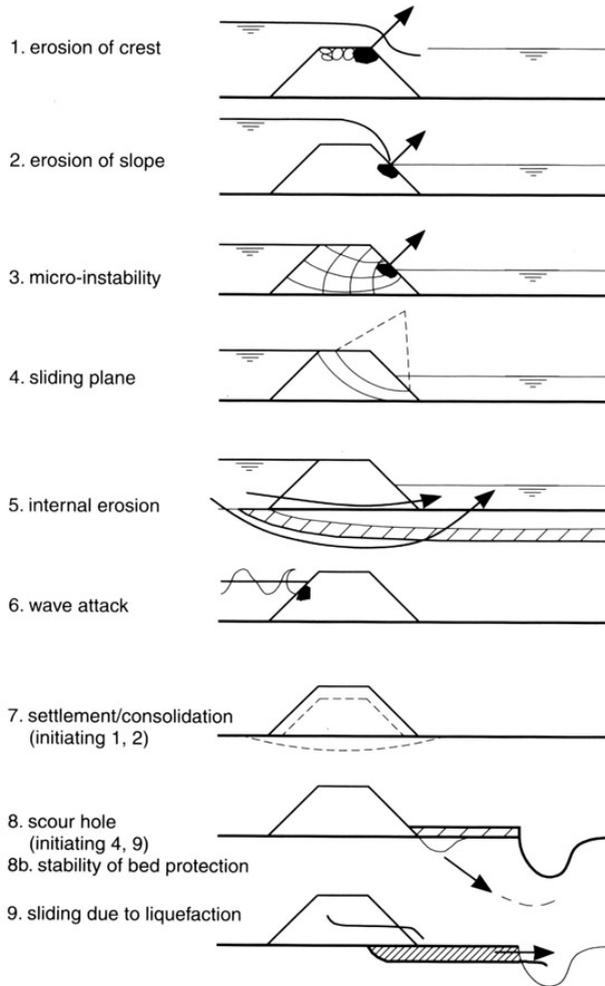


Figure 15-4 Rock-fill overflow dam failure modes at various construction stages.

15.3 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 15-5.

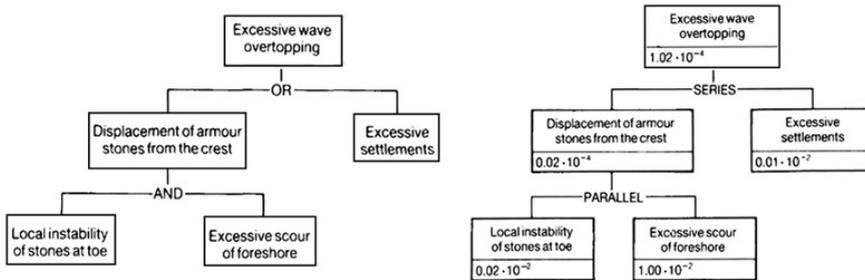


Figure 15-5 Fault tree and probability of failure.

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 (eg. 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 armour stone (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:

- a) Failure of cross-section of rock-fill dam
- b) Failure of construction equipment
- c) Failure of construction planning

Obviously, these fault trees and failure modes are only used to demonstrate the approach to be taken. For 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.

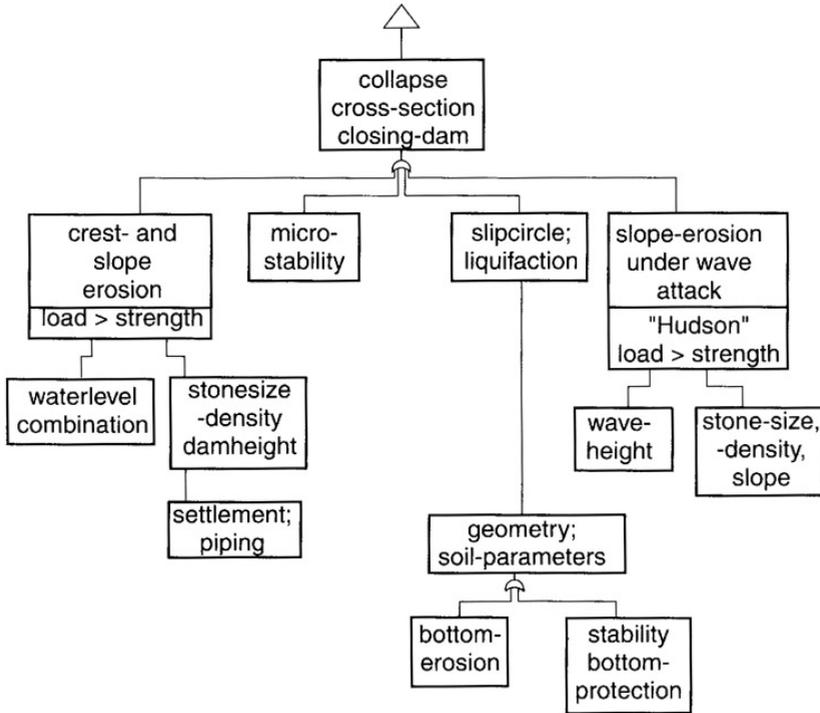


Figure 15-6 Estuary closure dam: fault tree for failure of cross-section.

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).

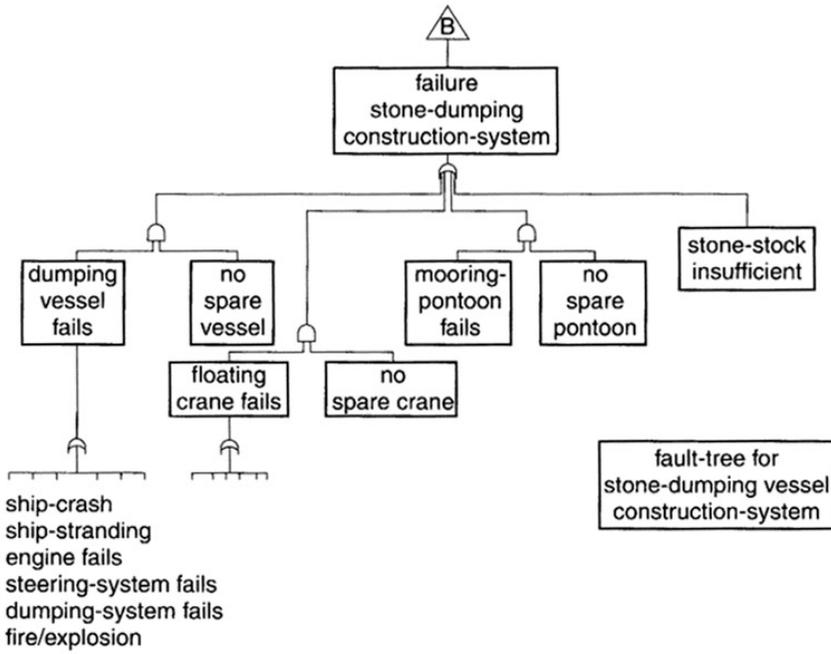


Figure 15-7 Estuary closure dam: fault tree for failure of construction equipment.

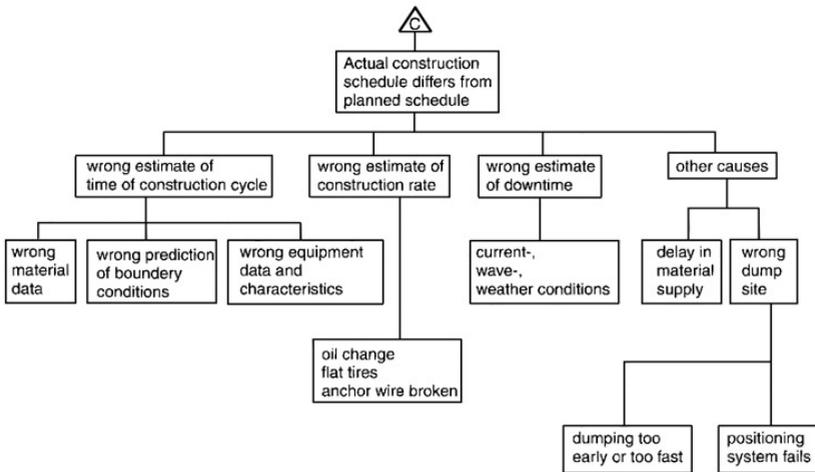


Figure 15-8 Estuary closure dam: fault tree for failure of construction planning.

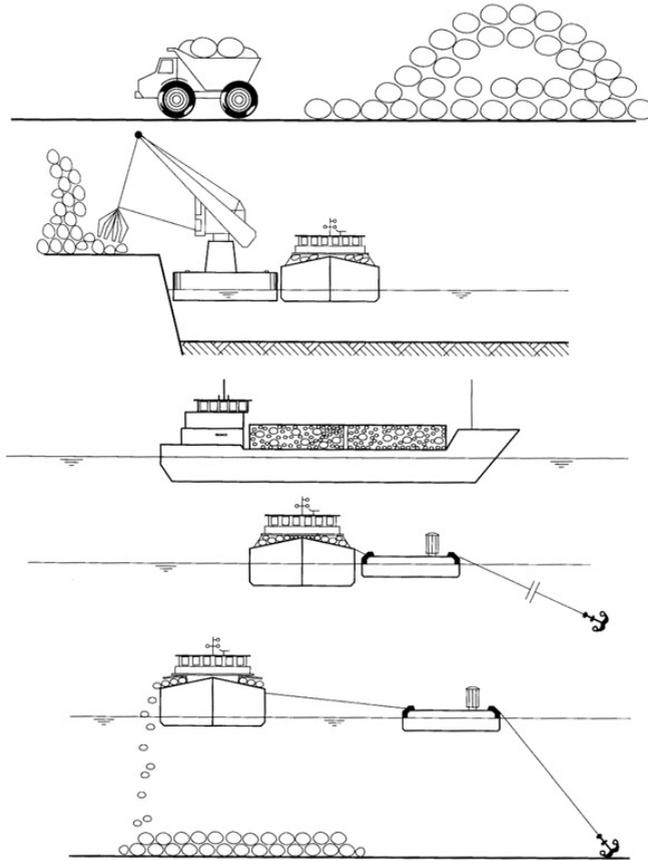


Figure 15-9 Estuary closure dam: equipment utilization **in relation to fault tree** (Figure 15-8).

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.

15.4 Optimisation

15.4.1 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 armour stone. For a low crested breakwater on the other hand, it is essential to carefully protect the inner slope.

15.4.2 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 (H_{nd}) 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 $H = 1.45 H_{nd}$. 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 $\delta\%$ 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 $\delta\%$, and the lifetime of the structure T , it can easily be derived that the fund to be reserved (S) is:

$$S = \int_0^T e^{-\frac{\delta}{100}t} dt = s \frac{100}{\delta} \left(1 - e^{-\frac{\delta T}{100}} \right) \quad (15.1)$$

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.

16 FLOW DEVELOPMENT IN CLOSURE GAPS

In Chapter 5, it was indicated that in the case of the closure of a river branch or an estuary, it is very important to know the velocities in the closure gap. This is because velocities in the gaps determine the stability of the material used and the choice of the closing strategy. There are fundamentally two ways to determine the velocities in the gap, both of which are useful at different stages in the design process and depending on circumstances.

At an early stage, it can be very useful to get a rough idea of the situation by making a simple manual calculation using the theory of Chezy. At a more definitive stage or in a more complex situation a mathematical model is often used. The most simple model is a 1-D model like DUFLOW or SOBEK. In this chapter, examples of these two options are presented.

16.1 Calculation of flow in a river channel

The example that is calculated in this chapter deals with a closure in a river system in which two main rivers flow more-or-less parallel in the same direction, each having its own catchment area and discharge characteristics. A connecting branch provides a link between the two. It is assumed that tidal influence is negligible. Such situations occur in various river deltas. This example is taken from the rivers Waal and Meuse (see Figure 4-9), with the connection near Heusden. (In the present situation the connection is closed near Andel, which is halfway along its length, by a dam and a ship lock).

In the calculation example, the river branch connecting the two points A and B of the main river system (Figure 16-1) has a fall of 2 m over a distance of 13 km. For the river flow, Chézy's formula for open channel flow applies:

$$u = C\sqrt{hi} \tag{16.2}$$

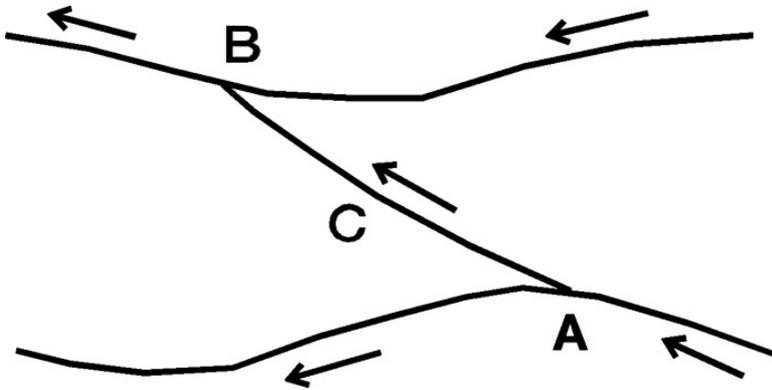


Figure 16-1 Channel view.

The flow velocity will be in the order of 0.80 m/s. It is assumed that the water levels at A and B do not change. A closure is planned halfway, at point C. As the closure progresses, three river sections have to be considered:

At C, in the closure gap, the profile diminishes while the flow velocity increases. However, the resulting discharge also diminishes and finally reaches zero.

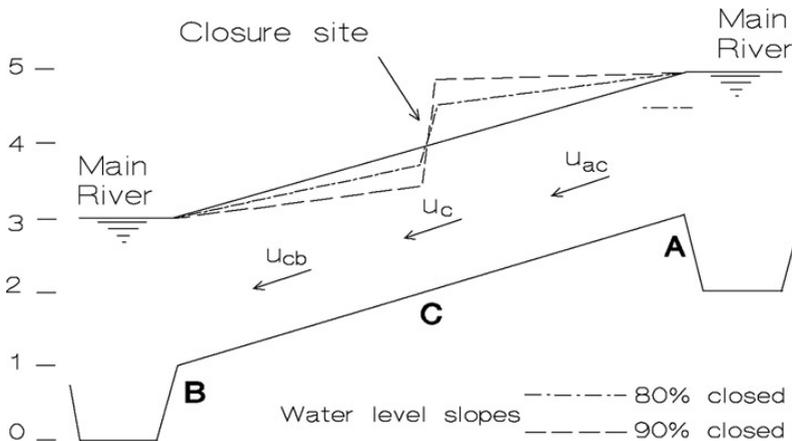


Figure 16-2 Closing a river channel.

Section AC is a river section in which the diminished discharge causes the flow velocity to drop while the water level is pushed up by backwater from C. CB, however, is a river section in which the level is drawn down (see Figure 16-2).

In the two river sections, the flow and water level calculation is a backwater curve calculation approximated by Chezy’s formula. In the gap, the weir formulae detailed in Section 5.2 must be used. The known parameters are the water levels at A and B, and the fact that the discharge quantity in all three section is equal (quasi static,

neglecting water storage in the sections of the river during the process). The unknown variables are the water levels on both sides of C and the magnitude of the discharge.

The sum of the reductions in head loss over AC and CB is equal to the fall over the gap at C, which is responsible for the high flow at C. In the example, a 90% blockage results in flow velocities of 0.40 m/s in AC, 0.55 m/s in BC and of 4.50 m/s in the gap. During the final stage, the water levels in the sections AC and CB are nearly horizontal, while the head over the gap is nearly the full difference in level between A and B, in this case about 2 metres. Applying the formula (5.5), indicates that the flow velocity in the final gap would thus rise to about 6 m/s.

In deltas, rivers may bifurcate repeatedly to form a complex branching system. Closing one of these branches is comparable to the above situation, with the assumption that the water level at either end of the channel is fully determined by the regime of the delta. The length of the river section to be closed may be considerable and likewise the fall in head during the final stage of closure may be quite large.

The calculations for the intermediate phases of closure may sometimes be rather complex in which case a mathematical model like DUFLOW can be used to make the calculation. For example, this model was used during the Jamuna bridge construction in Bangladesh for the closure of two secondary channels in the braided river system of the Jamuna River.

16.2 Calculation of flow in the entrance of a tidal basin

As an example of the flow velocities occurring during the closure of a tidal basin, a calculation has been made to illustrate the change in the tidal characteristics. The calculation has been made for two different basins, each having the same water surface area, 50 km². This area is kept constant for all tidal levels, which implies that no parts of the basins dry out during low water periods.

One basin is relatively short and rectangular, viz. 5 km wide and 10 km long. The bottom elevation is taken as constant all over the basin at a level of 6 m below mean sea level (MSL). For this basin, the water level will be nearly horizontal at all tidal heights over the full area. Thus, a calculation based only on conservation of mass may be appropriate. This can be ascertained by calculating the length of the tidal wave, for which the celerity is:

$$c = \sqrt{gd} \quad (16.2)$$

which is 7.7 m/s, and the period is in the order of 44700 seconds. Thus, the tidal wave has a length of about 350 km. As the length of the basin is well under 1/20 of this wavelength, the simplification is acceptable.

The other basin is a long, narrow, funnel-shaped estuary, in which the tidal wave propagates. The total length is 41.750 km and the width narrows from 2000 m to 800

m at the end. The bottom level rises from MSL.-6 m at the entrance to MSL.-2 m at the end.

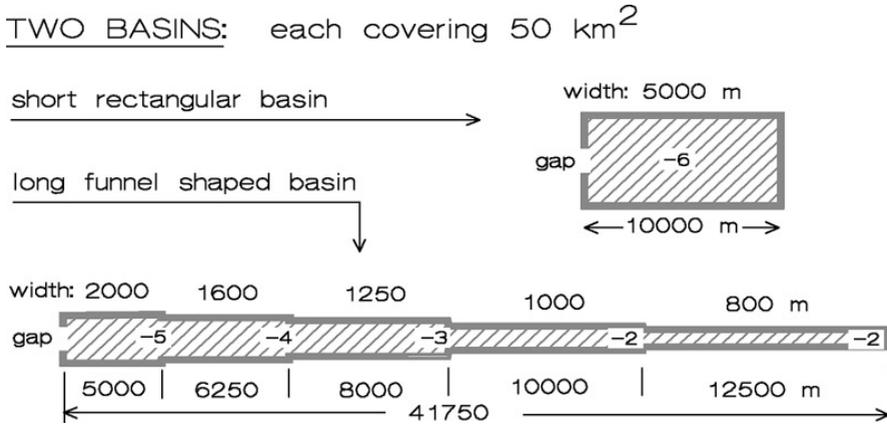


Figure 16-3 Two basins on which the calculations are based.

On average, the celerity of the tidal wave is about 5 m/s and thus the tide at the end of the basin will lag more than 2 hours behind. This is confirmed by the length relation, which is much more than $1/20$. The calculation needs to consider the conservation of mass as well as of momentum.

For comparison, both basins are closed by horizontal closure as well as by vertical closure. The results show the influences of the shape of the basin and the closure method. The four cases are illustrated by four sets of graphs showing curves for:

- water level just outside the basin near the gap
- water level just inside the basin near the gap
- water level at the end of the basin
- flow velocity in the closure gap

(Note: the water level curves for b and c are identical for the short basin)

The cases thus are characterized by:

- case 1, horizontal closure of short basin
- case 2, horizontal closure of long basin
- case 3, vertical closure of short basin
- case 4, vertical closure of long basin

All four cases have been calculated for stepwise reduction of the gap size in 5 stages (initial gap taken as 100%, 50%, 25%, 10% and 3%). For a vertical closure it is more difficult to define the gap size than it is for a horizontal closure, since for high sills the gap size for ebb differs a lot from the one for flood. Expressed in m^2 relative to MSL, a negative size is even possible. The sill levels have been selected such that comparison of the horizontal and vertical characteristics is possible. The different stages are shown in Figure 16-4.

5 closing stages

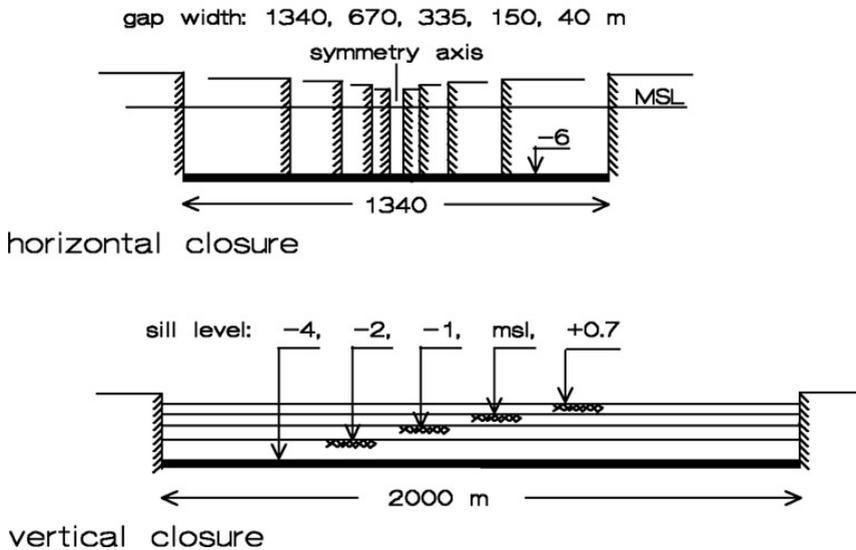


Figure 16-4 Closure gap dimensions and stages.

For reasons of fair comparison, in the calculations various parameters have been taken as identical and constant (which is in fact not true). These are:

- all discharge coefficients are taken as 1.0 for all gap dimensions, in both, horizontal and vertical closures
- all constructed parts of the closure dam are considered impermeable for water
- the tidal wave at the entrance is a single sine wave with a range of 3 m, which does not change as consequence of the progressing closure
- the Chezy-values of all sections in the calculation network are taken as 50 for all water depths
- the calculation is made using the mathematical model DUFLOW

(Note: the DUFLOW model has also been used for calculating the short basin. This enables confirmation of the former remark that curves b and c are identical.)

It should be realized that in a practical case the model has to be calibrated by reproducing an actually measured situation. The results of this will give the required data for Chezy-values and dimensions of the sections (depth-width relation to represent an irregular gully profile).

A few typical characteristics can be observed in the results. For the calculations with the short basin (cases 1 and 3), the water levels at the end of the basin do not show clearly as they are identical with the basin level near the gap.

Water levels in m above MSL (upper graph) and Flow velocities in m/s (lower graph) in relation to Closure Stage.

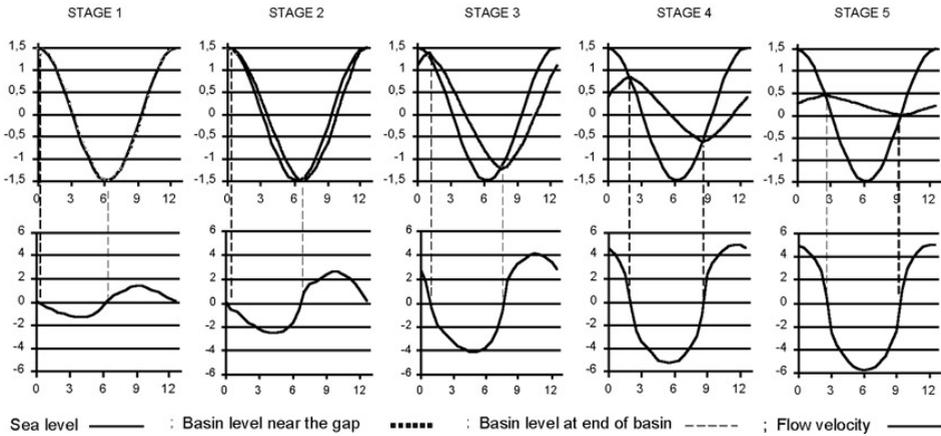


Figure 16-5 Horizontal closure of the short basin.

Water levels in m above MSL (upper graph) and Flow velocities in m/s (lower graph) in relation to Closure Stage.

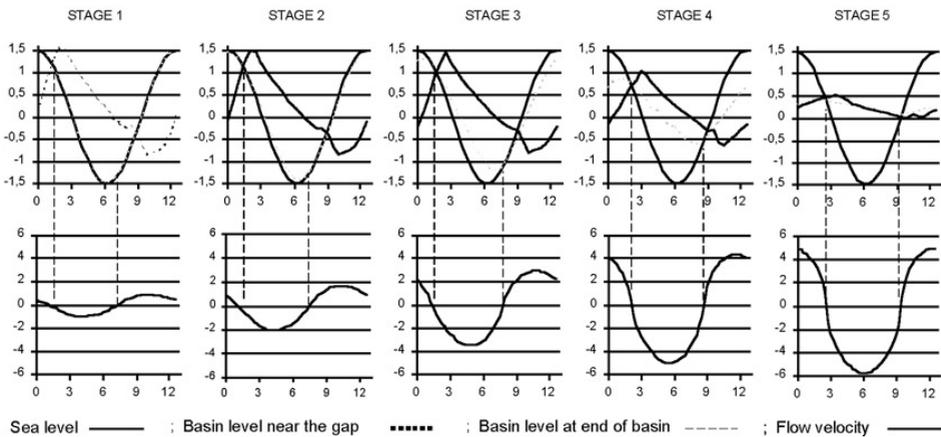


Figure 16-6 Horizontal closure of the long basin.

For the long basin (cases 2 and 4), the water level at the far end (dotted line) is lagging behind, as expected. Moreover, at the end of the basin, the high water reaches about the same level as it does near the gap but low water is much higher. Consequently, the mean level in the estuary rises towards the end by about 0.25 m. For all cases, the water levels in the basin near the gap show a diminishing range for each succeeding stage and the moments of high and low water occur later. These are identical for the horizontal closures (cases 1 and 2) and for the vertical closures (cases 3 and 4). They differ in the rise in mean level, however.

For the horizontal closures in stage 5 the rise is about 0.20 m. This is caused by the fact that during ebb the water level in the gap is slightly lower than during flood. For

the vertical closures this rise is about 1 m, which is caused by the high level of the sill, preventing the basin from discharging.

Water levels in m above MSL (upper graph) and Flow velocities in m/s (lower graph) in relation to Closure Stage.

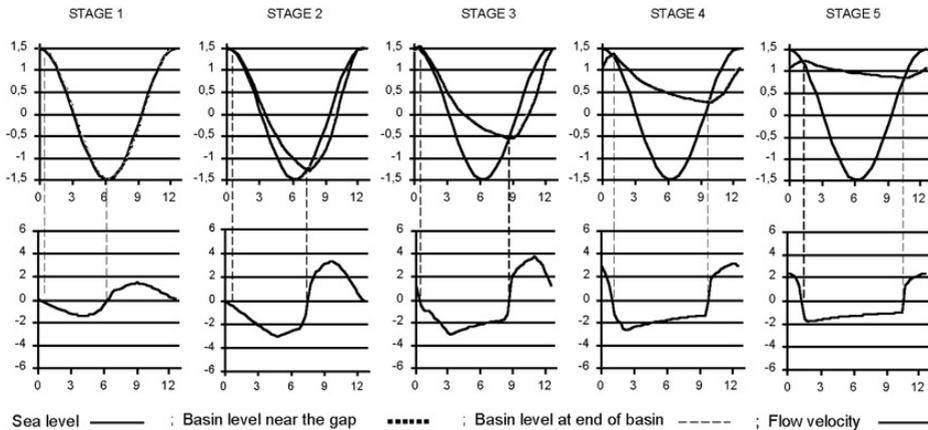


Figure 16-7 Vertical closure of the short basin.

Water levels in m above MSL (upper graph) and Flow velocities in m/s (lower graph) in relation to Closure Stage.

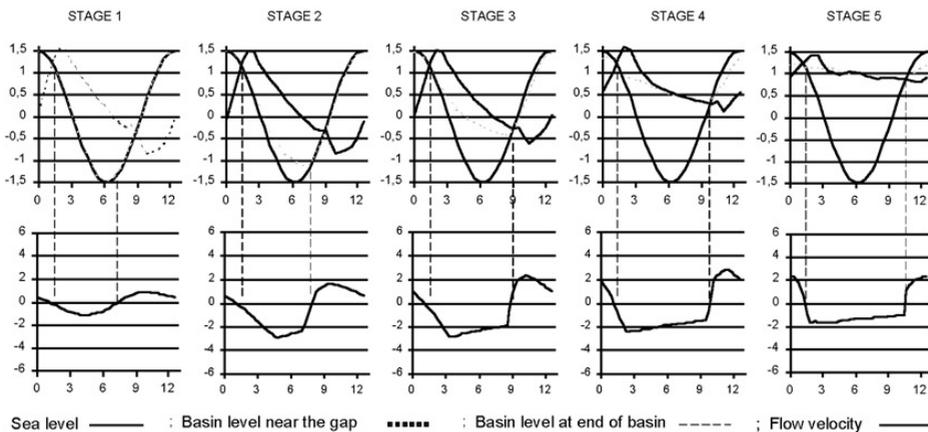


Figure 16-8 Vertical closure of the long basin.

Very typically for the long basin, the difference between water level curves at the gap and at the end diminishes as the closure progresses. In stage 5 there is hardly any difference and the basin apparently behaves as if it were a short basin with an almost horizontal water level all over the basin. This is true for both horizontal and vertical closure. In addition, the rise in mean level is identical to that in the short basin.

The flow velocities for the horizontal closures increase with every stage in a regular pattern. In the first few stages, the values for the short basin are slightly higher than for the travelling wave in the long basin, but in the last stage, they are identical. There is completely different flow behaviour in the vertical closures, however. In

stage 2, during low water at sea, the critical-flow situation already occurs. This shows in the straight cut-off line of the curve. Although in later stages the duration of the critical flow is greater, the maximum value of the ebb flow is diminishing. (In these calculations the critical-flow does not show in the flood period. (This may also happen in other cases.)

Although the flood does not reach critical-flow conditions, the maximum flood value also diminishes. This is caused by the fact that the mean level in the basin rises and thus reduces the head loss during flood. Again, the curves for short and for long basins are nearly identical. For the vertical closures, during low water at sea there is a very large drop of the water level over the sill in stages 4 and 5. As the sill level is MSL and +0.7 respectively in these stages, it can be concluded that the sill does not fall dry. The basin level always remains slightly higher than the sill, which is logical. (In actual cases, when large quarry stone is used sometimes the dam is quite permeable. In such cases the basin level may go down further.)

Comparing the flow in the short and long basins for vertical closures leads to the same conclusion as with the horizontal closures. In the first stages, it is slightly smaller in the long basin, in later stages, it is identical.

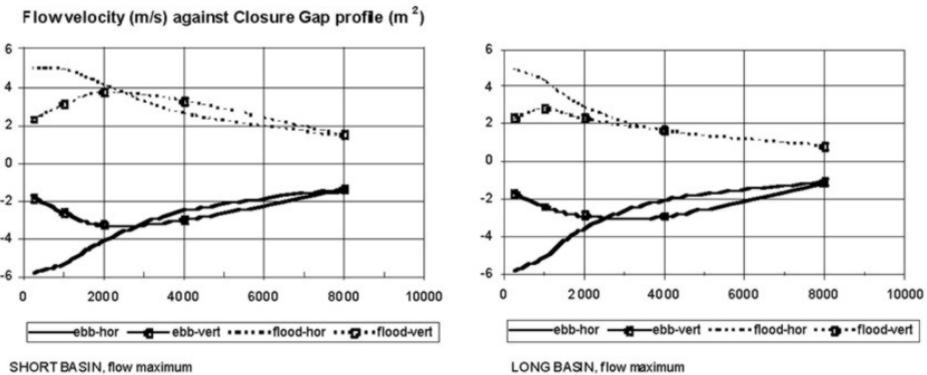


Figure 16-9 Overview of flow velocities.

Keeping flow velocities low by closing vertically is clearly demonstrated. This can be further illustrated in a graph showing the flow maxima against the profile of the gap, as is shown here for the short basin and the long basin. Both show identical trends. The results of the five stages calculated are presented, going from about 8000 m² down to nearly closed. For the gap profile the sectional area below MSL is taken. For the vertical closure in the last stage this would give a negative value. For that stage the data has been put in line with a 3% open gap. In the initial stages, the velocities during vertical closure are slightly higher than during horizontal closure. However, as vertical closure reaches critical-flow, the first ones diminish while the others still increase.

17 REVIEW

Chapter 17 reviews the result of the design process as a whole. It repeats especially the main choices to be made during design.

17.1 Breakwaters

17.1.1 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.

17.1.2 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.

17.1.3 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.

17.1.4 Service 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.

17.2 Closure dams

For closure dams, there are a few main directions the designer can follow. The first one is the choice between basic methods, the second one is the optimal use of the natural conditions and boundary conditions and the third concerns the selection of materials and equipment.

17.2.1 Decisive circumstances

There is no single prescription suitable for all closures because there are too many variables and boundary conditions. The unequivocal case is that of a well-defined tidal basin with a single closure gap of uniform dimensions. In practice, the situation is frequently more complex. Sometimes special conditions may so strongly determine a case that they either restrict or offer possibilities. Five typical examples of such criteria are detailed below:

1. The area of the basin can be easily subdivided into separate compartments

In essence, this is a matter of cost. Subdividing the area diminishes the storage capacity of the individual areas. Each closure can therefore be a lot easier, probably permitting the use of locally available materials so the total cost of these small-sized closures may be less than the cost of a single closure of the total area. However, additional costs may be incurred in the construction of the embankments separating the compartments.

Because of the later use of the area, the embankments may have to be removed. Sometimes, re-use of materials is a possibility, but some of the material is certainly lost. Depending on the layout of the area, subdivision can be designed in two ways. An elongated basin with a single channel can be taken in successive sections, while a complex channel pattern may require the successive closure of adjacent sections.

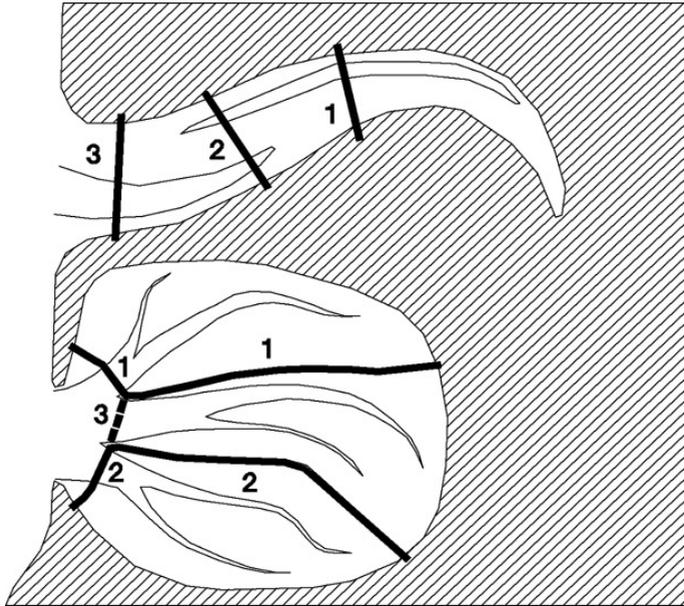


Figure 17-1 Two ways to reduce the area of a basin.

2. The basin is penetrated by tide via two separate entrances

Closure of the basin means closure of both entrances, with the option to close either one first or to close both simultaneously by any combination of methods, materials and phasing. All actions in one entrance will certainly affect the conditions in the other entrance and the balance between the two may be quite sensitive. In the case of a major imbalance, the tide conditions in the entire basin will also be affected and this will lead to changes in flow and subsequent erosion at several locations.

In such a case, a mathematical hydraulic model may be complex and difficult to calibrate. The problem is that somewhere in the basin the tidal waves will meet. Since these waves have a different history, their shapes, phases and amplitudes are not exactly the same. Nevertheless, generally a tidal devide (in Dutch called "wantij") is characterized by low flow velocities and an unusual relation between water level and flow.

The difficulty is how to estimate the correct Chezy-value for the gully system in this meeting area. For the existing situation, a wide range of values used in the mathematical model may give acceptable results and thus calibration gives no clue. However, as soon as the tide changes owing to the progress of the closure works and the meeting area moves, the unchecked value may be very important. Calculations with various assumed values will at least show the possible impact on the conditions. For simultaneous closure, the impact of every combination of construction phases on the tide penetration has to be determined. As for a single closure, this is done by

calculating with weir formulae. In these several schematic simplifications and practical coefficients are used. The resulting deviation has little impact on a single closure but for a dual system the balance may soon become unstable. Therefore, closure plans must allow for these deviations.

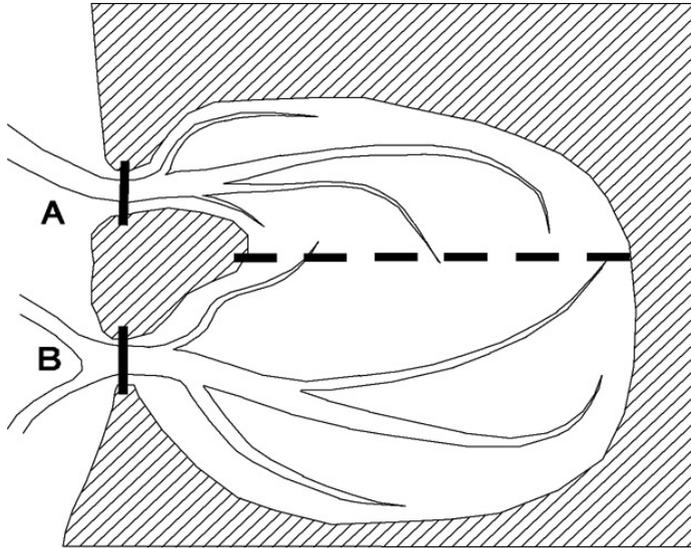


Figure 17-2 A basin with two entrances.

Even in the case of a very well thought-out plan of concurrent closure methods, a setback in the execution of one, also affects the other. Moreover, a major failure in one closure may lead to a complete disaster, as the other one has to be dismantled to maintain the balance of the basin.

The easiest way to overcome the problem is to make a temporary or permanent closure dam across the meeting point, which separates the two tidal systems and divides the basin into two compartments. Then, the two primary closures of the basin area are fully independent. The mathematics is more simple and reliable. The closure design for each one is independent of the other, as is the execution. Constructing the separating dam is a partial closure and frequently an obvious solution. However, in some cases this may not be allowed, for instance because it blocks a navigation route.

Another method is to plan to close the two entrances one after the other. The order of activities then is:

- fix the bottom topography of entrance "A" by protecting bottom and shores against future scour.
- close the entrance "B" by any closure method and accept the change in tide and conditions in the basin as well as in entrance "A".
- next, close the entrance "A".

The advantage is that the closures are independent in design, method and execution. The uncertainty about the response of the basin to the imbalance after the first closure has to be covered. This can be done by assuming that deep gullies scour across the meeting area and by taking the whole basin as a storage area for the tide calculations for the second closure. Compared with the closures for separated basins, the closure "B" may be easier because "A" is still an open entrance. However, "A" with the full basin behind the gap will be much more comprehensive.

It might have been possible to stabilize the meeting area and prevent the erosion of deep gullies. Then, the flow velocities would have increased but the topography would have remained intact. However, the cost involved in such erosion prevention will generally be higher than that of providing a temporary partial-closure dam to fully separate the systems.

3. The closure profile consists of two (or more) main gullies and shallows

Between the main gullies there will be an area of tidal flats. These more or less separate the gullies during the low water periods. During rising and falling tides, on the one hand they are storage areas and on the other hand they ensure balance between the gullies. Although not considered a tidal meeting, this has a lot in common with such a meeting. For this case too, the first problem for the designer is to prepare the mathematical model. Tide penetration is calculated by adopting a gully network, while tidal flats are assumed to provide a storage area only. However, imbalance creates flow, which results in erosion and the development of a gully across the shallow area.

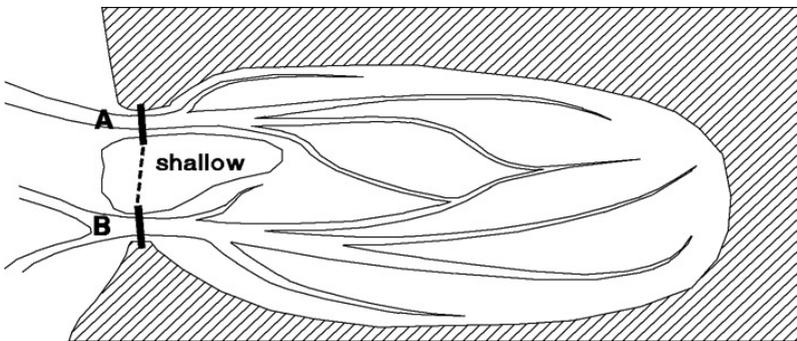


Figure 17-3 An estuary with two channels.

How quickly will that occur, how deep will this gully be and what will be the Chezy roughness? Separating the systems by dividing the basin is not logical as both gullies run into the same main storage area. Therefore, after construction of the dam-section across the shallow, the only possibilities remaining are:

- to close the two gullies simultaneously in a very balanced way.
- to close one gully first, accept or prevent erosion across the tidal flat and then close the other gap, taking into account a fully adapted situation.

In the second case, it is most likely that the dam section across the tidal flat will be built before the closure of the first gully. Since the main gullies are relatively close to this, the erosion of the short cut across the tidal flat will most likely develop along this section of the dam. Thus, the toe of this dam has to be heavily protected. In addition, flow conditions in the remaining gap will be very adversely affected. A better solution may be to create a short cut by dredging at an appropriate location to guide the tide towards the last gap.

4. The gap to be closed is not in an equilibrium state

This situation occurs in the case of a calamitous breach of a dam or dike. It may also happen when a construction phase goes wrong and creates unexpected conditions at the site of the gap. In these cases, time is a very important factor. Every day natural processes will try to achieve the equilibrium state and change the existing situation.

A first consideration is to analyze how quickly definite measures can be taken. Over this period, the situation will adapt and the magnitude of the change has to be estimated in order to plan the right measures. If this change is undesirable, temporary measures to halt or retard the deterioration can be considered. Such temporary measures include:

- stabilizing the attacked bottom of the gap by dropping coarse material. Stabilizing the sides of the gap is easier but may induce deeper scour. Generally, deeper scouring is worse than wider scouring.
- trying to avoid the erosion of gullies in the storage area, for instance by protecting critical spots with mats or quarry stone. A more developed gully network will result in easier penetration of the tide and increase the tidal volume.

In the meantime, data on the existing conditions can be measured and recorded, while a definite closure strategy is being drafted.

Usually, the existing situation has to be determined and secured before any construction phase can start. In some cases, such as calamities where life is endangered, a direct counter-attack is justified. The risk in such a case is that if the action fails, the situation is usually much worse than it was before the action. If an emergency closure cannot be obtained in a few days, certainty is more important than speed.

An example of a successful emergency closure was the blocking of the dike breach near Ouderkerk aan den IJssel (Holland) during the major storm flood in February 1953. A relatively small breach cut the dike, which secured a vast, densely populated

area of Holland, north of Rotterdam. Several hours later on the same night, a small vessel was taken and put onto the remainder of the outer slope of the dike, with neither any erosion protection nor any re-profiling of the gap to fit the vessel's shape. Piping under the vessel and around stem and stern could easily have scoured another gap. Then, the vessel would have broken and been pushed away, leaving a very large gap. However, the piping was blocked by using tarpaulins ballasted with sandbags (see Figure 17-4). The closure was a success and this central area of Holland remained dry.



Figure 17-4 Closure dike breach Ouderkerk aan den IJssel 1953.

5. Various alignments with different longitudinal profiles can be selected

These occur for instance, in a river branch with variable bottom topography. In a river bend there may be a deep triangular channel while in the cross-sections between bends a shallow box-profile may be available. (As in the alignments 1 or 2 in the upper half of the Figure 17-1, but this time as alternative locations.) Which of the two alignments is preferable?

Another example gives the situation which occurs after a dike located in a shallow area breaches. The breach will erode a deep scour hole very close to the original alignment of the dike. Owing to spreading of the flow, the surrounding shallow area will remain intact for some time, although erosion will gradually create gullies. The option is to restore the original dike or to build around the scour hole, either along the river (or sea) side or via the inside. Various considerations determine which option is the most attractive.

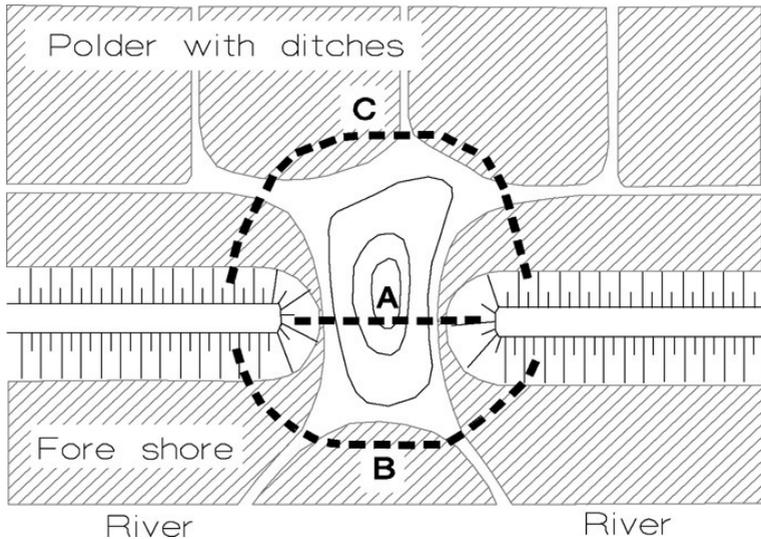


Figure 17-5 Closure alignments.

For many dike breaches in the past, closing around the scour hole was preferred. The old method involved sinking mattresses vertically. In order not to lose area, where possible, the alignment at the river side was taken, so that the scour hole became situated within the enclosed polder. Nowadays, in the Netherlands, these former scour holes still can be seen in the landscape as small circular ponds just at the inner toe of the dike where the alignment of the dike winds around them in a semi-circle. In Dutch such a pond is called a "wiel".

An important parameter is the amount of material needed to block the gap. The flow is determined by the nett cross-sectional profile of the gap in m^2 , while the gap has to be blocked by m^3 of material. For instance, a dam with slopes 1 in 1 with height "s" along a gap length "l", used to block a profile " $l \times s$ ", has a volume of " $l \times s^2$ ". An identical dam, of half the height along twice the length, blocks the same profile but requires only half the volume. On the other hand, the bottom protection (if needed) is twice as wide but may be more than half as long (in the flow direction). Other parameters relate to the equipment, the materials and the closure method used. A shallow gap may be difficult for large operating vessels to approach. It is preferable to use caissons in deep gullies. For a vertical closure, however, a long gap is advantageous because of the resulting lower current velocities. This is demonstrated in the examples of Sections 12.2 and 12.3.

APPENDICES

APPENDIX 1

Example of the determination of a design storm

A1.1 Statistics of individual observations

In this computational example we will look for the design storm in front of the coast of the Netherlands. We will try to find a storm with an exceedance frequency of 1/225 per year. This storm yields 20% probability of failure in a lifetime of 50 years as has been explained in Chapter 3. It is stressed again that the use of these values does not constitute a recommendation on the part of the authors.



Figure A1-1 The Noordwijk measuring station.

As described in Section 5.3.3, we start from a series of wave observations with a fixed interval. Each observation (e.g. with a duration of 20 minutes) results in a given value of the H_s . When an observation programme is continued during a long time, a time series is created of H_s -values. This time series is the basis of the statistical operations explained in this section.

In the example case we use, we have 20 years of data available from observations made at "Meetpost Noordwijk", off the Dutch coast (Figure A1-1)¹. Every half hour a wave observation is made, but in the file used in this example, these data are

¹ The dataset is provided by the Netherlands Ministry of Public Works, the location Meetpost Noordwijk was at a waterdepth of approximately 18 m. (this station was decommissioned in 2006). Data from this, and other Dutch sites, can be downloaded from <http://www.golffklimaat.nl>

reduced to one observation every three hours. A one-month sample is plotted in Figure A1-2.

In Table A1-1 the clustered data are presented. The number of observations of each wave height bin is given both per bin as well as cumulatively. The probability of any wave height H'_s being equal or less than a specific wave height H_s is defined as:

$$A1 \quad P = P(H'_s \leq H_s) \quad (A1.1)$$

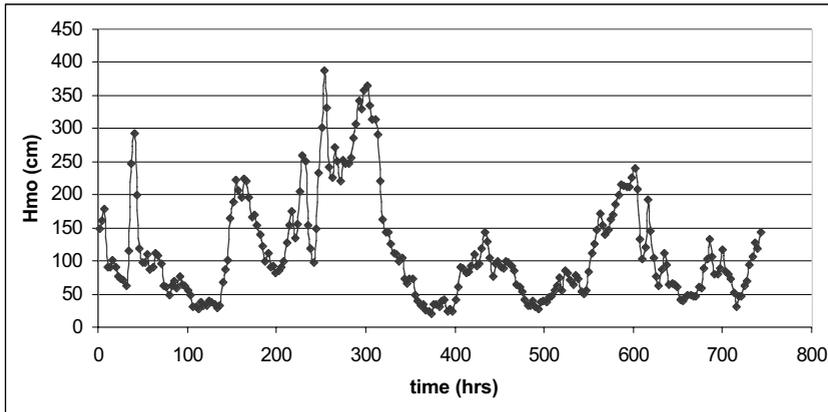


Figure A1-2 Data from Meetpost Noordwijk for one month (January 1979).

A probability of exceedance that H'_s is greater than a specific wave height H_s may also be defined as:

$$Q = Q(H'_s > H_s) = 1 - P \quad (A1.2)$$

Because a log relation is assumed, in the table also the value of $-\ln(Q)$ is given. Analysis shows that the correlation between H_s and $-\ln(Q)$ is 98.8 %. Plotting these data results in Figure A1-3. Although the correlation coefficient is quite large, it is clear that a log-distribution is not fully correct. The data are not on a straight line. For a small extrapolation this is not really a problem. The values from the regression line can be used to calculate the probability of exceedance of a given H_s . For example, a value of $H_s = 6$ m gives:

$$-\ln(Q) = 1.51 \cdot 6 - 0.5 = 8.5$$

$$Q = 2.035 \cdot 10^{-4} = 0.0002$$

Wave height class Hs (cm)		Number of observations		P	Q	-ln(Q)
		per bin	cumulative			
0	25	35	35	0.000599	0.999401	0.000599
25	50	8260	8295	0.141940	0.858060	0.153082
50	75	11424	19719	0.337423	0.662577	0.411618
75	100	10004	29723	0.508607	0.491393	0.710511
100	125	7649	37372	0.639493	0.360507	1.020245
125	150	5563	42935	0.734685	0.265315	1.326838
150	175	4389	47324	0.809788	0.190212	1.659615
175	200	3167	50491	0.863980	0.136020	1.994954
200	225	2360	52851	0.904363	0.095637	2.347200
225	250	1671	54522	0.932957	0.067043	2.702419
250	275	1234	55756	0.954073	0.045927	3.080692
275	300	851	56607	0.968634	0.031366	3.462047
300	325	556	57163	0.978149	0.021851	3.823487
325	350	392	57555	0.984856	0.015144	4.190168
350	375	276	57831	0.989579	0.010421	4.563938
375	400	206	58037	0.993104	0.006896	4.976819
400	425	136	58173	0.995431	0.004569	5.388507
425	450	82	58255	0.996834	0.003166	5.755400
450	475	66	58321	0.997964	0.002036	6.196632
475	500	38	58359	0.998614	0.001386	6.581307
500	525	30	58389	0.999127	0.000873	7.043930
525	550	20	58409	0.999470	0.000530	7.541769
550	575	22	58431	0.999846	0.000154	8.778531
575	600	9	58440	1.000000	0.000000	

Total 58440

Table A1-1: Wave data from Meetpost Noordwijk.

all data Noordwijk

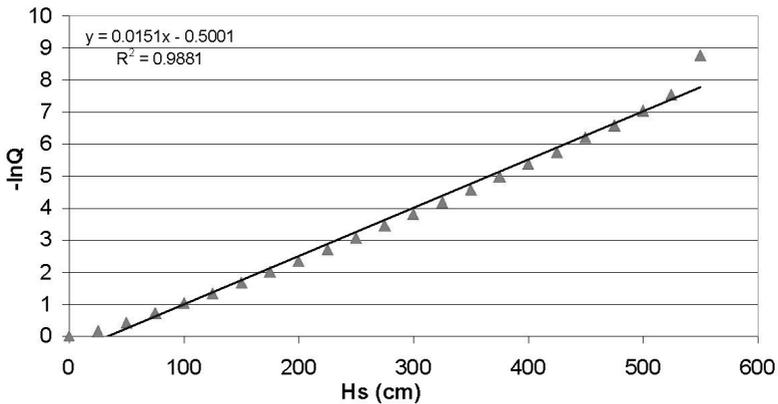


Figure A1-3 Cumulative data for Noordwijk.

This means that a wave height $H_s > 6$ m will occur during 0.02 % of the time. This result can also be obtained in a more direct manner. Plotting the values of Q on log-paper gives Figure A1-4. In this graph the exceedance of an observation larger than H'_s is given. The advantage of this plot is that the exceedance values can be read from the graph. (The statistical analysis and the plotting of data can easily be carried out using a spreadsheet software package. In this case, MS Excel has been used. It is beyond the scope of this book to outline the settings of the software). It is evident that the upper boundaries of the wave height bin should be used on the wave height axis (in this example 86% of the waves is higher than 50 cm, 99% is higher than 25 cm and 100% is higher than 0 cm).

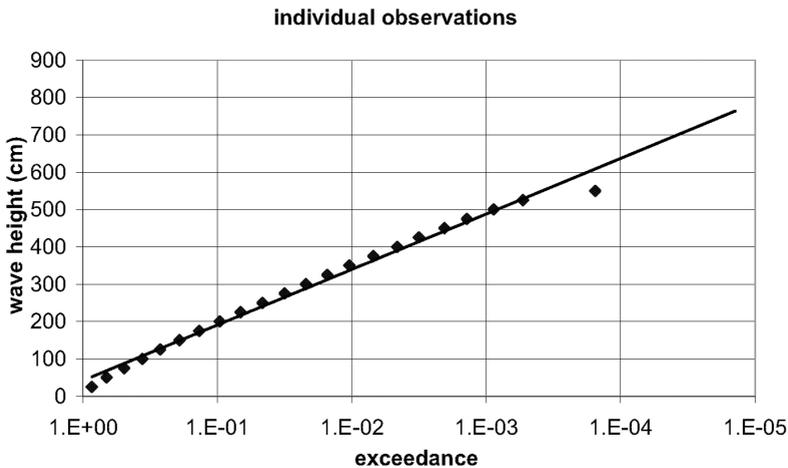


Figure A1-4 Exceedance graph for Noordwijk.

However, for design purposes, these values have no real meaning. It means that, if one goes at a random moment to the sea, there is at that given moment a probability of 0.0002 (i.e. 0.02%) of observing a H_s of 6 m or more. But this has nothing to do with a design storm with a H_{ss} of 6 m.

This form of statistics can be applied for the assessment of workability. In fact the above example means that 0.02 % of the time a wave condition can be met in which H_s is larger than 6 m. Suppose that for a port in this area pilot service has to be suspended when $H_s > 3.5$ m, it can easily be read that during 1% of the time there is no pilot service.

For the design of structures, we need the exceedance of a design storm and not the exceedance of an individual wave condition.

A1.2 The Peak over Threshold method

In order to transform these individual observations into storms, we may use the fact that sequential wave height observations are not random (see also Figure A1.2).

When we measure at 12:00 hrs a H_s of 4 m, it is very unlikely that we measure at 15:00 hrs a H_s of 0.4 m. Usually the observations of 12:00 hrs and 15:00 hrs will belong to the same storm. So we will define a certain threshold, e.g. $H_t = 1.5$ m. And we will look in our record when the wave height exceeds 1.5m. The threshold-value of $H_t = 1.5$ m is arbitrarily selected. Later we will investigate the sensitivity of this choice. The month of the record in Figure A1-2 (Jan 1979) shows 9 storms, providing 9 data points.

The reason for introducing a threshold is to avoid that small variations in wave height during long, calm periods have significant influence on the final result. Basically one should assume the threshold as high as possible, as long as the base for statistics contains sufficient data for a reliable analysis. Later it will be shown that the final answer is not very sensitive to the choice of the threshold value.

When we process the whole data set of 20 years, we find 1746 storms in total with a H_{ss} higher than 1.5 m. The 1746 storms are classified in wave height bins, according to the maximum $H_{ss\ of}$ each storm which results in Table A1-2.

Wave height class		Number of storms		$\alpha=1.24$							
H_{ss} (m)	per bin	cum.	P	Q	$\ln(Qs)$	$-\ln(H)$	Qs	G	W		
1.50	1.75	384	384	0.21993	0.78007	4.22098	-0.56	68.10	-0.415046	0.32522	
1.75	2.00	381	765	0.43814	0.56186	3.89284	-0.69	49.05	0.192121	0.64136	
2.00	2.25	266	1031	0.59049	0.40951	3.57655	-0.81	35.75	0.640938	0.91261	
2.25	2.50	157	1188	0.68041	0.31959	3.32863	-0.92	27.90	0.954366	1.11202	
2.50	2.75	148	1336	0.76518	0.23482	3.02042	-1.01	20.50	1.318085	1.34858	
2.75	3.00	111	1447	0.82875	0.17125	2.70471	-1.11	14.95	1.672191	1.58094	
3.00	3.25	81	1528	0.87514	0.12486	2.38876	-1.18	10.90	2.014645	1.80552	
3.25	3.50	63	1591	0.91123	0.08877	2.04769	-1.25	7.75	2.375535	2.04065	
3.50	3.75	31	1622	0.92898	0.07102	1.82455	-1.32	6.20	2.608194	2.19099	
3.75	4.00	32	1654	0.94731	0.05269	1.52606	-1.39	4.60	2.916351	2.38832	
4.00	4.25	23	1677	0.96048	0.03952	1.23837	-1.45	3.45	3.210883	2.57486	
4.25	4.50	11	1688	0.96678	0.03322	1.06471	-1.5	2.90	3.387796	2.68590	
4.50	4.75	20	1708	0.97824	0.02176	0.64185	-1.56	1.90	3.816515	2.95184	
4.75	5.00	9	1717	0.98339	0.01661	0.37156	-1.61	1.45	4.089424	3.11883	
5.00	5.25	7	1724	0.98740	0.01260	0.09531	-1.66	1.10	4.367707	3.28731	
5.25	5.50	9	1733	0.99255	0.00745	-0.43078	-1.7	0.65	4.896399	3.60263	
5.50	5.75	8	1741	0.99714	0.00286	-1.38629	-1.75	0.25	5.854211	4.15922	
5.75	6.00	5	1746	1	0		-1.79	0.00			
		1746									

Table A1-2: Data from Noordwijk using PoT of 1.5 m.

Using the same data, one can also determine the wave steepness. Without presenting the analysis, we found when looking at only those storms with a $H_T \geq 5$ m, a total of 29 storms in 20 years. The average duration of these heavy storms is 6.6 hrs, and the average wave steepness in these storms is 5.8 %, with a standard deviation of 0.6%.

These figures will be used in Section A1.4.3, where the probabilistic approach will be discussed.

The exponential distribution

Like was done before in equations (A1.1) and (A1.2) for H_s , the values of P and Q are computed for H_{ss} :

$$P = P(H'_{ss} \leq H_{ss}) \quad (\text{A1.3})$$

$$Q = Q(H'_{ss} > H_{ss}) = 1 - P \quad (\text{A1.4})$$

Plotting the values of Q logarithmically results in Figure A1-5, which gives the probability of exceedance of a storm. One can read this graph as: Given there is a storm (according to our definition $H_s > 1.5$ m), then the probability that that storm has an H_{ss} of more than 4 m is 0.05 (or 5%). [$4 = 0.78 \ln(x) + 1.63$]²

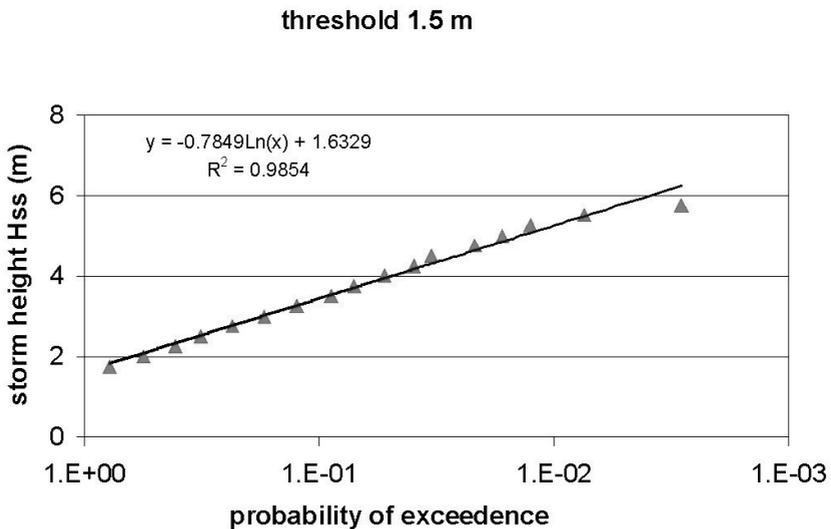


Figure A1-5 Wave exceedance using a threshold.

Still we do not know the probability of a storm that occurs with a probability of for example 1/225 per year. In the analysis done so far we looked only at the probability of a single storm. We still need to transform our information to a probability per year. This can only be done when we know the number of storms per year. This

² In general an exponential function has the form: $f_H(h) = B^{-1} \exp[-(h-A)/B]$, in which A the threshold value represents. This can be rewritten to the form as presented here.

number is known, since we have 1746 storms in 20 years, the average number of storms per year $N_s = 1746/20 = 87.3$. When we have 87.3 storms per year, a storm with a probability of exceedance of $1/87.3$ has to be the “once-per-year” storm (in correct statistical terms: the storm with a probability of exceedance of once per year). For transformation of the general probability of exceedance (Q) to the probability of exceedance of a storm in a year (Q_s), we multiply Q with the number of storms in a year.

Thus:

$$Q_s = N_s Q$$

The values of Q_s are also given in Table A1-2. Values of $Q_s > 1$ should not be called a “probability”, because probabilities cannot be larger than 1. Physically however, these values do have a meaning. They represent the expected number of storms in a year.

The values of Q_s can again be plotted on log-paper. This results in Figure A1-6. In fact, Figure A1-6 is the design graph to be used. So we can calculate our 1/225 per year storm with:

$$\begin{aligned} H_{ss} &= -0.785 \ln(Q_s) \\ &= -0.785 \ln\left(\frac{1}{225}\right) + 5.141 \\ &= 9.39 \text{ m} \end{aligned}$$

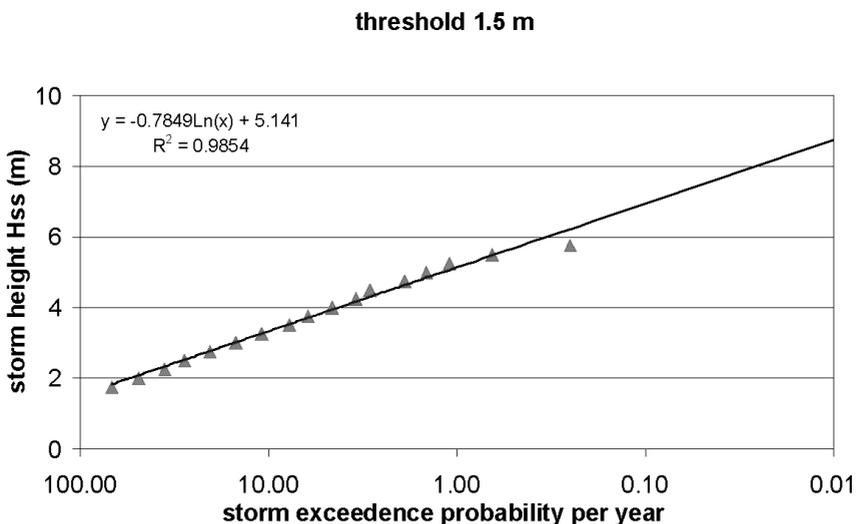


Figure A1-6 Storm exceedance using a threshold.

The Gumbel distribution

A more detailed inspection of Figures A1-4 and A1-5 reveals that, like in Figures A1-2 and A1-3, the points are not exactly on a straight line. This is caused by the fact that a simple exponential relation has been used. Because we deal with extreme values, an extreme value distribution like Gumbel or Weibull may result in predictions that are more reliable.

The Gumbel distribution is given by:

$$P(H \leq H_{ss}) = \exp \left[-\exp \left(-\frac{H_{ss} - \gamma}{\beta} \right) \right] \quad (\text{A1.5})$$

The coefficients β and γ can be found by regression analysis on the data. In order to do so, one has to reduce the equation of the Gumbel distribution to a linear equation of the type $y = Ax + B$. After that, standard regression will provide the values A and B , and subsequently values of β and γ . Taking two times the log, the Gumbel distribution can be reduced to:

$$\begin{aligned} \ln P &= -\exp \left(-\frac{H_{ss} - \gamma}{\beta} \right) \\ \underbrace{-\ln(-\ln P)}_G &= \frac{H_{ss} - \gamma}{\beta} = \underbrace{\frac{1}{\beta} H_{ss} - \frac{\gamma}{\beta}}_{AH_{ss} - B} \end{aligned} \quad (\text{A1.6})$$

The left-hand side of equation (A1.6) we call the reduced variate G :

$$G = -\ln \left(\ln \frac{1}{P} \right)$$

The values of G can be calculated for all P -values in Table A1-2. The values of G and H_{ss} are plotted in Figure A1-7. A linear regression leads to:

$$G = AH_{ss} + B$$

For the given dataset $A = 1.365$ and $B = -2.535$

$$\text{So: } \beta = 1/A = 1/1.365 = 0.733$$

$$\gamma = -\beta B = -0.733 \cdot -2.535 = 1.877$$

Like with the exponential distribution, the analysis given so far results in an absolute exceedance of a storm, not in a probability per year. As given before:

$$Q_s = N_s Q$$

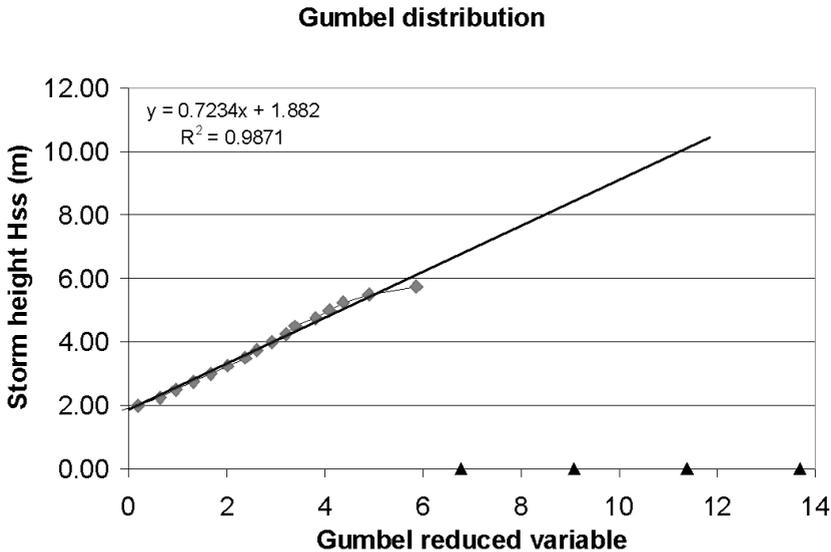


Figure A1-7 The Gumbel exceedance graph.

In this example a linear regression is used. Many other methods exist to estimate the parameters, for details is referred to statistical textbooks. From the Gumbel distribution it follows:

$$\begin{aligned}
 H_{ss} &= \gamma - \beta \ln \left(\ln \frac{1}{P} \right) \\
 &= \gamma - \beta \ln \left(\ln \frac{1}{1-Q} \right) \\
 &= \gamma - \beta \ln \left(\ln \frac{1}{1 - \frac{Q_s}{N_s}} \right) \\
 &= \gamma - \beta \ln \left(\ln \frac{N_s}{N_s - Q_s} \right)
 \end{aligned} \tag{A1.7}$$

So:

$$H_{s1/225} = 1.88 - 0.73 \ln \left(\ln \frac{87.3}{87.3 - 1/225} \right) = 9.10$$

A disadvantage of this approach is that in Figure A1-7 on the horizontal axis the reduced variable G is plotted, and not the probability of exceedance Q_s . Of course this can be transformed by:

$$G = -\ln \left(\ln \frac{N_s}{N_s - Q_s} \right) \quad (\text{A1.8})$$

So, for this given example (with $N_s = 87.3$ storms/year) this results in:

Q_s	G
1/10	6.77
1/100	9.07
1/1000	11.38
1/10000	13.68

Table A1-3.

These values can be inserted in Figure A1-7. One should realize that when the number of storms per year (N_s) changes, also the units on the converted horizontal axis will have to be changed.

The Weibull distribution

Instead of a Gumbel distribution, we also may try a Weibull distribution. The Weibull distribution is given by

$$P(H_{ss}) = Q = \exp \left[- \left\{ \frac{H_{ss} - \gamma}{\beta} \right\}^\alpha \right] \quad (\text{A1.9})$$

Also here, in order to find the values α , β and γ from regression analysis, we have to reduce the equation.

$$\begin{aligned} -\ln Q &= \left[\frac{H_{ss} - \gamma}{\beta} \right]^\alpha \\ (-\ln Q)^{1/\alpha} &= \frac{H_{ss} - \gamma}{\beta} \\ \underbrace{(-\ln Q)^{1/\alpha}}_W &= \underbrace{\frac{1}{\beta} H_{ss} - \frac{\gamma}{\beta}}_{AH_{ss} - B} \end{aligned} \quad (\text{A1.10})$$

So the reduced Weibull variate is

$$W = -(\ln Q)^{1/\alpha} \quad (\text{A1.11})$$

The Weibull distribution has three variables (α , β and γ). Linear regression will provide only two constants, A and B (and subsequently β and γ). So the determination of the third coefficient (α) will require some trial and error. Assuming different values of α will change the curvature of the points in the plot of W vs. H_{ss} . The calculation can be carried out quite easily in a spreadsheet. Changing the α immediately provides a new graph and a new value of the correlation coefficient. The value of α that provides the straightest line and the highest correlation coefficient is the best value for α . In our example this proves to be $\alpha = 1.24$, resulting in $\beta = 1.17$ and $\gamma = 1.22$. See also Table A1-2 and Figure A1-8. (Note: β and γ are not the values given in the regression line in Figure A1-8, because in this figure H_{ss} is plotted on the vertical axis; if one plots H_{ss} on the horizontal and W on the vertical, one gets the values of β and γ also directly).

Weibull distribution

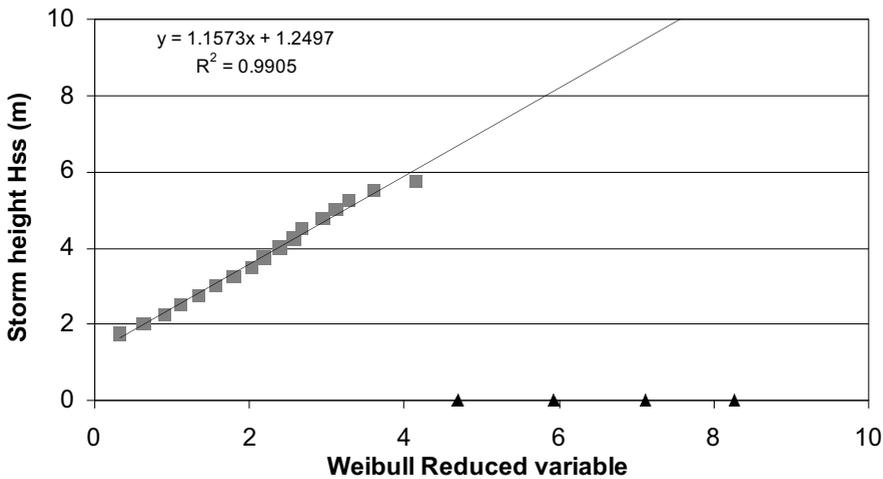


Figure A1-8 The Weibull exceedance graph.

Also here the exceedance has to be transformed to a probability per year. From the Weibull distribution it follows:

$$H_{ss} = \gamma + \beta \{-\ln Q\}^{1/\alpha} \quad (\text{A1.12})$$

Using $Q_s = H_s Q$ gives

$$H_{ss} = \gamma + \beta \left\{ -\ln \left(\frac{Q_s}{N_s} \right) \right\}^{1/\alpha} \quad (\text{A1.13})$$

So:

$$\begin{aligned} H_{ss}^{1/225} &= 1.22 + 1.17 \left\{ -\ln \left(\frac{1/225}{87.3} \right) \right\}^{1/1.24} \\ &= 1.22 + 1.17 \cdot 6.34 \\ &= 8.64 \text{ m} \end{aligned}$$

Again, a manual plot has to be made of H_{ss} as a function of W (see also Figure A1-8). This can be transformed using

$$W = -\ln \left(\frac{Q_s}{N_s} \right)^{1/\alpha} \quad (\text{A1.14})$$

So, for the given example (with $N_s = 87.3$ and $\alpha = 1.24$), Q_s and W are:

Q_s	W
1/10	4.68
1/100	5.92
1/1000	7.11
1/10000	8.24

Table A1-4.

Summary

The coefficient γ theoretically has the meaning of the threshold value H_T as used in the PoT-analysis. So as a check, we can compare these values:

H_T threshold value	1.50	2.00	2.50	3.00	3.50	4.00
γ Exponential	1.63	1.65	1.65	1.65	1.65	1.65
γ Gumbel	1.86	2.37	2.84	3.44	3.98	4.52
γ Weibull	1.23	1.77	2.29	2.93	3.51	4.10

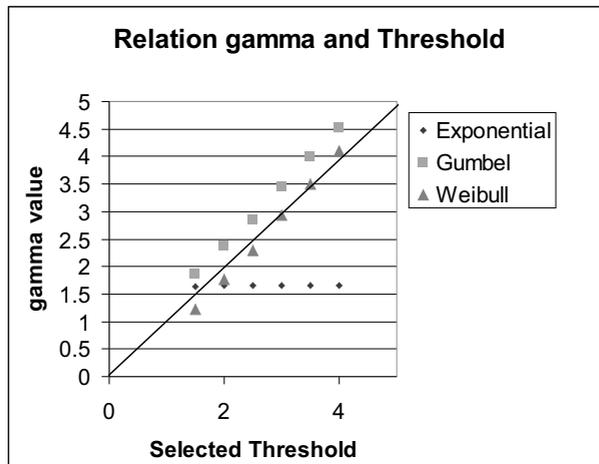
Table A1-5.

In this respect it is clear that the Gumbel and the Weibull distribution give a much better result. This becomes more relevant when the number of available storms in the database is lower. Summarizing we find the following H_{ss} values for our 1/225 design storm for different threshold values:

H_T		1.50	2.00	2.50	3.00	3.50	4.00
	N_s	87.3	59.6	38.9	19.4	10.5	5.3
$H_{ss\ 1/225}$	Exponential	9.39	9.30	9.00	8.85	8.58	8.31
$H_{ss\ 1/225}$	Gumbel	9.10	8.95	8.66	8.46	8.11	7.81
$H_{ss\ 1/225}$	Weibull	8.64	8.55	8.32	8.20	7.97	7.77

Table A1-6.

In this comparison for the Weibull distribution a value $\alpha = 1.24$ has continuously been used. As explained before, α has to be determined by optimizing the correlation coefficient and visually on the fact that the points are on a straight line as much as possible. For easy reference we have continued to use $\alpha = 1.24$ as derived for a H_T of 1.5 m. For large values of H_T the number of wave height bins available for use in the analysis becomes lower. Consequently, the calculation becomes less reliable.

Figure A1-9: Relation between γ and the selected threshold.

A1.3 What to do if only random data are available?

The above PoT-analysis can *only* be carried out in case a *sequential* database with observations is available. In case only the grouped statistics of observations are available (like Table A1-1) a PoT-analysis is not possible. The same problem occurs when data are collected by random observations, like the visual observations from the Global Wave Statistics³. Often the number of observations in each wave class bin is normalized in such a way that the total is 100% or 1000 %.

³ The Global Wave Statistics [HOGBEN & LUMB, British Maritime Technology, 1986] is a book containing visual observations collected by ships at given times, but random locations (i.e. the position of the ship at that moment). This makes that all observations in a given area are completely uncorrelated.

<i>H_s</i> -bin		Class		<i>P</i>	<i>Q</i>	LN(<i>Q</i>)	12 hrs	storm duration		$\alpha = 0.8$
	Obs	Cumul	<i>s/y</i>				ln()	ln(<i>H</i>)	<i>W</i>	
0	25	35	35	0.00060	0.99940	0.00	729.56	6.592	3.219	-10.564
25	50	8260	8295	0.14194	0.85806	0.15	626.38	6.440	3.912	-10.259
50	75	11424	19719	0.33742	0.66258	0.41	483.68	6.181	4.317	-9.747
75	100	10004	29723	0.50861	0.49139	0.71	358.72	5.883	4.605	-9.161
100	125	7649	37372	0.63949	0.36051	1.02	263.17	5.573	4.828	-8.562
125	150	5563	42935	0.73469	0.26531	1.33	193.68	5.266	5.011	-7.978
150	175	4389	47324	0.80979	0.19021	1.66	138.85	4.933	5.165	-7.353
175	200	3167	50491	0.86398	0.13602	1.99	99.29	4.598	5.298	-6.733
200	225	2360	52851	0.90436	0.09564	2.35	69.81	4.246	5.416	-6.095
225	250	1671	54522	0.93296	0.06704	2.70	48.94	3.891	5.521	-5.464
250	275	1234	55756	0.95407	0.04593	3.08	33.53	3.512	5.617	-4.808
275	300	851	56607	0.96863	0.03137	3.46	22.90	3.131	5.704	-4.165
300	325	556	57163	0.97815	0.02185	3.82	15.95	2.770	5.784	-3.573
325	350	392	57555	0.98486	0.01514	4.19	11.05	2.403	5.858	-2.992
350	375	276	57831	0.98958	0.01042	4.56	7.61	2.029	5.927	-2.422
375	400	206	58037	0.99310	0.00690	4.98	5.03	1.616	5.991	-1.822
400	425	136	58173	0.99543	0.00457	5.39	3.34	1.205	6.052	-1.262
425	450	82	58255	0.99683	0.00317	5.76	2.31	0.838	6.109	-0.801
450	475	66	58321	0.99796	0.00204	6.20	1.49	0.396	6.163	-0.315
475	500	38	58359	0.99861	0.00139	6.58	1.01	0.012	6.215	-0.004
500	525	30	58389	0.99913	0.00087	7.04	0.64	-0.451	6.263	0.369
525	550	20	58409	0.99947	0.00053	7.54	0.39	-0.949	6.310	0.936
550	575	22	58431	0.99985	0.00015	8.78	0.11	-2.185	6.354	2.657
575	600	9	58440	1	0	0	0	0	0	0

Table A1-7 Uncorrelated data (example from Noordwijk, but not taking into account the fact that they are observed every three hours).

A typical sample of such a table is given in Table A1-7 (In fact the input data are identical to Table A1-1). In such a case, one can determine the exceedance frequency of a single H_s , as indicated in the section before. Because the data are not sequential, and because the number of storms per year (N_s) is not known, the PoT-analysis is not possible. Some of the data in the table may come from the same storm.

In order to solve this problem, we assume that we can divide the year in N_s periods of t_s hours, during which the wave height H_s does not vary. The basic idea behind this assumption is that, because of the persistence of winds, storm periods will have more-or-less the same duration. Therefore, the assumed time interval t_s is called "storm duration". Now, one can stipulate that each of the random observations describe an observation of one such storm.

This means that we usually have an unknown number of observations; however we know the percentage of exceedance of each observation. Because such an observation does in fact not represent the wave height of one (half-hour) sample (which we called H_s), but represents the average H_s during a t_s -hour storm, we will use the symbol H_{ss} for this observation.

Let us assume for the time being that we have a storm-duration of 12 hours. This means that we have $365 \cdot 24/12 = 730$ periods of 12 hours in a year ($N_s = 730$). This means that a “once-per-year” storm occurs in $1/730$ times of the cases ($=1.37 \cdot 10^{-3}$). And a “once-in-ten-year” storm occurs in $0.137 \cdot 10^{-3}$ times of the cases.

In the example one can see that observations with $H_{ss} > 4.75$ m occur in 0.001386 of the cases. It is clear that the “once-a-year” storm is approximately 4.75 m. The value $Q_{ss} = Q_s \cdot 730$ indicates the probability of a storm in a year.

It is obvious that from a statistical point of view values of $Q_s > 1$ have no meaning. However, physically this number indicates the number of storms per year.

The values of Q_{ss} can be plotted on log paper, which results directly in a design line, showing the probability of exceedance of a given design storm.

Statistically one can process the values of Q_s in the same way as has been done before with Q . Practically this computation can be done in a spreadsheet in a simple way. In fact, Table A1-3 is a print from a spreadsheet. A multiplier is computed with the value:

$$M = \frac{\text{number of observations}}{365 \cdot 24 / \text{storm duration}} = \frac{58440}{730} = 80$$

Now we calculate the total number of storms smaller than our bin-value by taking {total-(cumulative number)} and divide that by M . The result is given in the column with the heading s/y (storms per year). And this value can be plotted, resulting in Figure A1-10.

Also in this case, we calculate the slope and intercept of the regression line. In our example the slope (A) is 1.02, and intercept (B) equals 4.16. (In order to calculate these values, one should either make an extra column with $\ln Q_s$ or make a plot where Q_s is on the y axis and H_{ss} on the x-axis). For extrapolation one can use the equation:

$$Q = \exp\{-\beta(H_{ss} - \gamma)\} \quad (\text{A1.15})$$

or in reversed order:

$$H_{ss} = \gamma - \frac{\ln Q}{A} \quad (\text{A1.16})$$

in which A = slope of regression line and $\gamma = -B/A$ ($= 4.16/1.02 = 4.07$).
So, the 1/225 storm-height is

$$H_{ss \frac{1}{225}} = 4.07 - \frac{\ln\left(\frac{1}{225}\right)}{1.02} = 9.39 \text{ m}$$

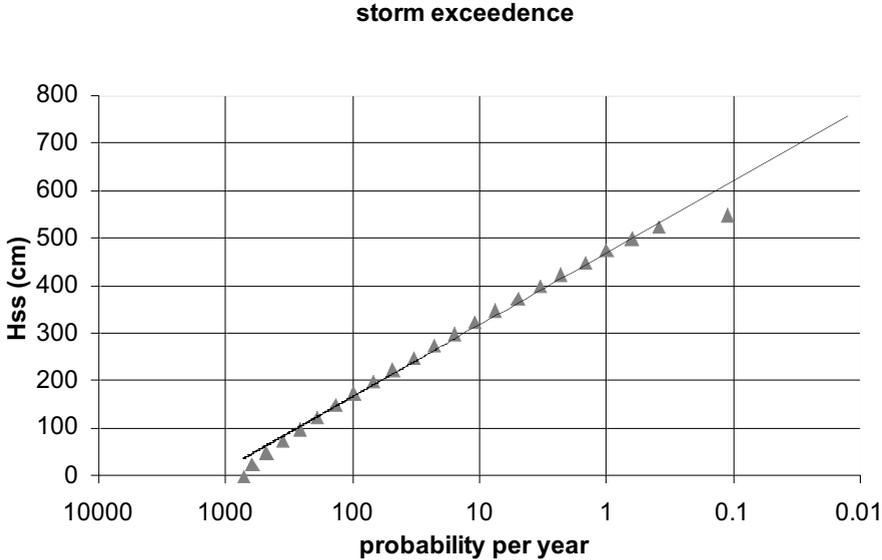


Figure A1-10 Storm exceedance graph using Random Observations.

The value of 9.39 found in this way should be compared with the values between 8.31 and 9.39 found in the PoT analysis. This analysis too, can be improved by using the Weibull distribution instead of a simple log-distribution.

As was done with the PoT-analysis, the value of W_s is calculated using

$$W_s = \left(\ln \frac{1}{Q_s} \right)^{1/\alpha} \tag{A1.17}$$

See Table A1-3. At some place in the spreadsheet, a value of α is given and the data of the last column are calculated using that value of α . One can easily calculate slope, intercept and correlation. With trial and error, one can find that value of α which results in the highest correlation. In this example a value of $\alpha = 0.85$ (which is different than from the previous case) gives a correlation of 99.8 %. The parameters of β and γ follow from the slope and intercept of the regression line:

$$\beta = 1/A$$

$$\gamma = -B/A$$

The values of α , β and γ can be used directly in the equation to calculate the probability:

$$W_s = \exp\left\{-\left(\frac{H_{ss} - \gamma}{\beta}\right)^\alpha\right\} \quad (\text{A1.18})$$

or in reversed order:

$$H_{ss} = \gamma + \beta(-\ln W_s)^{1/\alpha} \quad (\text{A1.19})$$

So, the 1/225 storm height is:

$$H_{ss}^{1/225} = 4.62 + 0.43\left[-\ln\left(\frac{1}{225}\right)\right]^{1/0.85} = 8.16 \text{ m}$$

One can make a plot of H_{ss} vs. W_s . This indicates that all points are nicely on a straight regression line. However, the value of W_s has no direct technical meaning. It becomes useful in case W_s is translated into Q_s using:

$$W_s = \left(\ln \frac{1}{Q_s}\right)^{1/\alpha} \quad (\text{A1.20})$$

This means that one has to redefine the horizontal axis. Be aware that values of $W < 0$ do not represent probabilities, but are only used to have a sufficient basis for extrapolation.

The resulting value of $H_{ss} = 8.16$ m should be compared with the values between 7.77 and 8.64 found in the PoT analysis. As stated before, the anticipated storm duration was 12 hours. Different durations result in different values for H_{ss} .

	3 hrs	6 hrs	9 hrs	12 hrs
Exponential	5.41	6.73	8.06	9.39
Weibull	8.48	8.32	8.23	8.16

Table A1-8.

From this example it follows that in case a PoT-analysis is not possible, an analysis on the basis of tabulated (random) data can very well be done. The choice of the storm duration however remains a problem. Given the accuracy of the final answer, this choice is not extremely sensitive, at least when using the Weibull distribution.

A1.4 Computation of the armour units

A1.4.1 The classical computation

The classical way of computing the required block size is using the design formula, applying a design wave height with an exceedance of P_f during the lifetime of the structure. For example a lifetime of 50 years and a probability of failure of 20% during lifetime gives the following exceedance (Poisson Distribution, see also SCHIERECK 2001, Chapter 10):

$$f = -\frac{1}{t_L} \ln(1-p) = -\frac{1}{50} \ln(1-0.2) = 4.5 \cdot 10^{-3} = \frac{1}{225} \quad (\text{A1.21})$$

Using the Weibull distribution from Section A1.2 for the example of Noordwijk, this results in a H_{ss} of 8.64 m. The design formula, as given by Van der Meer for cubes is:

$$\frac{H_{ss-design}}{\Delta d_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0 \right) s_m^{-0.1} \quad (\text{A1.22})$$

For N_{od} Van der Meer recommends a value of 0.5. The wave analysis in Chapter 5 has shown a wave steepness of 5.6%. There are approximately 4000 waves in a storm. For cubes with normal concrete one may use a concrete density of 2400 kg/m³, which results in a value of $\Delta = 1.75$.

Substituting these values in the above design formula yields a d_n of 2.4 m, or a block weight of 91 tons. In case one applies the Hudson formula with a K_D value of 5 (head) and a slope of 1:1.5, one gets a d_n of 3.3 m, and a weight of 83 tons (realise that the use of basalt split in the concrete may increase the density to 2800 kg/m³, which will give with Van der Meer a block weight of "only" 50 tons).

Because the depth in front of the Scheveningen breakwater is limited to 6 m below MSL (i.e. 9.5 m below design level), one may assume that waves never can become larger than $0.5 \cdot 9.5 = 4.75$ m (see Figure 5-26). Filling in these values results in a block weight of 15 tons.

However, one should realize that the number of occurrence of a storm with an H_{ss} of more than 4.75 is much more frequent. Using the Weibull distribution, one can find that a H_{ss} of 4.75 is exceeded once every 0.6 years. This means that during the design life (50 years) the breakwater will encounter $50/0.6 = 85$ storms, with in total $400 \cdot 85 = 33000$ waves. Using this number of waves, we find a block size of 2.1 m, and a weight of 24 tons. Because the number of waves is not included in the Hudson formula, Hudson gives a block weight of only 14 tons.

During the design of the breakwater of Scheveningen, only the Hudson formula was available; the design of this breakwater however was not only based on this formula but also verified with model tests. The applied blocks in Scheveningen have a weight of 25 tons and a density of 2400 kg/m³.

A1.4.2 The method of Partial Safety Coefficients

The method of partial coefficients is worked out in PIANC [1992]. In this method safety coefficients are added to the design formula. There are safety coefficients for load and for strength.

The partial safety coefficients for load

The probability of exceedance during service life of the design storm with a recurrence equal to the design life

$$P_{f\text{-lifetime}} = 1 - \left(1 - \frac{1}{t_L}\right)^{t_L} \quad (\text{A1.23})$$

in which t_L is the design life. For a life time of 50 years, $P_{f\text{-lifetime}}$ is 0.64

The design storm to be applied has of course a much smaller probability, and consequently a longer return interval. The probability that a construction may fail during life time P_f is for example set to 20 %. The return period of the design storm then becomes

$$t_{Pf} = \left(1 - (1 - P_f)^{1/t_{LL}}\right)^{-1} \quad (\text{A1.24})$$

which gives 225 years according to our example.

To determine the partial safety coefficient one has to start with an extreme value distribution, for example the Weibull distribution. For this purpose the distribution is given as:

$$Q_{tL} = \left\{1 - \exp\left[-\left(\frac{H_{ss} - \gamma}{\beta}\right)^\alpha\right]\right\}^{N_s t_L} \quad (\text{A1.25})$$

In this equation α and β are the parameters of the Weibull distribution, and γ is the threshold value. N_s is the number of observations per year.

This equation can be reworked to

$$H_{ss} = \gamma + \beta \left[-\ln \left\{ 1 - \exp \left(\frac{\ln Q_{tL}}{N_s T_L} \right) \right\} \right]^{1/\alpha} \quad (\text{A1.26})$$

For Q_{tL} one should enter the non exceedance probability for $N_s t_L$ events during lifetime. This means that $Q_{tL} = P_{f\text{-lifetime}}$. For a design life of 50 years Q_{tL} is 0.364, for 100 years Q_{tL} is 0.366.

For a practical case this leads to the following values of H_{ss} :

Input from wave climate (from Meetpost Noordwijk):

α	1.24
β	1.17
γ	1.22
N_s	87.3

		lifetime 50 years	lifetime 100 years
H_{ss} for $t = t_L$ (50, 100)	H_{ss}^{1L}	7.71	8.15
H_{ss} for $t = 3 t_L$ (150, 300)	H_{ss}^{3L}	8.39	8.81
H_{ss} for $t = t_{20\%}$ (225, 450)	H_{ss}^{tpf}	8.64	9.06

Table A1-9.

The safety coefficient is given as:

$$\gamma_{H_{ss}} = \frac{H_{ss}^{tpf}}{Q_{tL}} + \sigma'_{Q_{tL}} \left(1 + \left(\frac{H_{ss}^{3L}}{H_{ss}^{1L}} - 1 \right) k_\beta P_f \right) + \frac{0.05}{\sqrt{P_f N}} \quad (\text{A1.27})$$

In this equation P_f is the allowable failure during lifetime (not to be mistaken with $P_{f\text{lifetime}}$, which has been defined as the probability of exceedance of the “once in t_L -years storm” during the lifetime t_L).

The standard deviation $\sigma'_{Q_{tL}}$ is given in PIANC report [PIANC 1992] (copied in Table A1-10) as a function of the type of observations available. The data from Noordwijk are based on accurate observations, so a value of $\sigma'_{Q_{tL}} = 0.05$ is realistic.

P_f was 20%, N is the number of “storms” in the PoT-analysis, for this example it is 1746. $k_\alpha = 0.027$ and $k_\beta = 38$ (see Table A1-2).

This leads to:

$$\gamma_{H_{ss}} = \frac{8.64}{7.71} + 0.2 \left(1 + \left(\frac{8.39}{7.71} - 1 \right) 38 \cdot 0.2 \right) + \frac{0.05}{\sqrt{0.2 \cdot 1746}} = 1.13 \quad (\text{A1.28})$$

The safety coefficient consists of three parts. The first term gives the correct partial safety coefficient, provided no statistical uncertainty and measurement errors related

to H_{ss} are present. The second term signifies the measurement errors and the short-term variability related to the wave data. The last term signifies the statistical uncertainty of the estimated extreme distribution of H_{ss} . The statistical uncertainty treated in this way depends on the total number of wave data N , but not on the length of the observation period.

Parameters	Method of determination	Typical value for σ'
<i>Wave height</i>		
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	Numerical models	0.1-0.2
H_{ss} taking into account typical nearshore effects (refraction, shoaling, breaking)	Manual calculation	0.15-0.35
<i>Other wave parameters</i>		
Mean wave period offshore on condition of fixed H_{ss}	Accelerometer buoy	0.02-0.05
	Estimates from amplitude spectra Hindcast, numerical model	0.15
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
	Pitch-roll buoy	0.4
Mean direction of wave propagation offshore	Measurement of horizontal velocity components and waterlevel or pressure	5°
	Hindcast, numerical model	10°
		15-30°
<i>Water level</i>		
Astro tides	prediction from constants	0.001-0.07
storm surges	numerical models	0.1-0.25

Table A1-10 Typical variation coefficients for sea state parameters [from PIANC 1992].

If extreme wave statistics are not based on N wave data, but for example on estimates of H_{ss} from information about water level variations in shallow water, then the last term disappears and instead the value chosen for σ' must account for the inherent uncertainty.

In Table A1-11 the values of σ' and N have been changed to the values for simple manual calculations and a shorter dataset. It is clear from this table that the effect of the length of a dataset is less important than accurate observations.

	base example	use $\sigma' = 0.35$	use $N = 10$ storms	use $\sigma' = 0.35$ and $N = 10$
basic safety coefficient	100%	87%	99%	84%
measurement and short term errors	0%	13%	0%	13%
statistical uncertainty	0%	0%	1%	3%

Table A1-11.

The partial safety coefficient for strength

The safety coefficient for the strength can be calculated using

$$\gamma_z = 1 - (k_\alpha \ln P_f) \quad (\text{A1.29})$$

in which k_α and k_β are coefficients determined by optimisation and given in the PIANC manual [PIANC 1992]. These values are copied in Table A1-12. The value of P_f is the allowable probability of failure during lifetime.

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 A1-12 Coefficients used to determine the partial safety factor γ_z .

In the Coastal Engineering Manual the same approach is followed, however in that manual the values of γ_H and γ_z are directly given as a function of P_f and σ' .

For cubes, one can apply the Van der Meer equation:

$$\frac{1}{\gamma_z} \left(6.7 \frac{N_{od}^{0.4}}{N^{0.30}} + 1.0 \right) s_m^{-0.1} \Delta d_n \geq \gamma_{Hss} H_{ss}^{tL} \quad (\text{A1.30})$$

For the harbour of Scheveningen, one may use the wave data of Noordwijk. Filling in values of $N_{od} = 1$, $N = 1500$, $\Delta = 1.75$, $s_m = 2.5\%$, this gives $d_n = 2.07$, or 25 tons.

Notice that in the above equation for the wave height the H_{ss} is used which has a probability of exceedance of once in the lifetime of the structure, i.e. the “once in 50 years storm” (7.71 m). This wave height is multiplied by γ_H (1.13), resulting in a total wave height of 8.71 m.

Traditionally one should use a wave height with a probability of 20% during the lifetime of 50 years. This wave has a yearly exceedance of $1/225 = 4.4 \cdot 10^{-3}$.

See also Chapter 3. The “once in 225 years wave” is 8.64 m (in the PIANC guidelines it is called H_{ss}^{ipf}) and compares quite well to the calculated value of 8.71 m.

Note that in the latter example, the limited water depth has not been taken into account.

A1.4.3 Probabilistic approach

Instead of the method with partial safety coefficients, one may apply a full probabilistic computation, either on level 2 or level 3. For level 2 one may apply the FORM method, for level 3 one may apply the Monte-Carlo method. In the examples below the computer program VaP from ETH-Zürich has been applied.

The first step is to rewrite the design equation as a limit state function (a Z-function). For cubes the Z-function is:

$$Z = \left(A \frac{N_{od}^{0.4}}{N^{0.30}} + 1.0 \right) s_m^{-0.1} \Delta d_n - H_{ss}^{tL} \quad (\text{A1.31})$$

In this equation seven variables are used. In a probabilistic approach one has to determine the type of distribution for each parameter.

The constant 6.7 from the Van der Meer equation is replaced here by a coefficient A with a normal distribution. This coefficient has a mean of 6.7 and a standard deviation describing the accuracy of the equation itself. According to Van der Meer the standard deviation of the coefficient A is approximately 10% of its value.

Wave steepness is assumed to have a Normal distribution. The average steepness as well as the standard deviation of the steepness can be calculated from the dataset of Meetpost Noordwijk. If one considers only the heavier storms (i.e. storms with a threshold of e.g. 4.5 m), the average steepness is 0.058, with a standard deviation of 0.0025. Realise that a zero correlation between steepness and wave height is assumed, which is not unrealistic.

We then have 56 storms in 20 years (i.e. 2.8 storms per year) with an average duration of 7.2 hours. This means that the average period is 6.9 seconds, and that there are consequently 3700 waves in a storm. However, the duration of the storms varies quite a lot (it may go up to 20 hours, which means that we have 10000 waves. This means that N will have a large standard deviation. One can fit the calculated waves in each storm and fit this to a distribution. However, the effect of the number of waves is not that high so one may assume a Lognormal distribution (one cannot use a Normal distribution, because the number of waves may become negative when using high standard deviations).

Because the standard deviation in the block size (concrete cubes) is so small, this parameter can be considered as a deterministic value. The acceptable damage level is a target value, therefore this should also be a deterministic value.

H_{ss} has a Weibull or Gumbel distribution. For the wave-height one may enter for example a Weibull distribution, using the values of α , β and γ as determined before.

This results in the following input table:

parameter	type	mean	standard deviation
A	Normal	6.7	0.67
N_{od}	Deterministic	1	-
N	Lognormal	3000	3000
s_m	Normal	0.058	.0025
Δ	Normal	1.75	0.05
d_n	Deterministic	2.4	-

Table A1-13.

One can compute the probability of ($Z < 0$), which is the probability of failure.

The target probability is $1/225 = 0.0044$ per year. However, VaP gives the probability per event. Because we have 87.3 storms per year, the target probability per event becomes $0.0044 / 87.3 = 50 \cdot 10^{-6}$. The weight of a cube of 2.4 m is 37.5 tons.

For this example using the FORM method a probability of failure of $45 \cdot 10^{-6}$ is found, which is quite near the target value. A Monte-Carlo computation gives a probability of failure of $70 \cdot 10^{-6}$, so quite comparable.

This computation is based on the fact that we have defined $N_{od} = 1$ as start of failure. However, this is quite some damage. Lowering the value of N_{od} to 0.5 means that we have to increase the block size to 2.65 m (39 tons) in order to obtain the same probability of failure. Using a N_{od} of 0 means an increase to 3.80 m (44 tons).

Limit State Function				
$G = (A \cdot N_{od}^{0.4} / N^{0.3+1}) \cdot s^{-0.1} \cdot \Delta \cdot D_n - H_{ss}$				
Variables				
Variables of G:				
A	N	6.700	0.670	
Delta	N	1.750	0.050	
Dn	D	2.400		
Hss	WS	2.390	1.240	1.220
N	LN	7.660	0.833	
Nod	D	1.000		
s	N	0.058	0.003	

Figure A1-11.

In the above calculations the uncertainty in the determination of the parameters of the Weibull distribution was not taken into consideration. Although this is common practice, it is not fully correct. The mathematically correct way is to consider α , β and γ as stochastic parameters with a mean and a standard deviation. One can determine these values directly from the dataset, but it is not possible to apply these values directly in a probabilistic computation. In practice this problem is solved by introducing an extra variable M . This variable has a mean value of 1 and a standard deviation which expresses the variation in the prediction of H_{ss} using a Weibull or Gumbel distribution. The Van der Meer formula then becomes:

$$Z = \left(A \frac{N_{od}^{0.4}}{N^{0.30}} + 1.0 \right) s_m^{-0.1} \Delta d_n - MH_{ss}^{tL} \quad (A1.32)$$

A problem is that the standard deviation of M depends on the value of H_{ss} . One can determine this value using the design value for H_{ss} .

The standard deviation is given as:

$$\sigma'_M = \frac{\sigma_M}{H_{ss-design}} \quad (A1.33)$$

For σ_M an expression has been derived by Goda [Goda, 2000]:

$$\sigma_M = \sigma_z \sigma_x \quad (A1.34)$$

in which σ_x is the standard deviation of all H_{ss} values in the basic dataset and σ_z is defined by:

$$\sigma_z = \frac{\left[1.0 + a(y-c)^2 \right]^{1/2}}{\sqrt{N}} \quad (A1.35)$$

$$a = a_1 \exp \left[a_2 N^{-1.3} \right]$$

in which the coefficients are given by:

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

Table A1-14.

y is the reduced variate for the design value, our design value is a 1/225 wave, so $y = [\ln(87.3 \cdot 22)]^{1/1.24} = 6.34$.

In the example used, we have a Weibull with $\alpha = 1.24$; because this value is not in the table, an interpolation is needed. For the example the following values were being used: $a_1 = 2.0$, $a_2 = 11.4$ and $c = 0.35$.

This results in a value of $\sigma_z = 0.024$. Given a σ_x of 6.85 (follows from dataset), this means:

$$\sigma'_M = \frac{\sigma_M}{H_{ss-design}} = \frac{\sigma_x \sigma_z}{H_{ss-design}} = \frac{6.85 \cdot 0.024}{8.64} = 0.02 \quad (\text{A1.36})$$

A recalculation with these figures gives a probability of failure of $157 \cdot 10^{-6}$. In order to bring this back to the required $50 \cdot 10^{-6}$ we have to increase the cube size to 2.42 m. So, this can be neglected.

However, if our dataset would have been considerably smaller (for example only 100 storms), this would change the value of σ_z to 0.175 and consequently σ'_M to 0.14. In order to get a probability of failure in the order of the required $50 \cdot 10^{-6}$ we have to increase the cube size to 2.70 m (from 37.5 to 55 tons). Again this is for deep (18 m) water conditions.

This shows that the size of the dataset has a considerable impact on the required block size.

A1.4.4 Probabilistic calculation in case of a shallow foreshore

Statistical software does discard physical limitations. E.g., applying statistics in deep water circumstances will give very high waves with very low probability. But in shallow water these high waves cannot exist at all. In general the significant wave height in shallow water is limited by the depth. So:

$$H_{ss} = \gamma_b h \quad (\text{A1.37})$$

in which the breaker index γ_b has a value in the order of 0.6. For individual waves γ_b may have values up to 0.78, but for the significant wave height this value is much lower, sometimes even down to 0.45.

For foreshores with a gentle slope one may assume that γ_b may have an average value of 0.55, with a standard deviation of 0.05. The water depth in this equation is the total depth, i.e. the depth below mean sea level + the rise of water level due to tide and storm surge.

With this information, one can rewrite the Van der Meer equation for cubes to:

$$Z = \left(A \frac{N_{od}^{0.4}}{N^{0.30}} + 1.0 \right) s_m^{-0.1} \Delta d_n - \gamma_b (h_{surge} + h_{depth}) \quad (\text{A1.38})$$

The water depth below mean sea level has a Normal distribution, but the standard deviation in this parameter is usually so low, that it can be considered as a deterministic value. Of course in case large bed fluctuations are to be expected, one may also enter this value as a stochastic parameter with a Normal distribution. The surge has an extreme value distribution. For this value one can apply a Gumbel distribution.

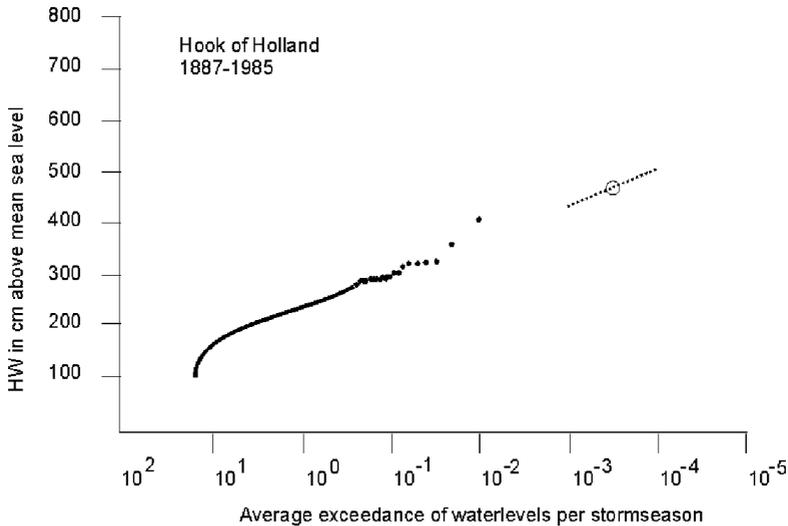


Figure A1-12 Extreme water levels in Hook of Holland.

Again we look at the example of Scheveningen harbour. The water depth in front of the breakwater in Scheveningen is 6 m below mean sea level. Long term water levels are available from Hook of Holland, which are approximately identical to Scheveningen. The exceedance can be given by:

$$Q = 1 - \exp \left[- \exp \left(- \frac{h_{surge} - \gamma}{\beta} \right) \right] \quad (A1.39)$$

In this equation is γ the intercept at the 10^0 -line, and β is the slope parameter. From the diagram one can derive that γ equals 2.3 and that the slope β is 0.30.

The equation is based on the maximum surges per year, so the exceedance is also per year, and not per storm. This implies that in this case the target probability of failure is $1/225 = 0.0044$.

This leads to $d_n = 1.85$ m, or a block weight of 17.7 tons.

In principle one can in this case also add the statistical uncertainty by adding a factor M in front of the surge height in the equation. However, because of the long dataset and the limited extrapolation, this uncertainty is very small and may be neglected.

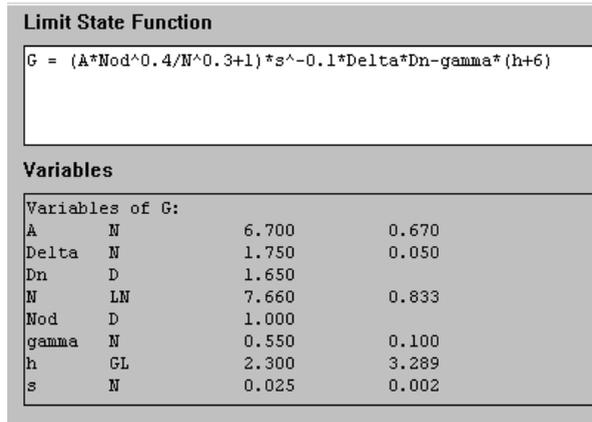


Figure A1-13.

Changing the standard deviation in γ from 0.05 to 0.1 has a considerable effect. To obtain the same target probability of failure, one has to increase the blocks to $d_n = 2.2$ m (30 tons).

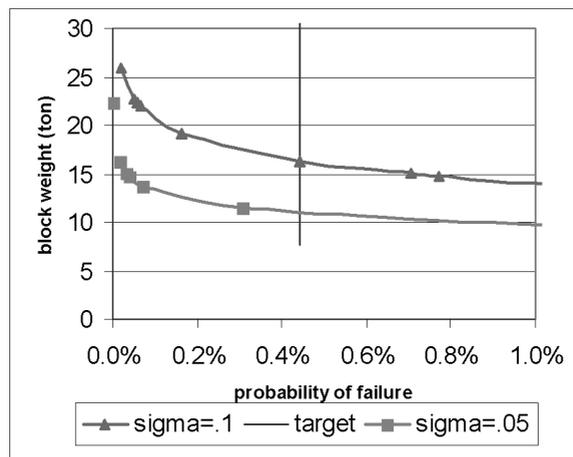


Figure A1-14 Effect of standard deviation of γ_0 on block weight.

A simplification made in this computation is that it is assumed that in deep water, wave steepness remains the same after breaking. This is probably not the case. The higher waves will break (usually as spilling breakers) which decreases their height considerably, but usually not the period. As a consequence the wave steepness in broken waves is much less than the wave steepness at sea. Because in the Van der Meer formula for cubes a low steepness gives smaller cubes than a high steepness, neglecting the change in steepness is a conservative approach. Be aware that in case of natural rock the opposite is true.

APPENDIX 2

Quarry operations

A2.1 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 one has to start with a reconnaissance for a suitable quarry. This is rather specialised work for a engineering geologist. In 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:

A2.1.1 Joints (see Figure A2-1)

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 bedding-joints, foliation joints and cleavage-joints accordingly.

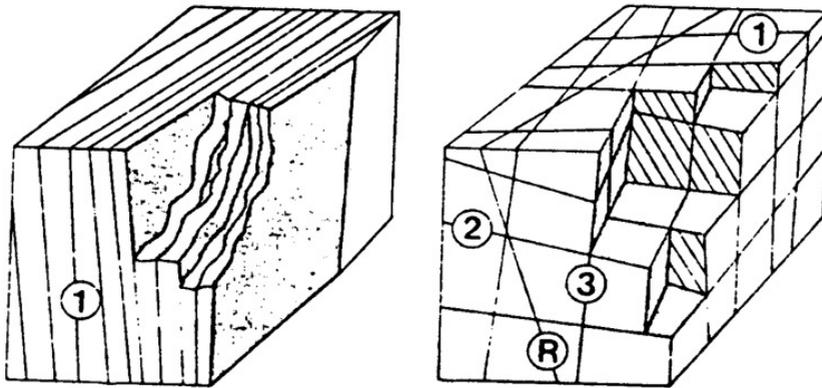


Figure A2-1 Types of joints.

A2.1.2 Fault (see Figure A2-2)

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.

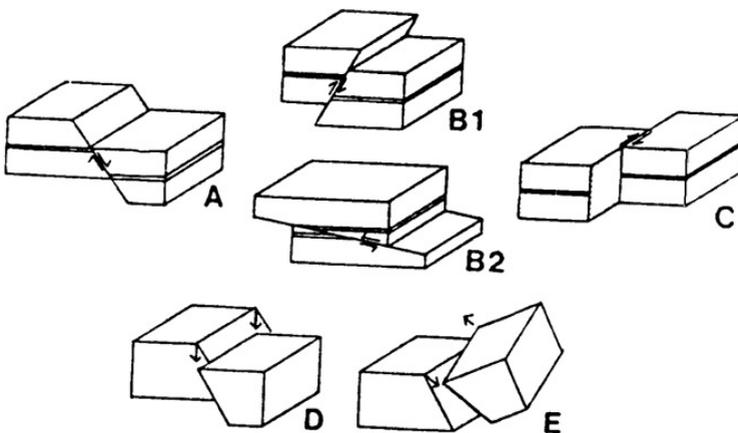


Figure A2-2 Types of faults.

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.

A2.1.3 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 schistosity-planes, weakness zones and faults.

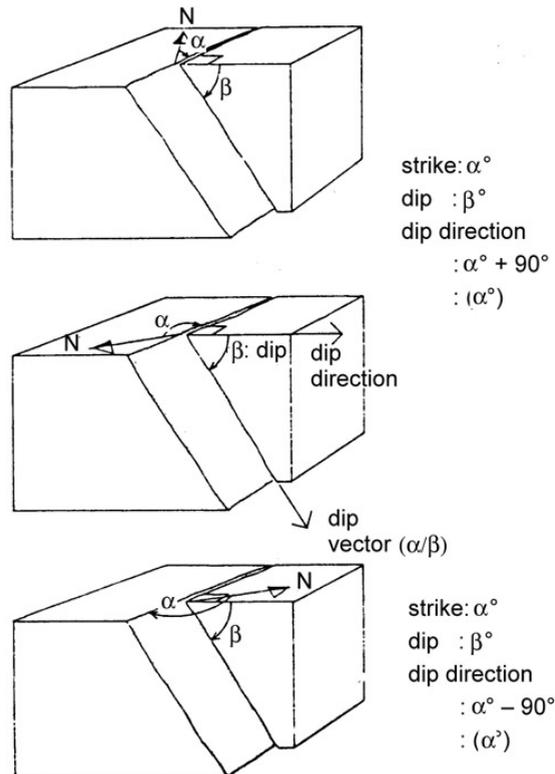


Figure A2-3 Attitude of discontinuity.

The ten parameters selected to describe discontinuities and rockmasses are as follows:

1) Orientation:

Attitude of discontinuity in space (see Figure A2-3)

2) Spacing:

Perpendicular distance between adjacent discontinuities (see and Figure A2-4).

3) Persistence:

Discontinuity trace length as observed in an exposure.

4) Roughness:

Inherent surface roughness relative to the mean plane of a discontinuity.

5) Wall strength:

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

6) Aperture:

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

7) Filling:

Material that separates the adjacent rock-walls of a discontinuity and that is usually weaker than the parent-rock.

8) Seepage:

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

9) Number of sets:

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

10) Block-size (see Figure A2-5):

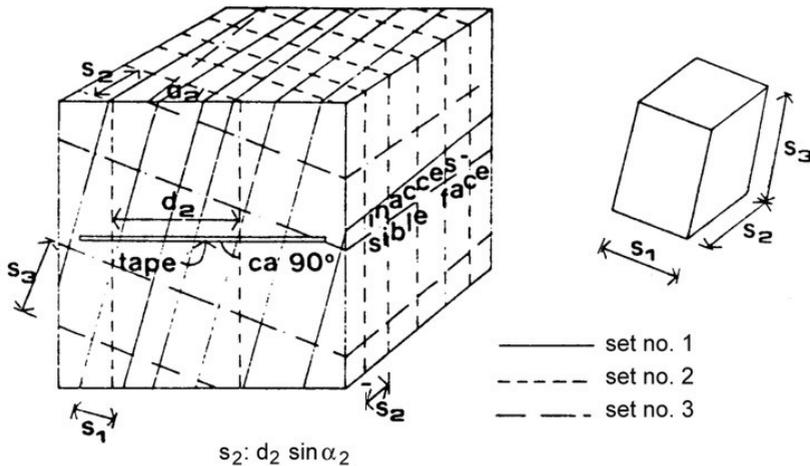


Figure A2-4 Distance between adjacent discontinuities.

Rock-block dimensions resulting from the mutual orientation of intersecting joint sets and resulting from the spacing of the individual sets. Individual discontinuities may further influence the block and the shape. Block-size can be described either by means of the average dimension of typical blocks (block-size index l_b) or by the total number of joints intersecting a unit volume of the rock mass (Volumetric Joint Count J_v) (see Table A2-1)

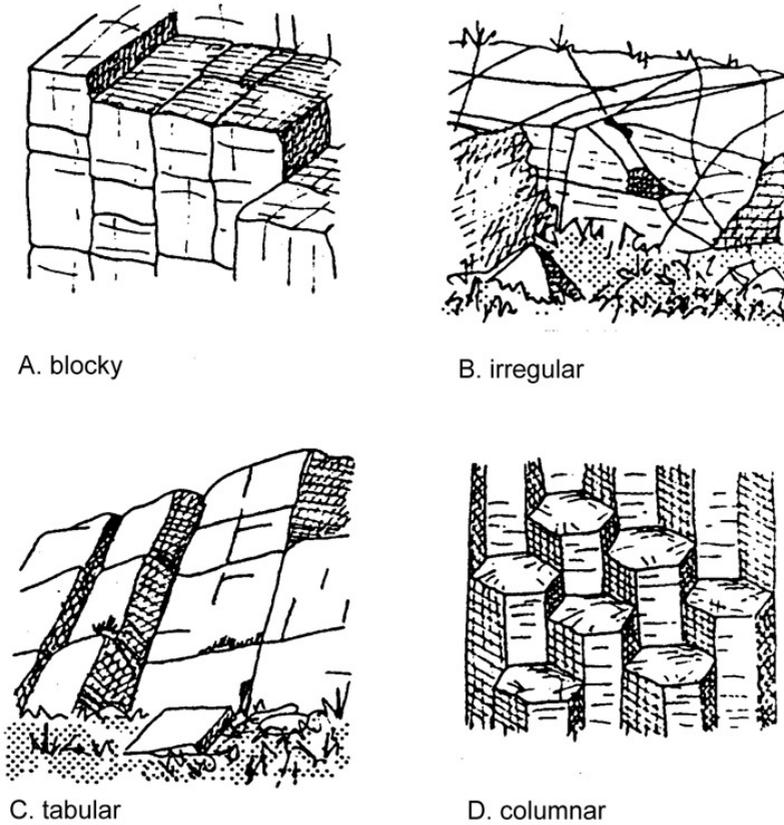


Figure A2-5 Block-size.

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 A2-1 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 A2-7 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
- in the near vicinity of the quarry sufficient space should be available to open work yards, depots, etc.

A2.2 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 armour stones 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.

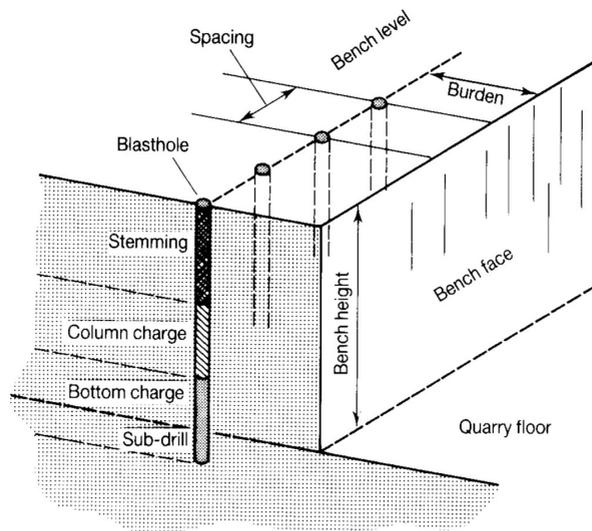


Figure A2-6 Blasting lay-out.

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 blasthole 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 A2-2 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
Hornafjörður	74	58	47	34	26	21	15	7
Breidárlón	68	50	43	36	27	20	12	5
Vopnafjörður	73	60	52	44	31	23	17	10
Húsavík	55	40	33	26	20	15	10	3

Table A2-2 The predicted optimum yields for quarries for some recent breakwater projects in Iceland, Norway, India and South Africa.

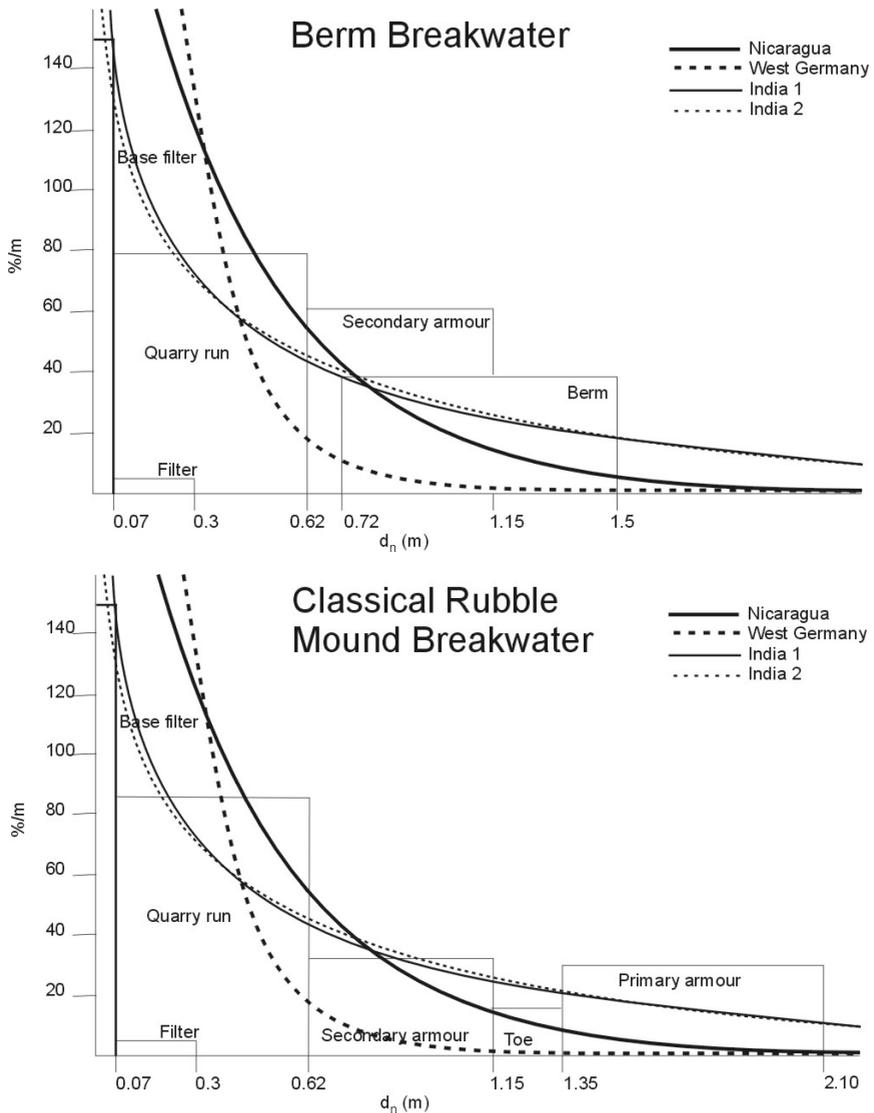


Figure A2-7 Fragmentation curve of different quarries and demand curves for classical rubble mound breakwater and for berm breakwater.

It is possible to plot both the fragmentation curve (or yield curve) as well as the demand curve in the same figure. Usually a yield curve follows the Rosin-Rammler equation:

$$y = 1 - \exp \left[- \left(\frac{d}{d_{63}} \right)^n \right]$$

where:

- y cumulative weight in % finer than d
- d particle size (block size)
- d_{63} characteristic particle size (63 % smaller than d_{63})
- n index of uniformity

The result of this equation is cumulative weight in %, but this can easily be converted into a percentage per running meter. The demand can also be expressed in this way. In Figure A2-7 the yield curve of a quarry in Nicaragua, a quarry in Germany and two yield curves from a quarry in India are given. Also the demand for both a classical rubble mound breakwater as well as a berm breakwater is given. It is clear that for the rubble mound a lot of non used material (waste) will be produced (VRIJLING AND NOOIJ VAN DER KOLFF [2005]). According to Figure A2-7 (quarry in Nicaragua), 10% of each blast will be in blocks of $d_n = 1.35$ m (or 6.5 ton) and larger. Consequently it is necessary to blast 10 X ton of material to obtain X ton of blocks of 6.5 ton and larger. The other 90% of the material must, however, also be classified, transported, stored and eventually be disposed of. And only a part of that 90% can be used in other parts of the breakwater. In view of the cost involved it is often necessary to search for productive use of this finer material.

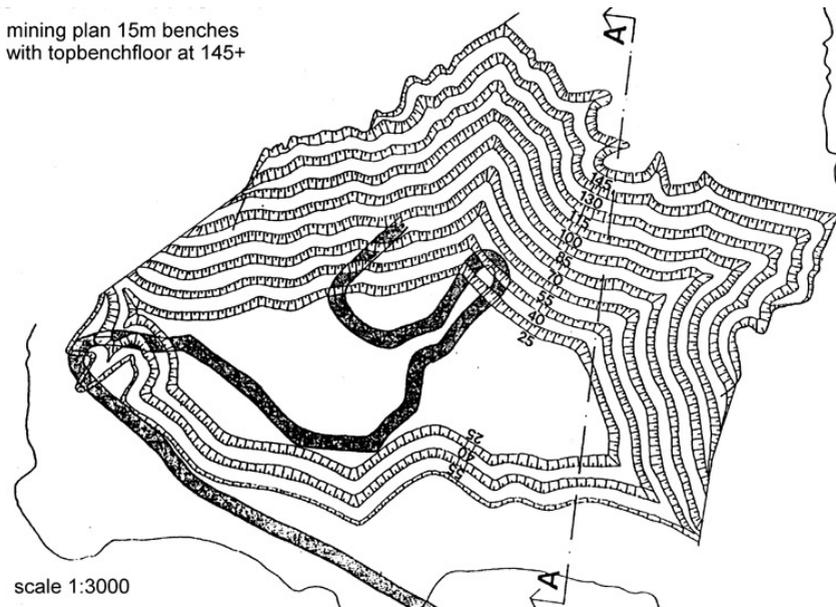


Figure A2-8 Mining plan.

Figure A2-7 shows at the same time the dramatic consequence of a deviation from the expected curve. For example the German quarry is absolutely unfitted to produce the required stones. Therefore a test blast of up to 100.000 ton of material is a necessary investment in the pretender stage (see for example the two test blasts in

India, both blasts resulted in quite identical curves, and may therefore considered as a good prediction of the later yield of the quarry).

A2.3 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 A2-8 and Figure A2-9).

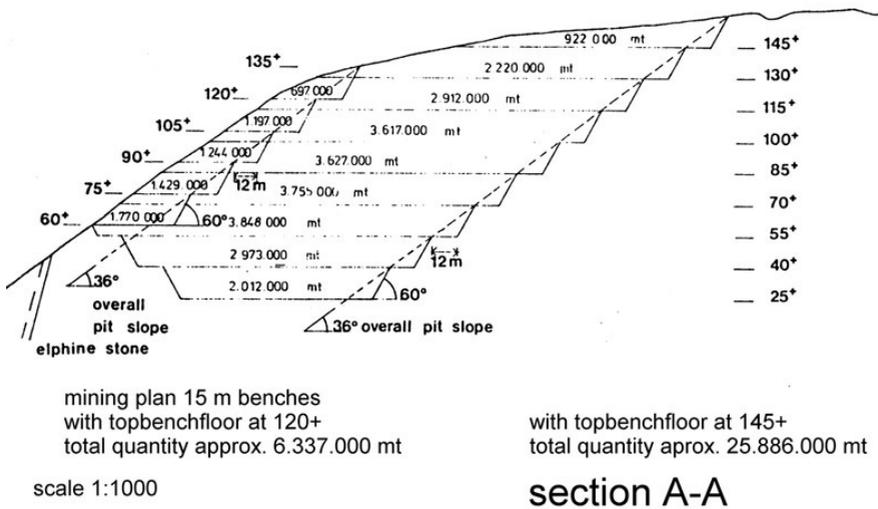


Figure A2-9 Mining plan.

APPENDIX 3

Concrete armour units

A3.1 Shape

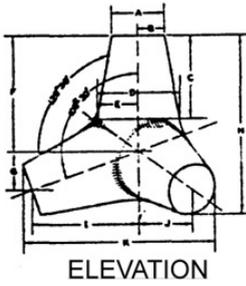
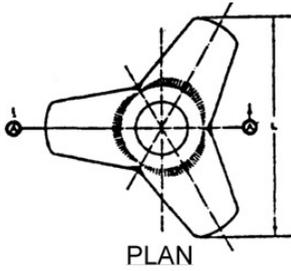
Compared with quarry stone, concrete armour units have the advantage that shape; the designer or contractor can choose size and density at liberty. As indicated in Chapter 7, a large number of different shapes has been developed by consultants, researchers and contractors. A rather complete overview is given by PIANC MarCom 36 [2005].

The 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 A3-1 to Figure A3-5, 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.

$$d_n = \sqrt[3]{\frac{M}{\rho_r}}$$

The latest developments turn away from the very slender units because of structural problems encountered, and discussed elsewhere in this Appendix. These latest units are Accropode-II, Core-loc, and Xbloc, developed respectively by Sogreah (France), the US Waterways Experiment Station and by Delta Marine Consultants (Netherlands).

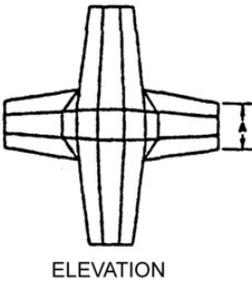
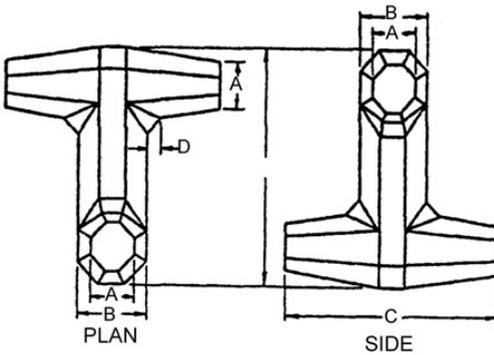


VOLUME OF INDIVIDUAL ARMOUR UNIT = $0.280 H^3$

where:

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

Figure A3-1 Tetrapod.



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

where

- A = 0.20 C
 B = 0.32 D
 C = overall dimension
 D = 0.057 C

Figure A3-2 Dolos.

Dimensions of these blocks are also given in the Figures. De Core-loc has originally been developed as a repair unit for damaged Dolos breakwaters, but it is also applied for new constructions.

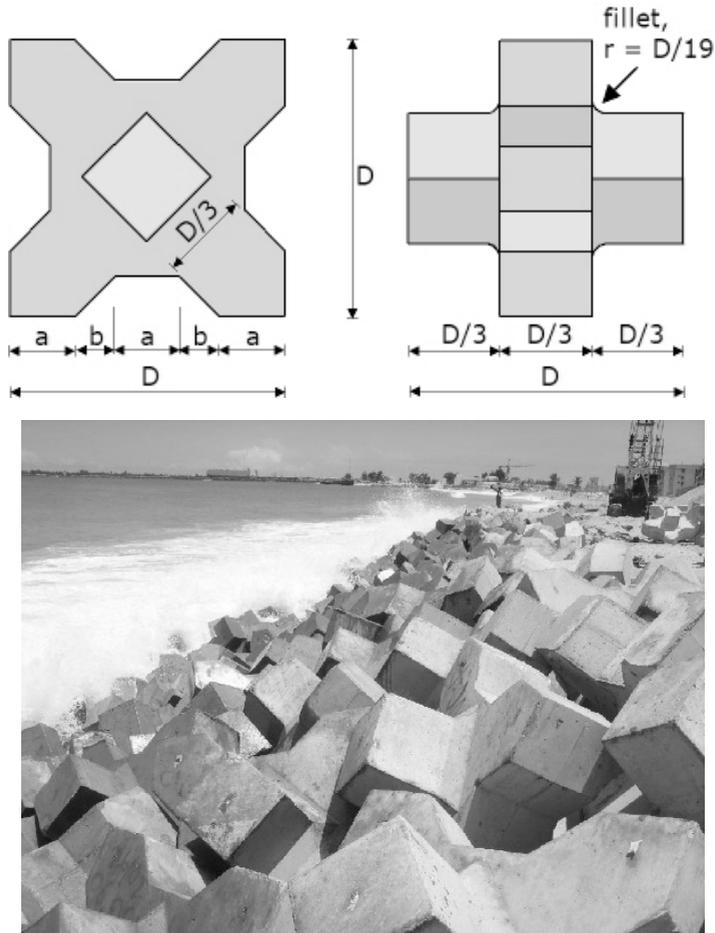


Figure A3-3 Xbloc.

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.

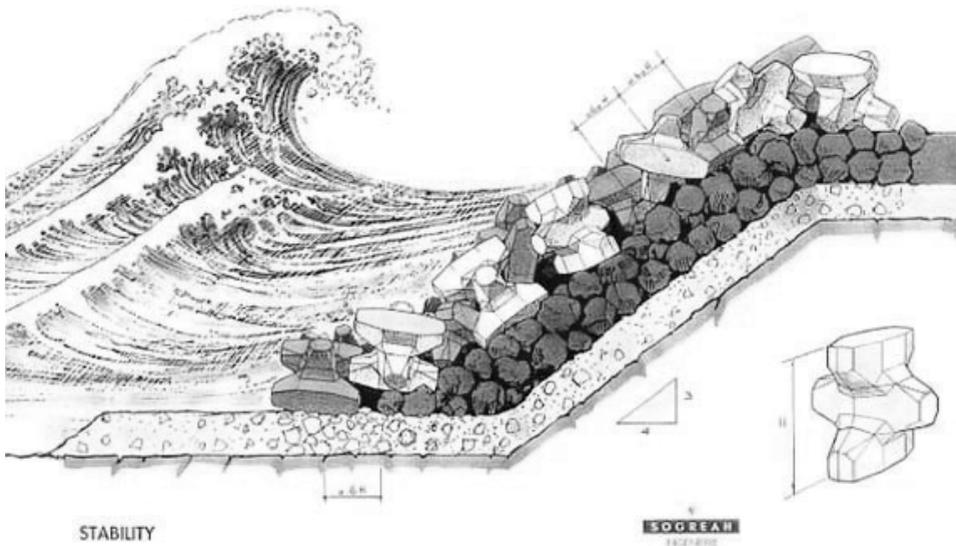


Figure A3-4 Accropode.

A3.2 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. ($V :: D^3$).

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.

A3.3 Density

The non-reinforced concrete as used in armour units will have a density ρ_r of 2200 to 2400 kg/m³ 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.

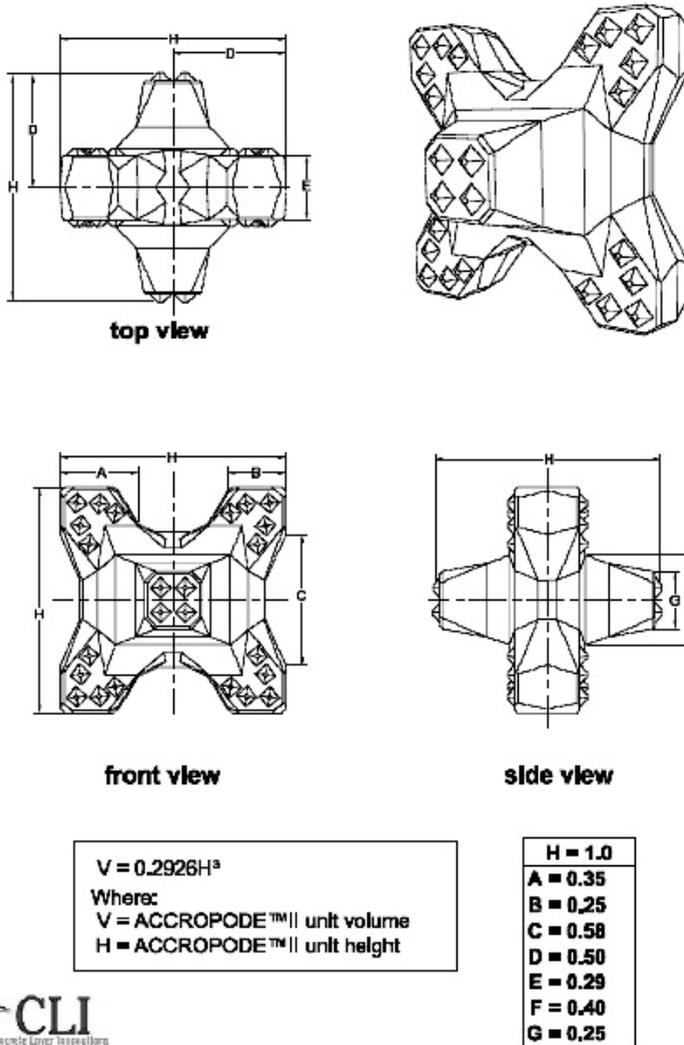


Figure A3-5 Accropode II.

A3-4 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 turn around 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 require 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).

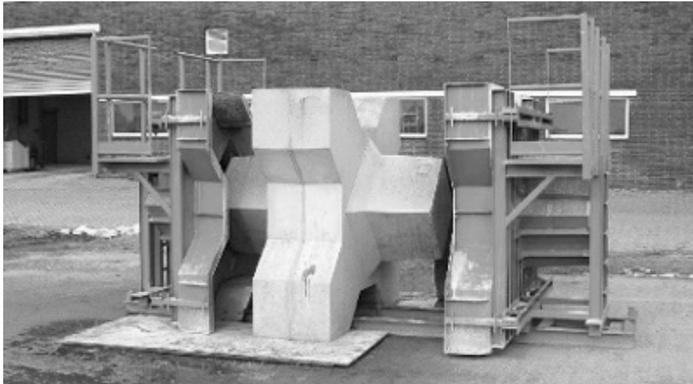


Figure A3-5 Xbloc formwork.

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
- 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.

A3.5 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.

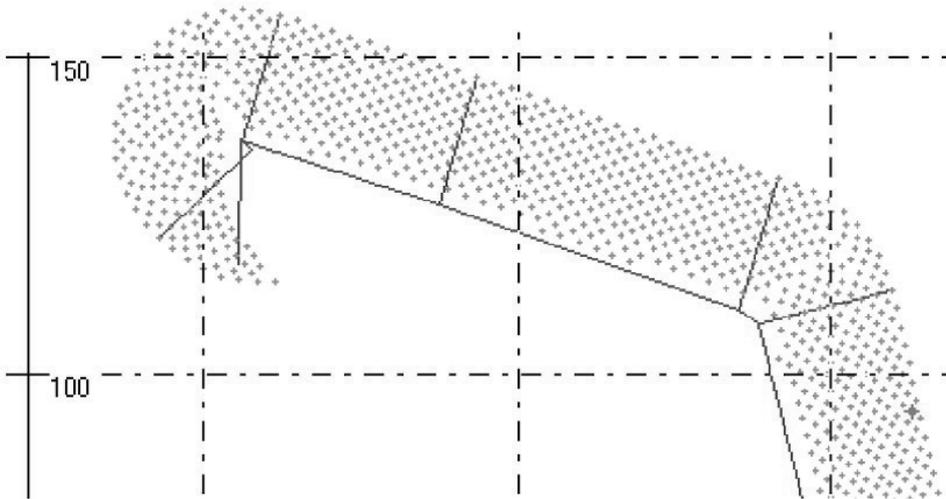


Figure A3-6 Xbloc placing grid for Port Oriel (TEN OEVER [2006]).

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 the 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].

APPENDIX 4

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

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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.

TABLE OF CONTENTS

1. Introduction
2. Historical Development of Upright Breakwaters in Japan
 - 2.1 Examples of Upright Breakwaters in Modern History of Japanese Ports
 - 2.2 Some Features of Japanese Upright Breakwaters
3. Review of Wave Pressure Formulae for Vertical Walls
 - 3.1 Hiroi's Formula
 - 3.2 Sainflou's Formula
 - 3.3 Minikin's Formula and Others
4. Design Formulae of Wave Pressures for Upright Breakwaters
 - 4.1 Proposal of Universal Wave Pressure Formulae
 - 4.2 Design Wave
 - 4.3 Wave Pressure, Buoyancy and Uplift Force
 - 4.4 Stability Analysis
 - 4.5 Example of Wave Pressure Calculation and Stability Analysis
5. Discussion of Several Design Factors
 - 5.1 Precautions against Impulsive Breaking Wave Pressure
 - 5.2 Structural Aspects of Reinforced Concrete Caisson
 - 5.3 Armor Units for Rubble Mound
6. Concluding Remarks
- References

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 owe 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.

2 Historical development of upright breakwaters in Japan

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 I. 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.

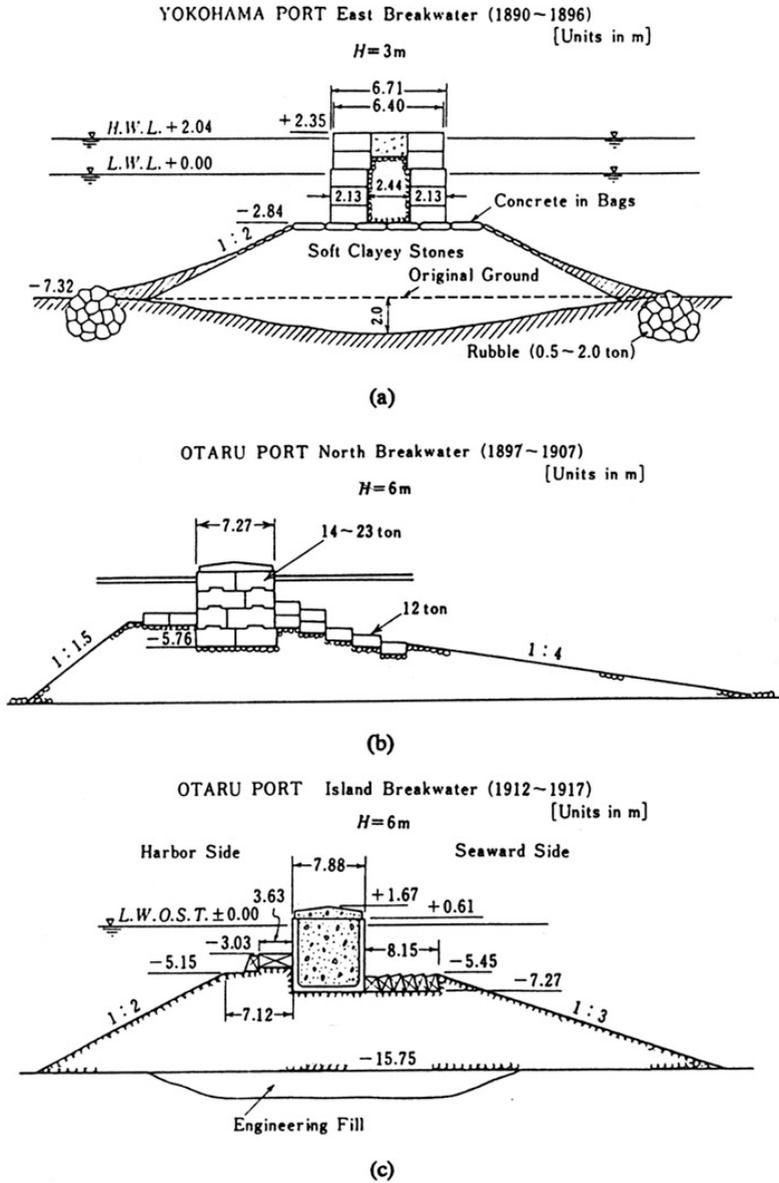


Fig. 1 (a-c) Historical development of upright breakwater in Japan after Goda [1985].

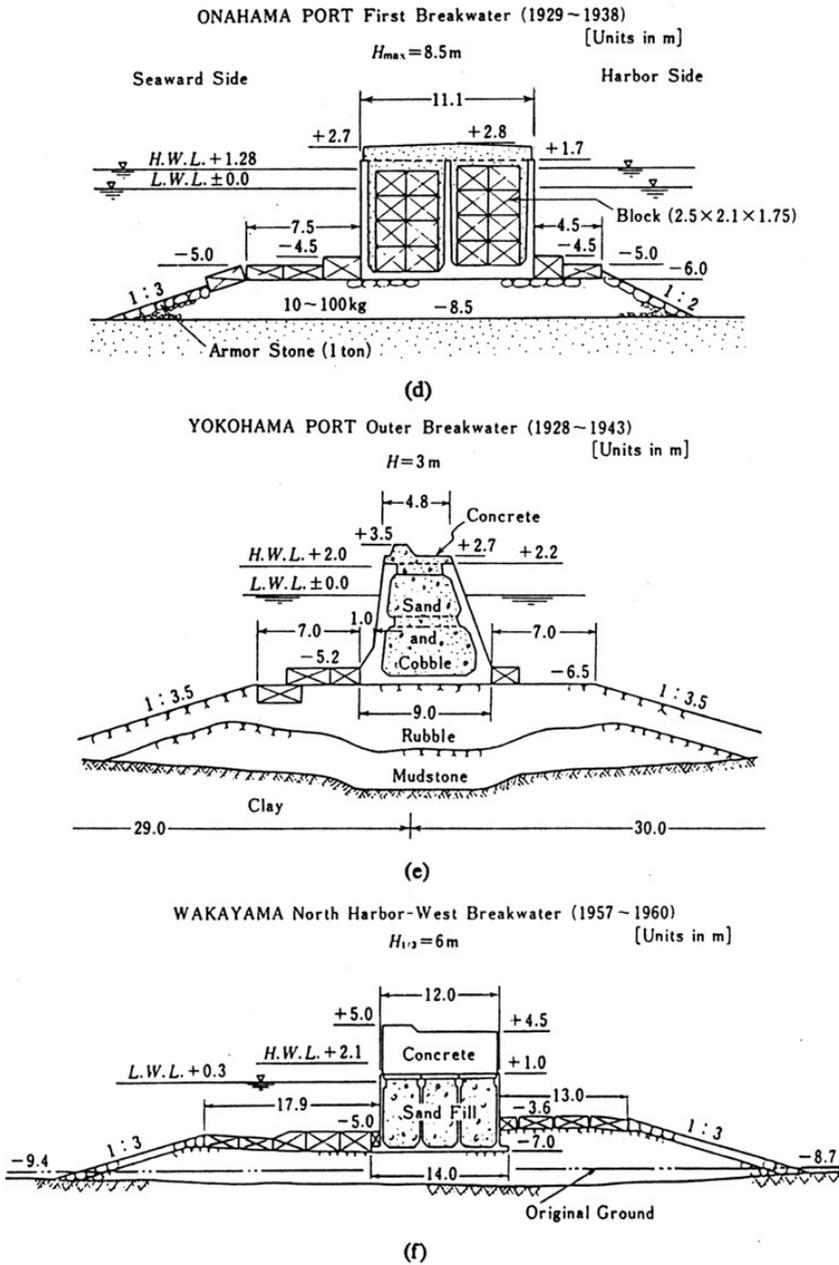


Fig. 1 (d-f) Historical development of upright breakwater in Japan (continued) after Goda [1985].

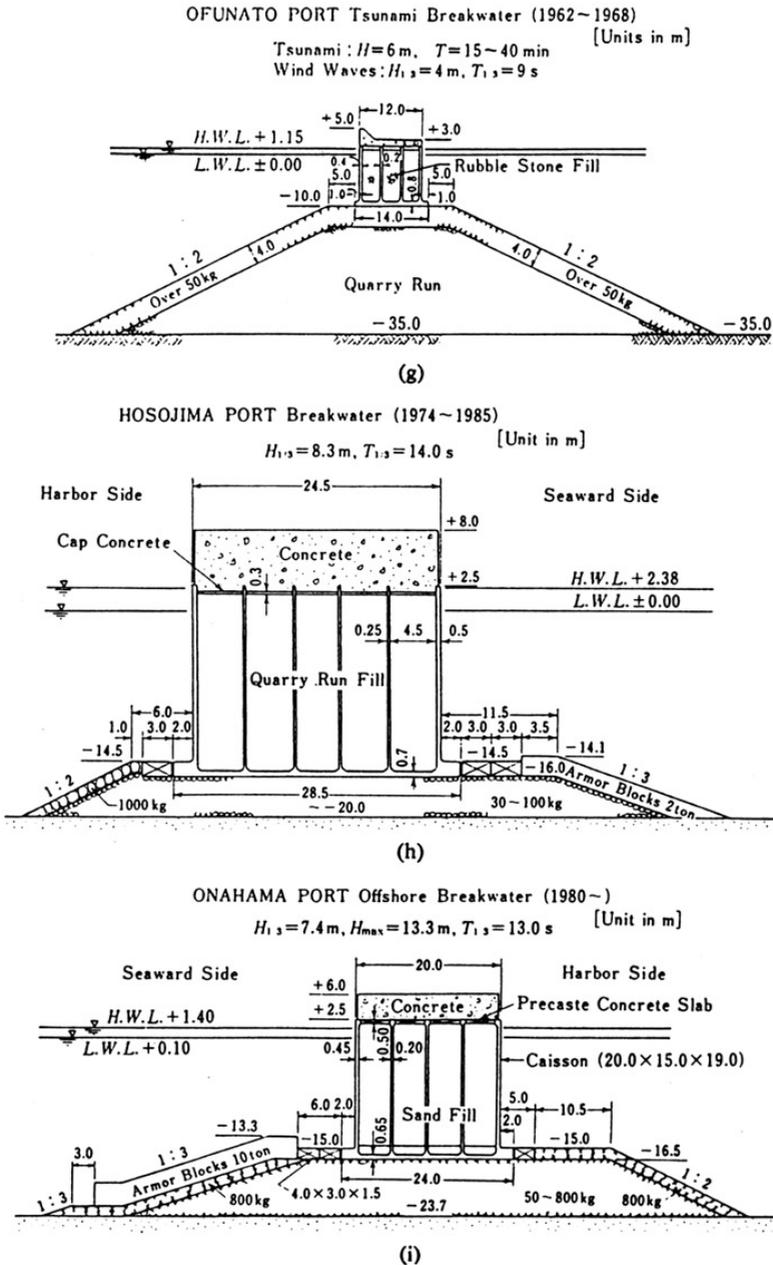


Fig. 1 (g-i) Historical development of upright breakwater in Japan (continued) after Goda [1985].

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 [1991b]. 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 1991a). The breakwater of Onahama Port shown in Fig. 1 (i) is of recent design using Goda's wave pressure formulas to be discussed later.

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 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.

3 Review of wave pressure formulae for vertical wall

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:

$$p = 1.5 w_0 H \quad (1)$$

where w_0 denotes the specific weight of sea water and H the incident wave height. This pressure distribution extends to the elevation of $1.25 H$ above the design water level or the crest of breakwater if the latter is lower, as shown in Fig. 2.

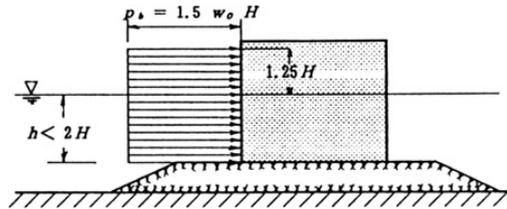


Fig. 2 Wave pressure distribution by Hiroi's formula.

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 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.

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.

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

$$\left. \begin{aligned} p_1 &= (p_2 + w_0h)(H + \delta_0)/(h + H + \delta_0) \\ p_2 &= w_0H/\cosh kh \\ \delta_0 &= (\pi H^2/L) \coth kh \end{aligned} \right\} \quad (2)$$

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 $H_{1\%}$.

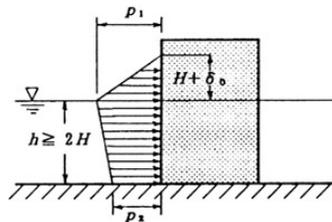


Fig. 3 Wave pressure distribution by Sainflou's formula.

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 formula was replaced with that by Hiroi's formula. The modified formula was sometimes called the partial breaking wave pressure formula in 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.

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_m and the hydrostatic pressure p_s as sketched in Fig. 4:

Dynamic pressure:

$$\left. \begin{aligned} p_n &= p_{\max}(1 - 2|z|/H)^2 : |z| \leq H/2 \\ p_{\max} &= 101w_0d(1 + d/h)H/L \end{aligned} \right\} \quad (3)$$

Hydrostatic pressure:

$$p_s = \begin{cases} 0.5w_0H(1 - 2z/H) & : 0 \leq z < H/2 \\ 0.5w_0H & : z < 0 \end{cases} \quad (4)$$

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.

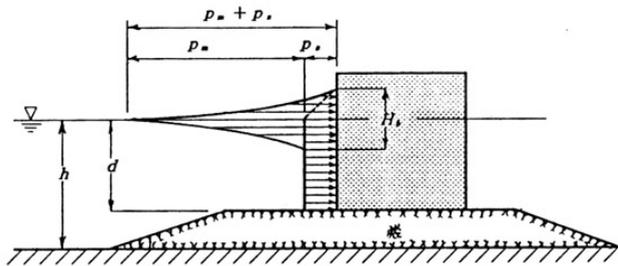


Fig. 4 Wave pressure distribution by Minikin's formula.

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.

4 Design formulae of wave pressures for upright breakwaters

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.

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.

(1) Wave height

$$H_{\max} = \begin{cases} 1.8H_{1/3} & : h/L_0 \geq 0.2 \\ \min \{(\beta_0 * H_0' + \beta_1 * h), \beta_{\max} * H_0', 1.8H_{1/3}\} & : h/L_0 < 0.2 \end{cases} \quad (5)$$

$$H_{1/3} = \begin{cases} K_s H_0' & : h/L_0 \geq 0.2 \\ \min \{(\beta_0 H_0' + \beta_1 h), \beta_{\max} * H_0', K_s H_0'\} & : h/L_0 < 0.2 \end{cases} \quad (6)$$

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]:

$$\left. \begin{aligned} \beta_0 &= 0.028 (H_0'/L_0)^{-0.38} \exp[20 \tan^{1.5} \theta] \\ \beta_1 &= 0.52 \exp[4.2 \tan \theta] \\ \beta_{\max} &= \max \{0.92, 0.32 (H_0'/L_0)^{-0.29} \exp[2.4 \tan \theta] \} \end{aligned} \right\} \quad (7)$$

$$\left. \begin{aligned} \beta_0^* &= 0.052 (H_0'/L_0)^{-0.38} \exp[20 \tan^{1.5} \theta] \\ \beta_1^* &= 0.63 \exp[3.8 \tan \theta] \\ \beta_{\max}^* &= \max \{1.65, 0.53 (H_0'/L_0)^{-0.29} \exp[2.4 \tan \theta] \} \end{aligned} \right\} \quad (8)$$

in which the symbol $\max\{a,b\}$ stands for the larger of a or b , and $\tan \theta$ 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].

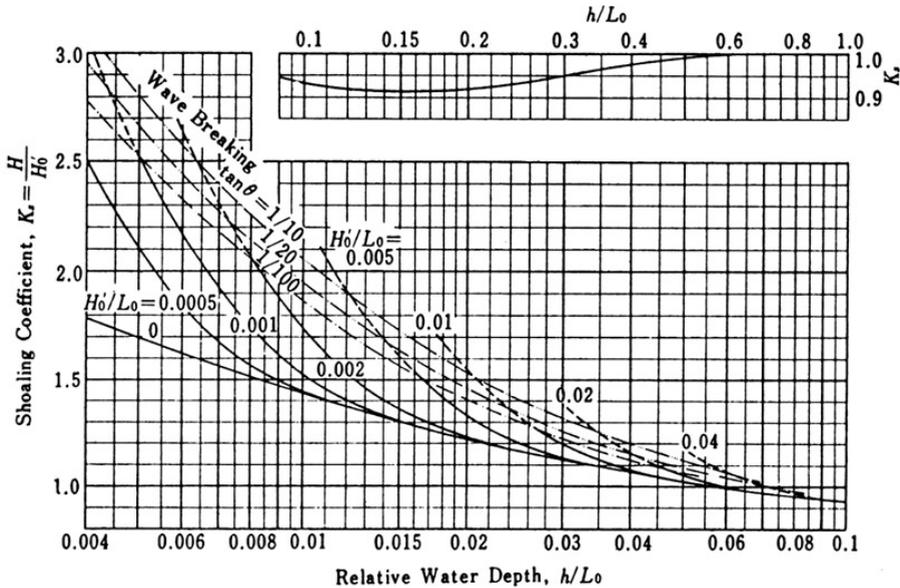


Fig. 5 Diagram of nonlinear wave shoaling coefficient K_s .

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.

For evaluation of H_{\max} by the second part of Eq. 7 or within the surf zone, the water depth at a distance $5 H_{1/3}$ 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 $5 H_{1/3}$ produces an appreciable increase in the magnitude of wave force and the resultant widening of upright section.

(2) Wave Period

The period of the highest wave is taken as the same with the significant wave period of design wave, i. e.,

$$T_{\max} = T_{1/3} \quad (9)$$

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.

(3) 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° 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.

4.3 Wave pressure, buoyancy and uplift pressure

(1) Elevation to which 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:

$$\eta^* = 0.75(1 + \cos\beta)H_{\max} \quad (10)$$

For waves of normal incidence, Eq. 10 gives the elevation of $\eta^* = 1.5 H_{\max}$.

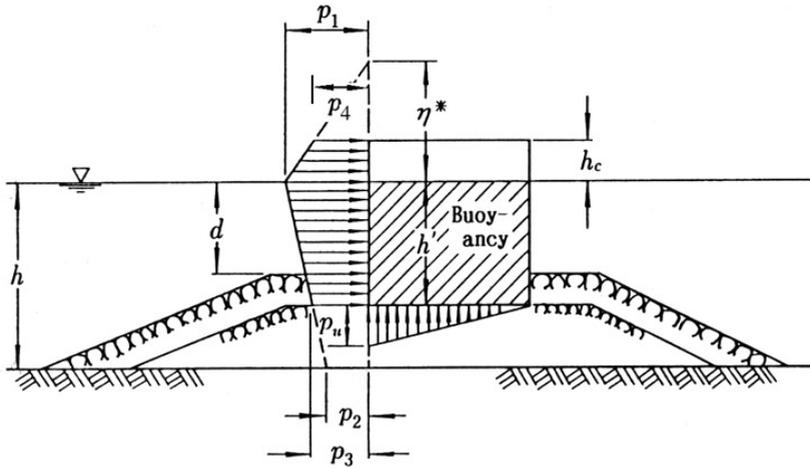


Fig. 6 Wave pressure distribution by Goda's formulas.

(2) 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:

$$\left. \begin{aligned} p_1 &= 0.5 (1 + \cos\beta) (\alpha_1 + \alpha_2 + \cos^2\beta) w_0 H_{\max} \\ p_2 &= p_1 / \cosh kh \\ p_3 &= \alpha_3 p_1 \end{aligned} \right\} \quad (11)$$

in which

$$\left. \begin{aligned} \alpha_1 &= 0.6 + 0.5 [2kh / \sinh 2kh]^2 \\ \alpha_2 &= \min \{ [(h_b - d) / 3h_b] (H_{\max} / d)^2, 2d / H_{\max} \} \\ \alpha_3 &= 1 - (h' / h) [1 - 1 / \cosh kh] \end{aligned} \right\} \quad (12)$$

where h_b denotes the water depth at the location at a distance $5H_{1/3}$ seaward of the breakwater.

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 $\cos^2\beta$.

(3) 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.

$$p_u = 0.5(1 + \cos\beta) \alpha_1 \alpha_3 w_0 H_{\max} \quad (13)$$

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.

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:

$$\text{Against sliding: } S.F. = \mu(W - U)/P \quad (14)$$

$$\text{Against overturning: } S.F. = (Wt - M_U)/M_P \quad (15)$$

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. 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 $c = 2 \text{ tf/m}^2$ and the angle of internal friction of $\phi = 35^\circ$ are recommended.

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: $H_0' = 7.0 \text{ m}$, $T_{1/3} = 11 \text{ s}$, $\beta = 10^\circ$
 Depth etc.: $h = 18 \text{ m}$, $d = 10 \text{ m}$, $h' = 11.5 \text{ m}$, $h_c = 4.5 \text{ m}$
 Bottom slope: $\tan \theta = 1/50$

The incident wave angle is the value after rotation by the amount up to 15° . The geometry of upright breakwater is illustrated in Fig. 7.

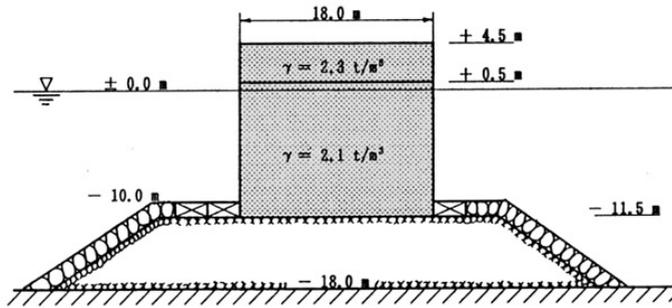


Fig.7 Sketch of upright breakwater for stability analysis.

i) Design wave height H_{max} and the maximum elevation of wave pressure η^*

The coefficients for wave height calculation are evaluated as

$$L_0 = 188.8 \text{ m}, H_0'/L_0 = 0.0371, h/L_0 = 0.0953, K_s = 0.94$$

$$\beta_0 = 0.1036, \beta_1 = 0.566, \beta_{max} = \min \{0.92, 0.84\} = 0.92$$

$$\beta_0^* = 0.1924, \beta_1^* = 0.680, \beta_{max}^* = \min \{1.65, 1.39\} = 1.65$$

Then, the wave heights and the maximum elevation are obtained as

$$H_{1/3} = \min \{10.91, 6.44, 6.58\} = 6.44 \text{ m}$$

$$h_b = 18.0 + 5 \times 6.44/50 = 18.64 \text{ m}$$

$$H_{max} = \min \{14.02, 11.55, 11.84\} = 11.55 \text{ m}$$

$$\eta^* = 0.75 \times (1 + \cos 10^\circ) \times 11.55 = 17.19 \text{ m}$$

ii) *Pressure components*

The wavelength at the depth 18 m is $L = 131.5$ m. The coefficients for wave pressure are evaluated as

$$\begin{aligned} kh &= 2\pi \times 18/131.5 = 0.860 \\ \alpha_1 &= 0.6 + 0.5 \times [2 \times 0.860 / \sinh(2 \times 0.860)]^2 = 0.802 \\ \alpha_2 &= \min \{ [(18.64 - 10.0) / (3 \times 18.64)] \times (11.55/10)^2, \\ &\quad 2 \times 10/11.55 \} \\ &= \min \{ 0.206, 1.732 \} = 0.206 \\ \alpha_3 &= 1 - 11.5/18.0 \times [1 - 1/\cosh(0.860)] = 0.820 \end{aligned}$$

Then, the intensities of wave pressure and uplift pressure are calculated as

$$\begin{aligned} p_1 &= 0.5 \times (1+0.9848) \times [0.802+0.206 \times (0.9848)^2] \times 1.03 \times 11.55 \\ &= 11.83 \text{ tf/m}^2 \\ p_2 &= 11.83/\cosh(0.860) = 8.49 \text{ tf/m}^2 \\ p_3 &= 0.820 \times 11.83 = 9.70 \text{ tf/m}^2 \\ p_4 &= 11.83 \times (1 - 4.5/17.19) = 8.73 \text{ tf/m}^2 \\ p_u &= 0.5 \times (1+0.9848) \times 0.802 \times 0.820 \times 1.03 \times 11.55 = 7.76 \text{ tf/m}^2 \end{aligned}$$

The symbol p_4 denotes the pressure intensity at the top of upright section.

iii) *Total pressure and uplift, and their moments*

$$\begin{aligned} P &= 0.5 \times (11.83+9.70) \times 11.5 + 0.5 \times (11.83+7.76) \times 4.5 = 167.9 \text{ tf/m} \\ M_P &= 1366.2 \text{ tf-m/m} \\ U &= 0.5 \times 18.0 \times 7.76 = 69.8 \text{ tf/m} \\ M_U &= (2/3) \times 69.8 \times 18 = 837.6 \text{ tf-m/m} \end{aligned}$$

iv) *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 : $\gamma_c = 2.3 \text{ tf/m}^3$

The portion below the elevation +0.5 m: $\gamma'_c = 2.1 \text{ tf/m}^3$

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

$$W_d = 2.1 \times (11.5 + 0.5) \times 18.0 + 2.3 \times (4.5 - 0.5) \times 18.0 = 619.2 \text{ tf/m}$$

$$W = 619.2 - 1.03 \times 11.5 \times 18.0 = 406.0 \text{ tf/m}$$

The safety factors against sliding and overturning of the upright section are calculated as in the following:

$$\text{Against sliding: } S.F. = 0.6 \times (406.0 - 69.8)/167.9 = 1.20$$

$$\text{Against overturning: } S.F. = (406.0 \times 9.0 - 837.6)/1366.2 = 2.06$$

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.

5 Discussion of several design factors

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.

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.

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:

$$W = \gamma_r H_{1/3}^3 / [N_s^3 (S_r - 1)^3] \quad (16)$$

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:

$$N_s = \max \left\{ 1.8, \left[1.3 \frac{1 - \kappa}{\kappa^{1/3}} \frac{h'}{H_{1/3}} + 1.8 \exp \left[-1.5 \frac{(1 - \kappa)^2}{\kappa^{1/3}} \frac{h'}{H_{1/3}} \right] \right] \right\} \quad (17)$$

in which the parameter κ is calculated by

$$\kappa = [2kh' / \sinh 2kh'] \sin (2\pi B_M / L') \quad (18)$$

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.

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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APPENDIX 5

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	$1.58 \cdot 10^{-1}$
5.2	$8.4 \cdot 10^{-2}$
5.5	$7.62 \cdot 10^{-2}$
5.8	$3.8 \cdot 10^{-2}$
6	$2.47 \cdot 10^{-2}$
6.5	$7.35 \cdot 10^{-3}$
7.15	$3.0 \cdot 10^{-3}$
7.25	$2.63 \cdot 10^{-3}$
7.8	$9.0 \cdot 10^{-4}$
7.98	$8.0 \cdot 10^{-4}$
8.7	$1.5 \cdot 10^{-4}$

Table A5-1 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 A5-2. 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 A5-2 Development of damage.

The initial construction cost I of the breakwater is estimated to be € 8620 for the core and € $1320 \cdot H_{nd}$ 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 A5-3.

Design wave height H_{nd}	Initial cost breakwater "C"	Initial cost Armour Layer "A"
(m)	(€) per running meter	(€) per running meter
4	13900	5280
5	15220	6600
5.5	15900	7280
6	16540	7920

Table A5-3 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	$\Delta p \cdot \Delta w$	Δp	Δw	$\Delta p \cdot \Delta w$	Δp	Δw	$\Delta p \cdot \Delta w$
(m)	(1/year)	(€)	(€/year)	(1/year)	(€)	(€/year)	(1/year)	(€)	(€/year)
4	1.02	420	430	$4.6 \cdot 10^{-2}$	860	40	$3.8 \cdot 10^{-2}$	13900	530
5	$1.5 \cdot 10^{-1}$	530	80	$4.7 \cdot 10^{-3}$	1060	5	$2.6 \cdot 10^{-3}$	15220	40
5.5	$7.4 \cdot 10^{-2}$	580	40	$2.2 \cdot 10^{-3}$	1160	-	$8 \cdot 10^{-4}$	15900	10
6	$2.4 \cdot 10^{-2}$	630	15	$7.5 \cdot 10^{-4}$	1260	-	$1.5 \cdot 10^{-4}$	16540	3

Table A5-4 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 $s = \Sigma(\Delta p \cdot \Delta w)$ as shown in Table A7-5.

H_{nd}	$s = \Sigma(\Delta p \cdot \Delta w)$		
	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 A5-5 Average annual maintenance cost for various maintenance strategies.

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 A5-6.

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 A5-6 Capitalised maintenance cost for various maintenance strategies.

It is now a simple exercise to add the initial cost I and the capitalised maintenance cost S as in Table A5-7.

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 5-7 Total cost for various maintenance strategies.

The optimum values are printed in bold.

APPENDIX 6

Closing sequence in case of multiple channels

A6.1 Introduction

A detailed plan showing the construction phases during the closure of an estuary entrance comprising several channels and tidal flats has not yet been presented. In this appendix some examples of a hypothetical closure will demonstrate various options. A number of alternatives will be outlined and their relations with some historic cases will be discussed. Data on flow velocities and discharges has been taken from mathematical calculations. Only data that is relevant to the decisions in the design is presented.

The example assumes a tidal estuary that has to be closed along a fixed alignment. The longitudinal profile of the total closure (see Figure A6-1) consists of (from left to right):

- a foreshore, 250 m wide, 0.5 m lower than mean sea level.
- a secondary gully of 200 m width, with an average depth of 4 m below mean sea level (MSL),
- a tidal flat 300 m wide, at a level of about MSL,
- the main gully 250 m wide, with an average depth of 6.5 m below MSL and the greatest depth along the bank.

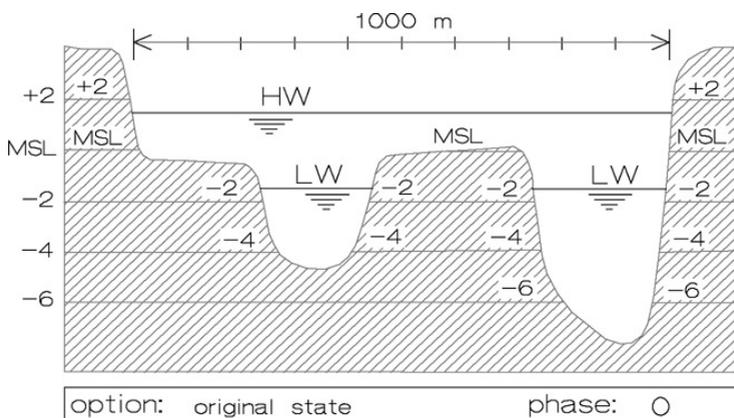


Figure A6-1 Original state, phase 0.

The longitudinal profile of the closure gap is thus 4000 m² at high water and 1800 m² at low water. The tide is a semi-diurnal sine wave with a range of 3 m. In all

calculations the tidal range is taken as constant; neap-spring variation is ignored. The storage area of the basin is 20 km² at high water and 5 km² at low water.

Three main options will be studied:

1. dam sections first across the shallows and then closing the gullies (Section A6.2).
2. dealing with the gullies first and closing the shallows later (Section A6.3).
3. simultaneous closures (Section A6.4).

Each option may have several alternatives.

A6.2 Blocking the shallows first

Dam sections across the shallows will create two gaps. Many possibilities are created, but some are unattractive. For example the secondary gap is very shallow for caissons, while for a vertical closure, the total length of the two sills (only 450 meters) is rather short (this will be clarified when describing options b and c). To obtain the required data for a well-considered decision it is necessary to use a mathematical model. Horizontal closure by tipping quarry stone in both gaps is a very good possibility, but for the purpose of this example, a combination of placing caissons and tipping quarry stone will be considered.

Some construction phases are presented in Figure A6-2, Figure A6-3 and Figure A6-4. The programme reads:

Two gaps are created by damming the shallows. Bottom protection is provided for in the remaining gaps. Then, both closure gaps are slightly diminished in sectional profile by sills. For a caisson closure abutments are also made. These are concrete structures or sheet-pile walls that shape the vertical sides of the closure gap. Next, caissons are positioned in the main gap and finally the secondary gap is closed by tipping quarry stone.

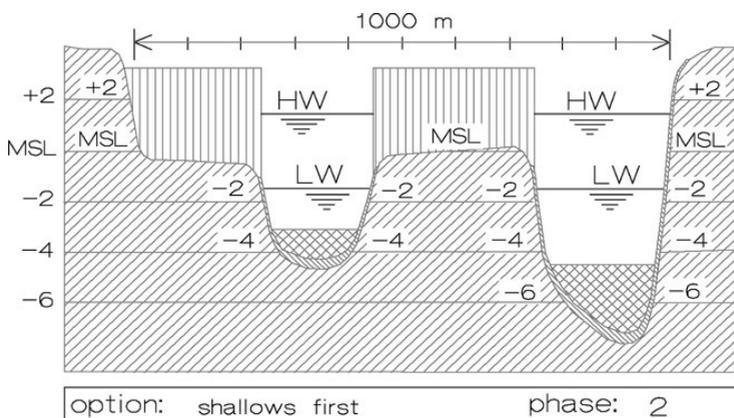


Figure A6-2 Shallows first, phase 2.

The programme is summarized in Table A6-1.

phase	action	foreshore	sec. gully	tidal flat	main gully
0	original state	250 m; -0.5	200m; -4	300m; msl	250m; -6.5
1	bottom protection + shallows	dammed	200m; -3.5	dammed	250m; -6
2	partial sills in both gaps	dammed	200m; -3	dammed	250m; -4.5
3	final sill, abutments	dammed	200m; -2.5	dammed	190m; -4.5
4	first caisson in place	dammed	200m; -2.5	dammed	128m; -4.5
5	sec. caisson in place	dammed	200m; -2.5	dammed	66m; -4.5
6	third caisson in place	dammed	200m; -2.5	dammed	closed
7	narrowing on sec. sill	dammed	100m; -2.5	dammed	closed
8	further narrowing	dammed	50m; -2.5	dammed	closed
9	last gap	dammed	10m; -2.5	dammed	closed

Table A6-1.

As the two gaps are blocked virtually in succession, there may be considerable imbalance in the tidal system between the gaps. To prevent this, the tipping into the secondary gap has to occur simultaneously with the placing of the caissons. The effect of this procedure is that the flow conditions during the sinking of the last caisson are too high. It is always necessary to check whether the flow conditions during the last phase are acceptable.

Checking on these conditions gives the data in Table A6-2. At the moment when two caissons are being placed (phase 5) the maximum discharge via the secondary gap has doubled and reaches about the same magnitude as the main gap originally had, while on the contrary, discharge via the main gap has halved. The secondary channel will have to accommodate the doubled quantities, for which its profile will be barely adequate. The scouring of a gully across the shallow from the main to the secondary channel is the likely consequence of the imbalance.

(The values in Table A6-2 have been calculated for the same tide variation at sea; spring/neap variations are not included. Maximum flow does not necessarily coincide with maximum discharge, neither do the maxima of the two gaps always coincide).

phase	situation	secondary gap				main gap			
		during ebb		during flood		during ebb		during flood	
		U_{\max}	Q_{\max}	U_{\max}	Q_{\max}	U_{\max}	Q_{\max}	U_{\max}	Q_{\max}
0	original	1.09	915	1.07	940	1.09	1810	1.07	1825
1	bp+dams	1.33	1010	1.27	1045	1.33	2070	1.27	2085
2	sills	1.67	1065	1.57	1135	1.67	1935	1.57	1995
3	abutment	2.12	1090	1.94	1215	2.12	1790	1.94	1865
4	1 placed	2.71	1305	2.39	1505	2.57	1385	2.26	1470
5	2 placed	3.57	1550	3.00	1875	3.19	820	2.69	895

Table A6-2.

(phase 2 is pictured in Figure A6-2)

The positioning of the last caisson takes place in the situation of phase 5 during HW-slack, because at LW there is insufficient water depth. At the end of the flood period the flow diminishes as follows:

- 1) 30 min. before slack: $u = 1.50$ m/s,
- 2) 20 min. before slack: $u = 1.20$ m/s,
- 3) 10 min. before slack; $u = 0.70$ m/s.

For a safe sinking operation, these values are far too high. Consequently, the programme has to be adapted. Instead of using ordinary caissons, caissons equipped with sluice gates can be used. The programme then reads as indicated in Table A6-3.

phase	action	foreshore	sec. gully	tidal flat	main gully	sluice gate
4	first placed, opened	dammed	200m; -2.5	dammed	128m; -4.5	56m; -3.5
5	sec. placed, opened	dammed	200m; -2.5	dammed	66m; -4.5	112m; -3.5
6	third caisson placed	dammed	200m; -2.5	dammed	0m	112m; -3.5
7	narrowing on sill	dammed	100m; -2.5	dammed	0m	112m; -3.5
8	further narrowing	dammed	50m; -2.5	dammed	0m	112m; -3.5
9	last gap in sec.	dammed	10m; -2.5	dammed	0m	112m; -3.5
10	close sluice gates	dammed	dammed	dammed	0m	closed

Table A6-3.

(phase 4 is pictured in Figure A6-3)

This time the positioning of the last caisson takes place in phase 5, with the sluice gates of the two other caissons opened (assumed to have 56 m effective width each and a floor thickness of 1 m). Then, at the end of the flood period the flow conditions appear to be acceptable:

- 30 min. before slack: $u = 0.75$ m/s,
- 20 min. before slack: $u = 0.50$ m/s,
- 10 min. before slack; $u = 0.20$ m/s.

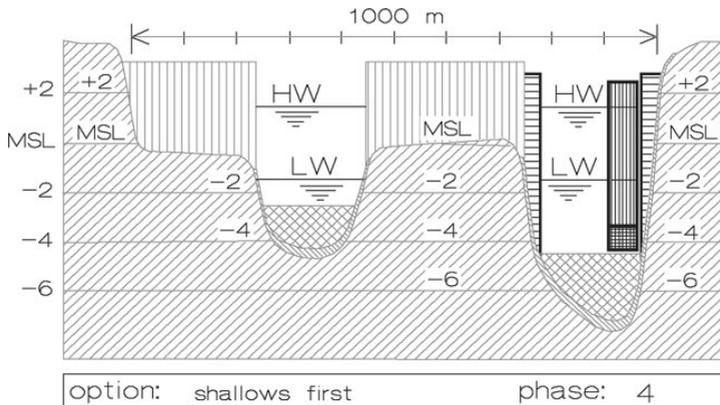


Figure A6-3 Shallows first, phase 4.

At first, the balance between the gaps will be better than in the former scheme because the gates provide a flow profile while tipping starts in the other gap. Although the secondary gap is closed before the gates close, the main channel does not exceed its original discharge. The balance can still be improved by closing a number of gates simultaneously with the tipping. However, that worsens the conditions for the tipping. The flow conditions are given in Table A6-4.

phase	situation	secondary gap				main gap **			
		during ebb		during flood		during ebb		during flood	
		u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}
5	1+2 open	2.60	1260	2.30	1445	2.32	1460	2.06	1580
6	3 placed	3.35	1480	2.85	1775	2.82	965	2.40	1095
7	100m gap	3.87*	830	3.40	1040	3.67	1155	3.03	1360
8	50 m gap	3.78*	410	3.57	535	3.95*	1220	3.36	1485
9	10 m gap	3.62*	80	3.58	105	4.05*	1245	3.58	1560

* means critical flow ** via the sluice gates

Table A6-4 Flow and discharge conditions.

(Phase 7 is pictured in Figure A6-4)

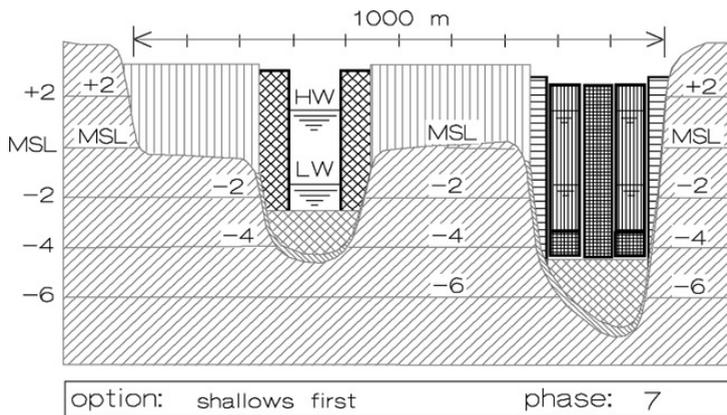


Figure A6-4 Shallows first, phase 7.

Critical flow also occurs during the ebb in the sluices. Per tidal cycle this lasts for nearly 2 hours in phase 8 and for 2.5 hours in phase 9. This can probably be prevented by providing the third caisson with sluice gates. This was not required for the sinking operation. If sluice gates are provided for the third caisson the conditions for the tipping of the stone are better, but the imbalance between the gaps increases. Whether doing this is worth the extra expense, depends on the savings on stone tipping and bottom protection. The flow and discharge conditions for three sluice caissons are given in Table A6-5.

u in m/s Q in m ³ /s		secondary gap				main gap **			
		during ebb		during flood		during ebb		during flood	
phase	situation	u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}
7	100m gap	3.35*	720	2.80	875	3.14	1570	2.63	1805
8	50 m gap	3.55*	385	3.09	480	3.51	1695	2.91	1980
9	10 m gap	3.49*	80	3.15	100	3.81*	1780	3.15	2120

Table A6-5 Flow and discharge conditions.

Critical flow in the sluices now occurs during the final operation and lasts for half an hour only. Maximum flow velocities in the secondary gap reduce by about 10%.

A historic case of the use of the above system of closing a tidal basin was the construction in of the "Brouwersdam" between 1965 and 1972. The damming the "Brouwershavensche Gat" created Lake Grevelingen. The dimensions of the channels and the basin were large. The total dam alignment was 6 km long. In this case the minor gully was closed by sinking 12 sluice-caissons, each 68 m long, on a sill levelled at 10 m below MSL. The main gully was closed by gradual closure with 50 kN concrete cubes. The profile of this gap was about 13000 m². Unlike the hypothetical example, the gradual closure was a vertical closure, dropping the cubes by means of a pre-installed cableway. As in the hypothetical example, the flow condition forms a limiting factor for the progress of the gradual closure.

A6.3 Blocking the main channel first

In this section the same estuary as above is closed by first reducing the profiles of the gullies. Next the main gully is completely blocked. The secondary gully will then be further reduced and finally the total remaining profile will be blocked. This option is worth consideration if it leads to a cheaper closure operation. When the channels are blocked first the obvious disadvantage is that the bottom of the shallows also has to be protected against scour. Cost savings on the other items need to compensate for this expensive extra. Such savings may result from a possible reduction in the dimension of the protection in the gully and from cheaper caisson design. A major saving would result if caissons without sluice gates were used, while another saving could be obtained by using two caissons only.

A determining factor for the decision to omit sluice gates is the positioning of the last caisson. The flow conditions will be best when the cross sectional area of the flow profile is as large as possible at that moment. This is the case when there is no sill in the secondary gap. Assuming the same dimensions of the caissons, the determining total flow profile is the original profile, reduced by bottom protection over the full length, by the abutments on both sides of the main gully, by the foundation sill and by the caisson(s) already placed. The HW-slack period then is characterised by:

17 30 min. before slack: $u = 0.80$ m/s,

18 20 min. before slack: $u = 0.60$ m/s,

19 10 min. before slack; $u = 0.25$ m/s.

Positioning will not be a problem. However there is a substantial imbalance between the two gullies. The maximum flood via the secondary gully is $1975 \text{ m}^3/\text{s}$, which is more than twice the original. A sill in this gully, up to the level -3m , brings the discharge down to $1860 \text{ m}^3/\text{s}$. The effect is small and flow velocities in the main gully increase up to:

- 30 min. before slack: $u = 0.95$ m/s,
- 20 min. before slack: $u = 0.65$ m/s,
- 10 min. before slack; $u = 0.35$ m/s.

Sinking is possible, but further raising of the sill is not acceptable. The construction of the sill up to -3m in the secondary gully can best be done concurrently with the foundation sill in the main gully. After that, caissons can be placed at short intervals to limit the duration of the imbalance.

Saving on the number of caissons depends on the flow conditions for creating the smaller gap for the two caissons, as the narrowing of the gully has to be done by pumping sand. The flow profile available is the original profile reduced by bottom protection and by the island for the abutment on the shallows. The longitudinal profile then consists of the following:

- 250 m foreshore, 200 m secondary gully and 250 m shallows, all provided with bottom protection
- an island section on the shallows of 50 m length
- an adjoining island in the main gully along 75 m, leaving a gap length for two abutments of 25 m each
- two caissons of 60 m each and 5 m extra (see Figure A6-5)

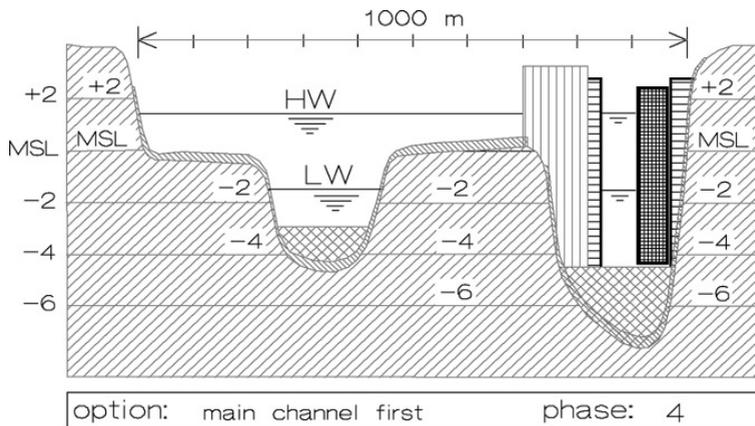


Figure A6-5 Main channel first, phase 4.

In the secondary gap, bottom protection will be present and some of the sill construction may exist. The calculations show that the maximum flow velocities in the gap are 1.70 m/s during ebb and 1.60 m/s during flood, which is no problem for the construction of the island.

The conclusion is that blocking the gullies first can be achieved by closing the main gap with two simple caissons, while a restricted sill is present in the secondary gully. The remaining flow profile consists of two 250 m-long shallow sections with a partly blocked gap in between. The construction phasing up to this moment thus reads as per Table A6-6.

phase	action	foreshore	sec. gully	tidal flat	island	main gully
0	original state	250m; -0.5	200m; -4	300m; MSL	none	250m; -6.5
1	bottom prot. + island	250m; MSL	200m; -3.5	250m; +0.5	125m	175m; -6
2	sills in both gaps	250m; MSL	200m; -3	250m; +0.5	125m	175m; -4.5
3	sill, abutments	250m; MSL	200m; -3	250m; +0.5	150m	125m; -4.5
4	first caisson placed	250m; MSL	200m; -3	250m; +0.5	150m	65m; -5
5	sec. caisson placed	250m; MSL	200m; -3	250m; +0.5	-	closed

Table A6-6.

Flow velocities and discharges are given in Table A6-7.

phase	situation	secondary gap **				main gap			
		during ebb		during flood		during ebb		during flood	
		u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}
0	original	1.09	915	1.07	940	1.09	1810	1.07	1825
1	bott. prot. + island	1.61	1155	1.52	1215	1.61	1695	1.52	1720
2	sills	2.01	1175	1.85	1295	2.01	1525	1.85	1600
3	abutment	2.42	1355	2.19	1525	2.29	1205	2.07	1285
4	after 1st	3.06	1585	2.68	1860	2.73	705	2.40	770
5	after 2nd	3.98*	1860	3.37	2310	closed	0	closed	0

** the central 200 m section only (the shallows falling dry during low tide).

Table A6-7.

The next step could be a horizontal closure by tipping quarry stone from either one side or from both sides. This creates high flow velocities in the secondary gap. The situation is comparable with the former option, except for the sluice gates. In this case the flow velocities will increase to about 4.5 m/s. Therefore, it is more appropriate to try to reduce the profile of the secondary gap, maintaining the flow across the shallows. Dumping quarry stone from dump-vessels will be impossible because of draught restriction. However, vertical closure by means of a temporary bridge (to be installed in the previous period) or a cableway is possible. The length is considerable (700 m), but 500 m of the bridge crosses shallow water so the cost of the foundation is limited.

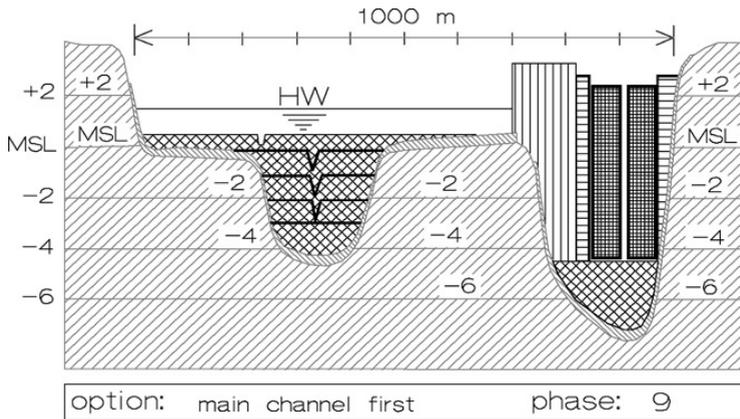


Figure A6-6 Main channel first, phase 9.

Another method, which involves a difficult operational procedure, takes into account the fact that during LW the dam section across the shallows falls dry for several hours. The first step is to bring the sill level up in two layers to above LW, (from -3 to -2 and then to -1). The water level in the basin will not follow the sea level and the relation between the levels of the sea, the basin and the sill determine the operational possibilities. The equipment to be used is a shallow-draught crane vessel and dump trucks with hydraulic cranes, approaching via the drying dam sections. The operational period per tidal cycle (the work-window) is short and the production is low, but the equipment is available on the market and investment in a bridge or cableway can be avoided. Gradually, layer by layer, the sill will be raised. For every layer, the determining moment exists when the last 10 m has to be placed. That missing part of the layer gives a dip in the sill level, which is subjected to higher (critical) flow.

The crane vessel can operate when anchored near the gap during the periods that flow velocities are lower than 2 m/s. For the layer from -3 m to -2 m (phase 5 to 6) this averages 2 hours during HW and 1 hour during LW. For the next layer (phase 6 to 7), up to -1 m, this is 1.5 hours at HW and three quarters of an hour during LW. After that, dump trucks may start to assist during the LW-period, because the water level at the seaward side will fall lower than the sill level. At the start of the next layer up to MSL (phase 7 to 8), the basin level falls below that level during about 1 hour (b-b in the figure). When finishing the layer, the water levels are less than 0.5 metres above the sill level for two hours. Though it is risky, trucks can operate in water as long as it is less than 0.5 metres deep.

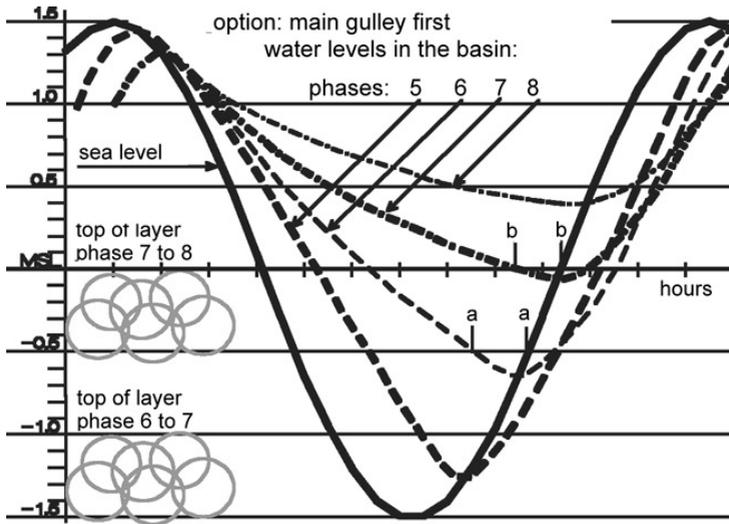


Figure A6-7 Main channel first, water levels in the basin.

The construction phasing thus continues for the 700 m section vertically (see Table A6-8):

phase	action	foreshore	sec. gully		tidal flat	
6	first layer	250m; MSL	97m; -2	6m; -2	97m; -2	250m; +0.5
7	first layer	250m; MSL	97m; -1	6m; -2	97m; -1	250m; +0.5
8	level foreshore	250m; MSL	97m; MSL	6m; -1	97m; MSL	250m; +0.5
9	level tidal flat	222m; +0.5	6m; MSL		222m; +0.5	250m; +0.5
10	level + 1	347m +1		6m; +0.5		347m; +1
11	final layer	dammed		6m; +1		dammed

Table A6-8.

Table A6-9 gives the flow velocities for the various levels of the sill.

u_{max} in m/s		deepest part		deepest but one		deepest but two		deepest but three	
phase	situation	ebb	flood	ebb	flood	ebb	flood	ebb	flood
5	after 2nd	3.98*	3.37	2.34*	2.85*	1.80*	2.50*		
6	up to -2	4.22*	3.43	3.81*	3.84	2.28*	3.03*	1.94*	2.32*
7	up to -1	3.82*	3.38	3.28*	2.68*	2.38*	3.15*	2.02*	2.32*
8	up to MSL	3.27*	2.92	2.50*	2.91*	2.06*	2.32*		not applicable
9	up to 0.5	2.32*	2.67	1.98*	2.32*		not applicable		
10	up to + 1	1.86*	2.18*	1.05*	1.55*				
11	up to HW	0.88*	1.55*	high water free					

* means limited by critical flow condition.

Table A6-9.

In the table the deepest part represents the dip in the sill mentioned above. Although in all sections of the sill critical flow limits the flow velocity, in the dip this is only

true for the ebb. Besides, ebb is determining for sill levels up to about MSL, above that flood flows are higher.

Considering the above results, it appears that the maximum flow velocity in the secondary gap during the raising of the sill is not very much less than would have been the case if a horizontal closure had been designed (4.22 m/s instead of about 4.50 m/s). This is due to the fact that the limiting critical flow condition for the -2m sill level under these circumstances is about the same as the normal flow condition for a narrow gap with a 3 m tidal range. The determining condition occurs for the dip in the low sill in the 200 m gap. In that stage of the process, the 500 m shallow sections are too elevated to be useful. This example proves by its exception that the general rule that vertical closure leads to smaller flow velocities than horizontal closure is not always applicable. From a financial point of view, neither the difficult operational procedure and small production capacity, nor the investment for a bridge or cable-way, can compete with the dump trucks operating from two sides for horizontal closure.

A rather critical point in the phasing of the closure is the situation near the island after the caissons have been placed. The main gully is blocked and the discharge of the secondary gully is more than doubled. An easy way for the water to pass through the gap is by following the main gully, circling across the shallows around the island and to return into the main gully. Scouring a short-cut like that is a typical example of the consequence of the imbalance. It would cause a disaster and has to be prevented.

A historic case of a closure in which first gullies were blocked by caissons and then the shallows were closed, is the closure of the Schelphoek, one of the major dike breaches in the south west of the Netherlands (1953). The situations of the breaches and the closure alignment are given in Section 2.5 showing the development of erosion gullies. The picture of the situation after 20 weeks shows the two gaps that had been shaped, typically suited for caisson placing, while the long overland stretches were protected by mattresses. After the caissons had been placed the overland sections were constructed by horizontal closure. A large number of shallow concrete units was placed every tide in such a short sequence that the overland flow could not scour a short-cut.

The vertical closure procedure, using dump trucks driving on the sill and using hydraulic cranes to level the cobbles, was executed during the closure of the Markiezaatskade (1983), one of the secondary dams required in the Delta Works. The dam closed a shallow tidal basin of about 20 km² with a tidal range of about 4 meters. The final gap was 800 m long and had a basic sill level of 2 m below MSL. In that case, the vertical closure was an advantage as the full 800 m had the same low sill level. The quarry stone dam was constructed in 1 or 0.5 m thick layers.

phase	action	foreshore	sec. gully	tidal flat	main gully
0	original state	250m; -0.5	200m; -4	300m; MSL	250m; -6.5
1	bottom prot. + sill (-3.5)	250m; MSL	200m; -3.5	300m; +0.5	250m; -3.5
2	sills dumped (-3)	250m; MSL	200m; -3	300m; +0.5	250m; -3
3	sills dumped (-2.5)	250m; MSL	200m; -2.5	300m; +0.5	250m; -2.5
4	sill by trucks (-1)	250m; MSL	200m; -1	300m; +0.5	245m; -1
5	up to MSL	445m; MSL	5m; -1	300m; +0.5	250m; MSL
6	up to +0.5	445m; +0.5	5m; MSL	300m; +0.5	250m; +0.5
7	up to +1	445m; +1	5m; +0.5	300m; +1	250m; +1
8	up to HW	445m; +1.5	5m; +1	300m; +1.5	250m; +1.5

Table A6-10.

The influence on the flow conditions appears from the lists below (to be compared with the table in the former option for the 700 m long weir):

phase	action	secondary gap				main gap			
		during ebb		during flood		during ebb		during flood	
		u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}	u_{max}	Q_{max}
0	original	1.09	915	1.07	940	1.09	1810	1.07	1825
1	protect. + sill	1.78	1230	1.66	1310	1.78	1535	1.66	1635
2	sills -3	2.06	1180	1.90	1310	2.06	1475	1.90	1635
3	sills -2.5	2.48	1110	2.23	1305	2.48	1385	2.23	1630
4	sill -1	2.99*	710	3.49*	1135	2.99*	870	3.49*	1390

Table A6-11.

The discharge quantities in the remaining gap diminish with the progressing construction of the sill. The determining flow conditions are those in the narrow dip of every layer. The flow velocities in the various parts of the 1000 m gap are listed in Table A6-12. The phase with the determining flow conditions is phase 4, shown in the graph.

Comparing the phases 4 to 8 of this option, with phases 7 to 11 of the former option, it appears that the maximum flow velocity is considerably less (3.62 m/s and 4.22 m/s) as consequence of the longer gap length. All flow conditions reach the critical flow stage, except for in the last dip in the sill, which occurs as consequence of the construction of the layer up to the level of -1 m. In the dip the flood flow is still sub-critical. The magnitude is slightly less than the critical flow above the -1 m elevated sill, which is due to the low discharge-coefficient used for this narrow gap.

Even more important is the fact that the time window for the equipment to operate on the sill is much longer than in the former option. This is due to the lower low-water level in the basin for the comparable level of the sill. On the other hand, an operational difficulty of the option under consideration is that the layer construction has to advance over 500 m from each side instead of 350 m.

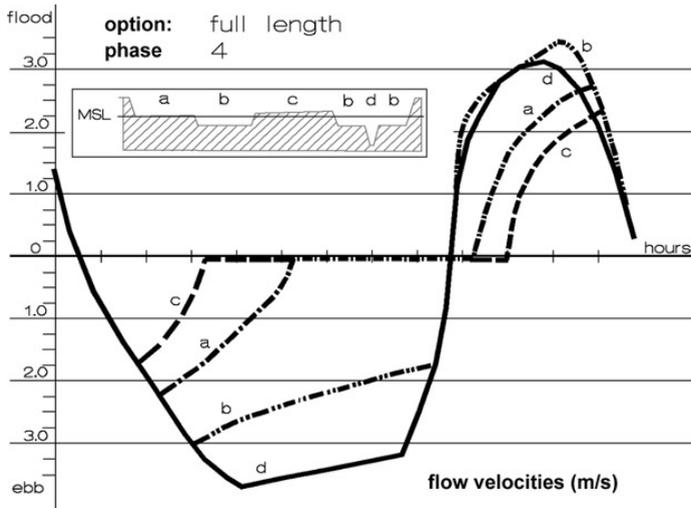


Figure A6-9 Full length vertically, phase 4.

u_{max} in m/s		deepest part		deepest but one		deepest but two		deepest but three	
phase	situation	ebb	flood	ebb	flood	ebb	flood	ebb	flood
4	sill -1	3.62*	3.09	2.99*	3.49*	2.24*	2.74*	1.68*	2.27*
5	up to MSL	2.97*	2.87	2.33*	3.05*	1.98*	2.32*	not applicable	
6	up to 0.5	2.32*	2.64*	1.95*	2.41*	not applicable			
7	up to +1	1.90*	2.05	1.11*	1.55*				
8	up to HW	0.88*	1.55*	high water free					

Table A6-12.

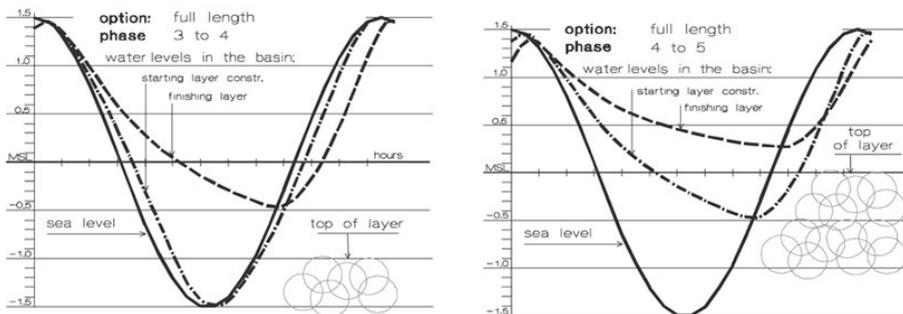


Figure A6-10 Full length, phase: 3 to 4 and 4 to 5.

An alternative way to avoid this problem of 500 m driving distance, is to prepare an approach-road towards the centre of the sill via an artificial island on the tidal flat. The construction can then advance from four sides each section being only 250 m long. This solution must be assessed against the extra cost of installing transshipment facilities. The total sill length is reduced only by the width of the construction island.

With certain dimensions the method is even changed and the first option with the final gaps in the channels, using vertical closure instead of caissons is followed.

An example of a closure by constructing sills in the gullies and providing one level over the full length of the alignment is the damming of the River Feni in Bangladesh in 1984/85. Owing to shallow water effects the tides were very variable. Spring ranges were double the neap ranges, while low water levels were always about the same. During spring tides the tidal wave entered the estuary as a tidal bore. Therefore, conditions during neaps were much more favourable than during springs and the last layers of the vertical closure had to be forced in a major effort over the neap tide period. During that closure day a neap-tide-safe profile had to be constructed on top of the sill, which had to be heightened up to a spring-tide-safe profile within a week's time.

The tide on the closure day rose from -0.5 m to +2.65 m (range 3.15 m), while the starting level of the sill was +0.70 m. On the closure day an embankment was constructed up to the level +3 m by piling up jute bags filled with clay. In total 1,000,000 bags were positioned by hand by 12,000 Bangladeshi people, in a time of five hours. In order to minimise the hauling distance the bags had been stored in 12 stockpiles along the alignment of the dam, which reduced the total length of the gap to about 1000 metres. Trucks tipping clay raised the profile to the spring tide safe profile.

APPENDIX 7

Construction equipment

General

This appendix (based on the Rock Manual [2007]) provides a review of the types of equipment used for placing armourstone in rockfill 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 discussed 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 **Error! Reference source not found.**, 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.

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 A7-1 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 m ³	2.80–3.50
Wheel loader	235–475	23–50	3.6–6.6 m ³	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 A7-1 Overview of equipment types with the ranges for power, mass, capacity and width.

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.

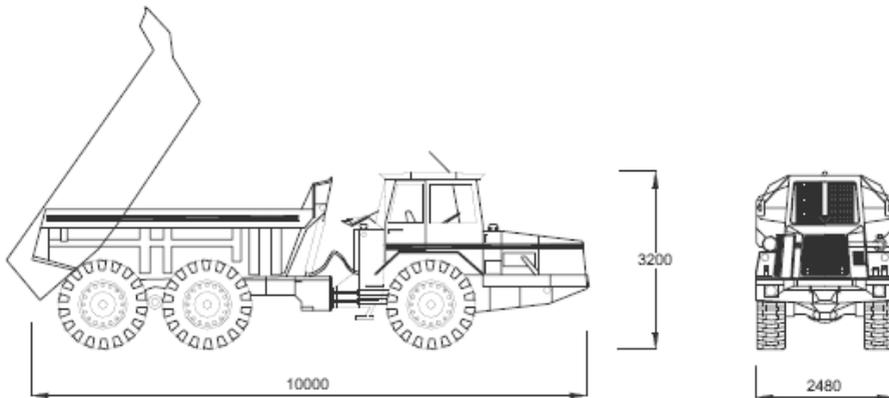


Figure A7-1 Large articulated dump truck, typical dimensions (mm).

Dump trucks are not designed to drive over armourstone; small material should therefore be used to blind 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 A7-2) may be used when the armour stones 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.



Figure A7-2 Wheel loader working from beached barge (Royal Boskalis Westminster).

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 A7-2 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 A7-2 Excavator size in relation to stone size.



Figure A7-3 Excavator working on the crest (courtesy J.D. Simm).



Figure A7-4 Crawler crane working on breakwater crest (courtesy Brien Wegner, USACE).

Hydraulic wire-rope or crawler cranes

Stone delivered by dump trucks or wheel loaders can also be placed by wire-rope

cranes (see Figure A7-4). 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.

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 A7-3) or a wire-rope crane (Figure A7-4). 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 A7-6) 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 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 A7-5 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.

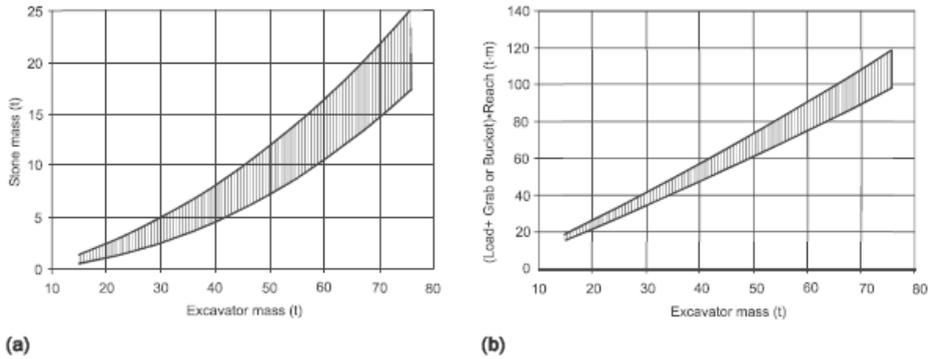


Figure A7-5 Indication of the relation between stone mass, required excavator size and maximum reach).

When fitted on to a hydraulic turntable, orange-peel or cactus grabs (see Figure A7-6a) 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 A7-6c) 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. 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.



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 A7-6 Examples of grabs used for armoured placement (Royal Boskalis Westminster).

The mass, reach and hoist moment of hydraulic excavators and crawler cranes (also called wire-rope cranes) are interrelated. Table A7-3 highlights some of the indicative relationships.

Characteristic	Unit	Relationship
Mass of hydraulic grapple	kg	$3.25 \times \text{grab volume (litres)} - 1910$
Mass of power fork	kg	$55 \times \text{excavator mass (tonnes)} + 200$
Mass of mechanically closed grapple	kg	$3.5 \times \text{grab volume (litres)}$
Mass of mechanically open grapple	kg	$2.5 \times \text{grab volume (litres)}$
Reach of hydraulic excavator	m	$5.8 + 0.06 \times \text{excavator mass (tonnes)}$
Hoist moment over front	tm	$1.6 \times \text{excavator mass (tonnes)} + 2.3$
Hoist moment over side	tm	$1.2 \times \text{excavator mass (tonnes)} - 7.6$
Reach of crawler cranes	m	$5.2 \times (\text{crane mass (tonnes)})^{0.4}$
Hoist moment of crawler crane	tm	$0.4 \times (\text{crane mass (tonnes)})^{1.31}$
Rope-operated grabs for crawler cranes:		
• closed-tine grab	t	$3.5 \times \text{grab volume (m}^3\text{)}$
• open-tine grab	t	$2.5 \times \text{grab volume (m}^3\text{)}$
Hydraulic grabs for hydraulic excavators:		
• closed-tine grab	t	$2.25 \times \text{grab volume (m}^3\text{)}$
• open-tine grab	t	$1.55 \times \text{grab volume (m}^3\text{)}$
• power fork	t	$0.06 \times \text{excavator mass (tonnes)}$

Table A7-3 Indicative relationships between various machine characteristics.

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
- flat-top barges with wheel loader (see Figure A7-2 for an example)
- crane barges equipped with rock trays/skips
- side stone-dumping vessels or side-unloading barges (see Figure A7-10 and Figure A7-9).

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 A7-7 shows the use of a skip (or rock tray) for breakwater core construction.



Figure A7-7 Placing armourstone from floating barge using a rock tray/skip.

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 A7-8.

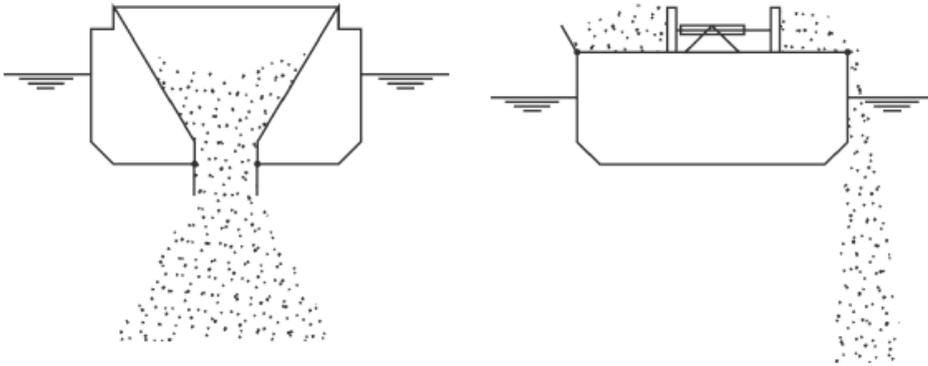


Figure A7-8 Types of self-unloading barges for direct dumping of bulk armourstone or core material.

The mechanism of unloading from side-unloading barges is by sideways movement of the sliding shovels, as shown in Figure A7-9. 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.

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 A7-9) 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 m³/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.

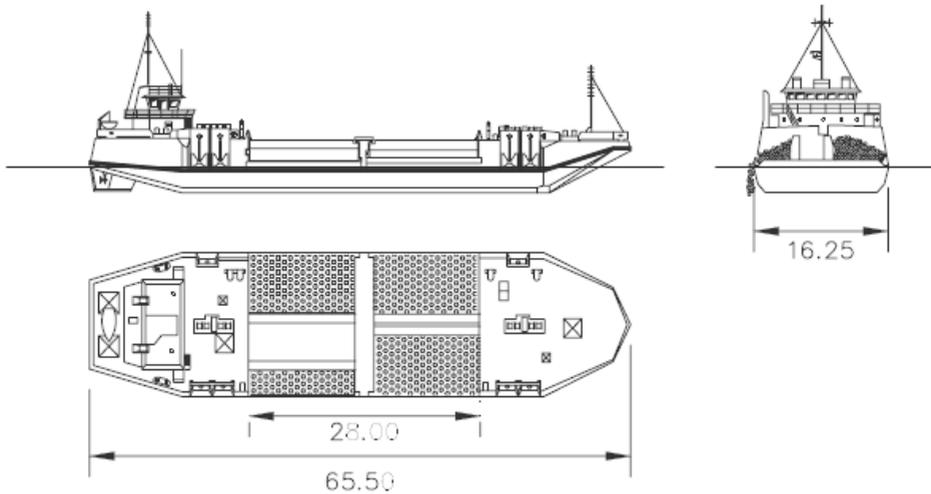


Figure A7-9 Plan view and cross-section of the 1000 t side stone dumping vessel Frans, dimensions in m (Van Oord).

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 armour stones can be dumped by side stone-dumping vessels, even very close to existing structures, as shown in Figure A7-10.



Figure A7-10 Side stone dumping vessel Cetus, dumping 1-3 t armourstone near a jetty (Royal Boskalis Westminster).

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 A7-11 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.



Figure A7-11 Placing armourstone with an excavator (Van Oord).

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 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 A7-12.



Figure A7-12 Crane mounted on the rock supply vessel Jan Steen (Van Oord).

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 Amphirolo are examples of these vehicles. The hovercraft supports itself by an air-bell 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 A7-13) 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.



Figure A7-13 The Amphirol.

For the heavy equipment the right system of movement has to be selected. If moving speed is important 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.

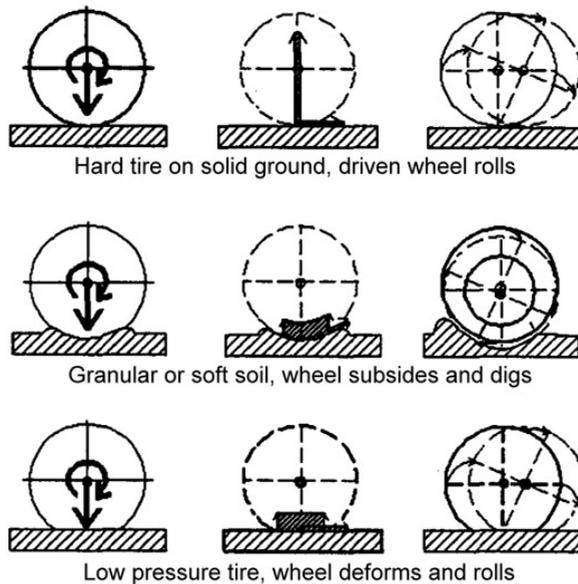


Figure A7-14 Motion of a driven wheel

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 are 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.

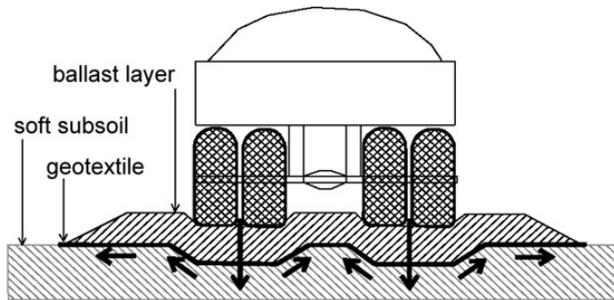


Figure A7-15 Temporary road on soft subsoil.

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 shear 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.

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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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List of symbols

Basic units: kg, m, s, €

Other units: $N = \text{kg}\cdot\text{m}/\text{s}^2$ $\text{Pa} = \text{N}/\text{m}^2$
 $J = \text{kg}\cdot\text{m}^2/\text{s}^2$ $W = \text{J}/\text{s}$

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
B_s	m	flow width in a closure gap
C	$\text{m}^{1/2}/\text{s}$	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)
Ho	-	stability number, as used in berm breakwater calculations ($Ho = H_s / \Delta d_{n50}$), 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)
$H_{s/1500}$	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_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
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_{omove}	-	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
To	-	dimensionless wave period, used in berm breakwater calculations ($To = T_m / \sqrt{g/d_{n50}}$); 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
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

Index

A

advantages of monolithic breakwater · 250
 advantages of the rubble mounds · 250
 armour layer · 159

B

bathymetry · 80
 berm breakwater · 115, 156
 breakwater head · 110
 building yard · 222

C

caissons · 222
 caissons closed with sluice gates · 218
 caissons provided with sluice gates · 218
 closing an estuary · 167
 closure by hydraulic filling with sand only · 206
 closure dams · 252
 combination of floating and rolling equipment · 205
 combined vertical and horizontal closure · 9
 comparison of Hudson and Iribarren formulae · 101
 composite types · 6
 concrete · 88
 concrete armour units · 158
 concrete blocks · 104
 construction materials · 85
 construction methods for granular material · 189
 construction methods for monolithic structures · 218
 construction of granular filters · 194
 construction of mattresses · 193
 crest · 159
 crest design · 158
 crest freeboard · 149
 cubes · 105
 cyclic design · 25

D

design phases · 25
 design practice · 152
 disadvantages of monolithic breakwater · 251
 disadvantages of the rubble mound · 250
 Dolos · 105
 dynamic forces · 127

E

equipment · 89
 exceedance graph · 264
 exponential distribution · 266

F

failure mechanisms · 232
 failure modes · 41, 231
 fault tree · 235
 filter · 161
 first under-layer · 160
 floating equipment · 203
 flood disaster of February 1953 · 19
 flow in a river channel · 242
 flow in closure gap · 242
 flow in the entrance of a tidal basin · 244
 flow through gaps · 51
 functions of breakwaters · 32
 functions of closure dams · 43

G

geotechnical data · 83
 geotechnics · 72
 geotextile · 161
 Global Wave Statistics · 273
 Goda · 285
 Goda formula · 130
 grading of quarry stone · 108

gradual closure · 8
 groundwater · 78
 guiding of currents · 37
 Gumbel distribution · 268

H

handling of quarry stone · 195
 high crest · 158
 horizontal (gradual) closure · 8
 Hudson formula · 278
 hydraulics of flow · 51
 hydraulics of tides · 50

I

Iribarren · 91
 irregular waves in shallow water · 69

L

labour · 89
 layer thickness · 153, 154
 liquefaction · 76
 log-distribution · 276
 lognormal distribution · 283
 long term statistics · 67
 low and submerged breakwaters · 112
 low crest · 158

M

mattresses · 193
 meetpost Noordwijk · 261
 metacentric height · 61
 meteorological data · 80
 method of partial coefficients · 279
 minimizing risks during construction · 211
 modelling · 56
 monolithic breakwaters · 220, 250
 monolithic types · 5
 morphology · 43
 mound types · 5
 multi-layered breakwater · 157

N

nautical characteristics · 42
 normal distribution · 283
 number of caissons · 230
 number of sluice gate caissons · 230

O

optimization · 240
 overtopping · 137
 overtopping and transmission for vertical walls · 146
 overtopping for rubble mounds · 143

P

Peak over Threshold method · 264
 permeability · 153
 porosity · 153
 port facilities · 36
 PoT analysis · 273, 276
 probabilistic approach · 283
 probability of exceedance · 267
 protection against shoaling · 38
 protection against waves · 33
 providing of quarry stone · 195
 provision of dock or quay facilities · 40

Q

quarry stone · 85, 195
 quasi static forces · 126

R

reflection · 138
 rock armour units · 158
 rolling equipment · 199
 rubble breakwaters · 250
 run-up · 139

S

sailing vessels · 35
 scour · 134
 scour prevention · 190
 scour protection · 161
 separate compartments · 252
 service limit state · 252
 shape of quarry stone · 107
 shore protection · 36
 short term statistics · 62
 sinking operation · 226
 sliding · 73
 special (unconventional) types · 6
 squeeze · 75
 stability of floating objects · 58
 stability of rock · 90
 standard cross-sections · 162
 sudden closure · 9

T

Tetrapods · 105
 toe · 109
 toe berm · 160
 transmission by rubble mounds · 147, 150
 transport · 222
 types of breakwaters · 5

U

use of floating equipment · 197

use of mattresses · 193

use of rolling · 197

use of rolling equipment · 197

V

Van der Meer formula · 285

vertical (gradual) closure · 9

vessels at berth · 33

W

Walcheren · 17

wave forces · 126

wave reflection · 137

wave run-up · 137

wave transmission · 137

Weibull distribution · 270

work-window of flow conditions during the sinking
operation · 228

Z

Zuiderzee · 17