Breakwaters and closure dams

On the cover picture:

Ship approaching a harbour, Willem Gruijter jr, (1817-1880)

Oil on Panel, 23 × 34 cm, Belasting- en Douanemuseum, Rotterdam

A ship is approaching a harbour, protected by a breakwater made from woodwork (palisade). Harbour approach indicated by a lighthouse, with an oil lamp. On the breakwater is fire basket as leading light. A wooden buoy is floating in front.

Breakwaters and closure dams

Henk Jan Verhagen
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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.

The first edition of this book (2001) was written by Kees d'Angremond and Ferd van Rooden. 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 been 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. We are especially grateful to Margaret Boshek, who checked both the English spelling as well as the readability of the book.

Henk Jan Verhagen, Kees d'Angremond
Delft, January 2009

Contents

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1 INTRODUCTION

1.1 Scope

For this book we have deliberately chosen that the text should follow a more or less logical design procedure for both breakwaters as well as closure dams. This means that in each step of the procedure attention is paid to both breakwaters and closure dams 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 closure dams, it is emphasized that only the constructural aspect of stopping the water movement is 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 possesses basic knowledge of hydraulic engineering. Only in some cases, where they are deemed useful for a proper understanding of the actual design process, are aspects of basic hydraulic engineering presented.

1.2 References

This book is an educational textbook, not a design manual or a reference book. The focus of this book is the understanding of the basic principles. It is not an overview of all existing formulas pertaining to breakwater or closure dam design. Also, because the results of new research will modify existing formulas, it is not useful to focus on the minute details of such formulas, but more on the physical concepts behind the formulas. Although a study book has its purpose, there are some outstanding reference books in the field cited by this textbook and these are often far more comprehensive than any study book. Therefore a number of books and

periodicals that are available to any engineer in charge of the design or construction of breakwaters or 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 Water defences (ENW, formerly TAW, in Dutch). For wave-structure interaction, refer to the European Overtopping Manual (PULLEN *ET.AL* [2007]). 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 yearly proceedings of the international conferences on Coastal Engineering and on Coastal Structures.
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 9. For Dutch students an English-Dutch glossary is available on the above mentioned "blackboard" site. The reader is also referred to a more general vocabulary on hydraulic engineering (http://www.waterdictionary.info).
In this book, the metric (mks) system (based on the definition of mass [kg], length [m], and time [s] has been used, except for some widely accepted nautical and hydrographic terms such as knots, fathoms and miles.

2 POSITIONING THE SUBJECT

2.1 General

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

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 other engineering works have such an extensive impact on the environment in all aspects. For instance, the main purpose of the construction of the Afsluitdijk closure dam in the Netherlands was to provide protection against high storm surge levels and to facilitate land reclamation. Additional advantages were fresh water conservation and a road connection between the provinces of Holland and Friesland. The purpose of a closure dam may be one or more of such objectives, but these are automatically accompanied by other side effects, some of which may be negative. A thorough study of these impacts is part of the design process. A feasibility study that does not detail and forecast the negative aspects of the closure works is incomplete and valueless. These unforeseen negative effects for the Afsluitdijk include: the drastic change in tidal amplitude in the Waddenzee, 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 the ground water table in the surrounding land areas, the changes to the fisheries industry, and effects on flora and fauna.

Non-technical aspects, including environmental, social and cultural values, cannot be expressed in financial terms. The evaluation of such considerations is not within the scope of this book. Nevertheless, engineers must identify the consequential effects to the best of their ability and present them in such a way that they are understood by decision-makers.
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 against nature. Every action taken to obstruct the water flow will immediately be counteracted by nature itself. The knowledge gained from experience, whether successful or not, 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. Allowing for flexibility in operations that are 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 of closure 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 construction procedures 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, similar equipment, either land based or water-borne, is used: e.g. 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 structure 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 simply large heaps of loose elements, such as gravel and quarry stone or concrete blocks. The stability of the exposed slope of the mound depends on the ratio between load and strength i.e. wave height (H) versus size and the relative density of the elements (Δd). On one extreme, for example, is a gravel beach that is subject to continuous changes in the equilibrium profile as the wave characteristics change and also due to longshore transport. On the other extreme, for example, is the 'statically stable breakwater', where the weight of the elements in the outer armour layer is sufficient to withstand the wave forces. Between these two extremes is the 'berm breakwater', where the size of the armour is not sufficient to guarantee stability under all conditions, but where some extra quantity of material is provided so that the slope of the structure can reshape between given limits. Typical values of $H/\Delta d$ for the three types of structures are given in Table 2-1.

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

Table 2-1 Characteristic values of $H/(\Delta d)$

Monolithic types

Monolithic breakwaters have a cross-section which acts as one solid block. Types of monolithic structures include caissons, a block wall, or a masonry structure. This type of structure can be categorized by a typical value of $H/\Delta d$ that is given (as caisson) in Table 2-1. The main differences between the mound and the monolithic types of breakwaters are caused by the interaction between the structure and the subsoil and also by the behaviour at failure. The mound-type structures can be considered flexible (i.e. they can follow uneven settlement of the foundation layers), whereas monolithic structures require a solid foundation that can cope with high and often dynamic loads. The behaviour of the structures when close to failure is also quite different. When a critical load value is exceeded, a monolithic structure will lose stability at once, whereas a mound type of structure will fail more gradually as elements from the armour layer are displaced. However, because of the sloped construction, the footprint of a rubble mound breakwater is much larger. Where construction restrictions related to depth or environmental issues are a concern a vertical wall breakwater may be the better option.

Composite types

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

Special (unconventional) types

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

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

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

Apart from a division between the categories described so far, there is also a distinction in terms of the *freeboard* of the crest above the still water level (SWL)[1]. Traditional structures usually have a crest level that is only overtopped occasionally. It is also possible to choose a lower crest level that is overtopped more frequently, or even completely submerged. When a low crest level is combined with the design philosophy of a berm breakwater, (i.e. a reshaping mound) it is termed a reef-type breakwater. Examples of all types of breakwaters are shown in Figure 2-1 to Figure 2-4.

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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).

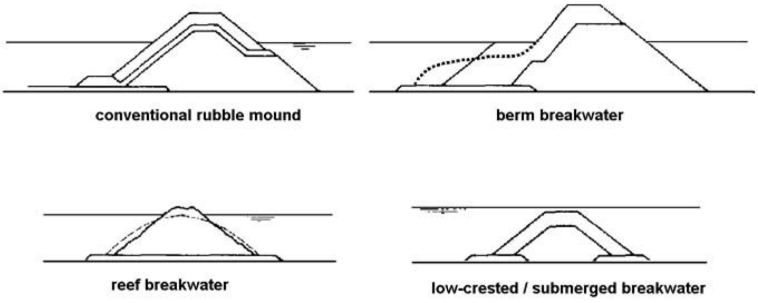


Figure 2-1 Mound breakwater types

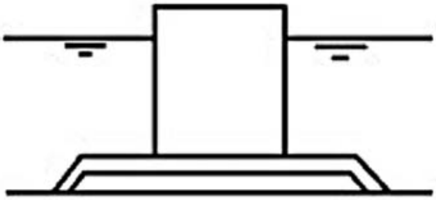


Figure 2-2 Monolithic breakwater type

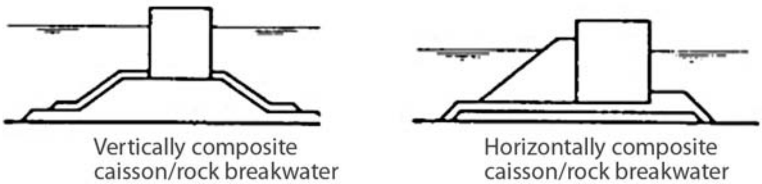


Figure 2-3 Composite breakwater types

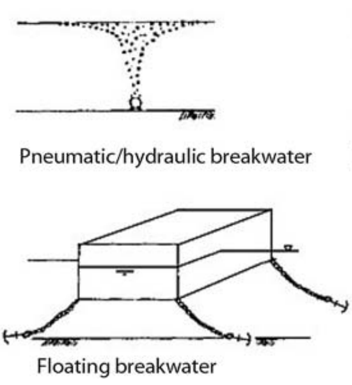


Figure 2-4 Special breakwater types

In this 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.

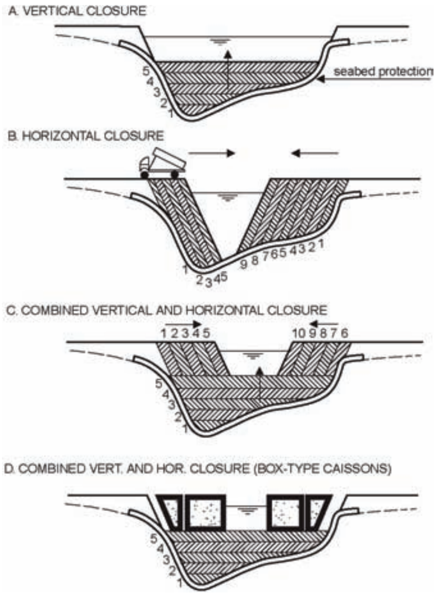


Figure 2-5 Basic methods of closure

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

[2] In very exceptional cases helicopters are used.

either a vertical, horizontal, or a combined closure:

- Horizontal (gradual) closure: sideways narrowing of the closure gap.
- Vertical (gradual) closure: consecutive horizontal layers closing 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.

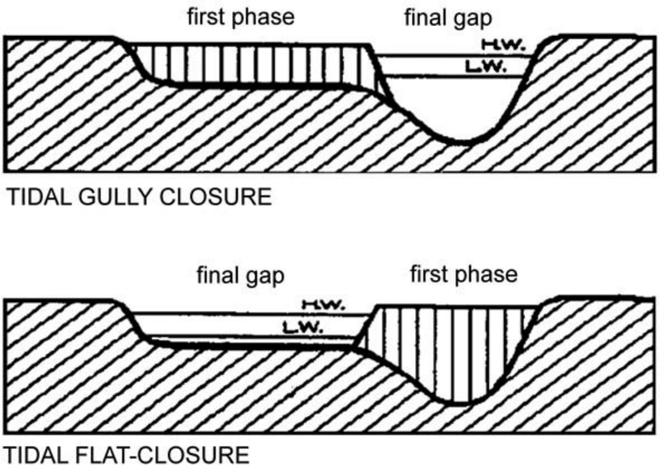


Figure 2-6 Closure named after topography

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.

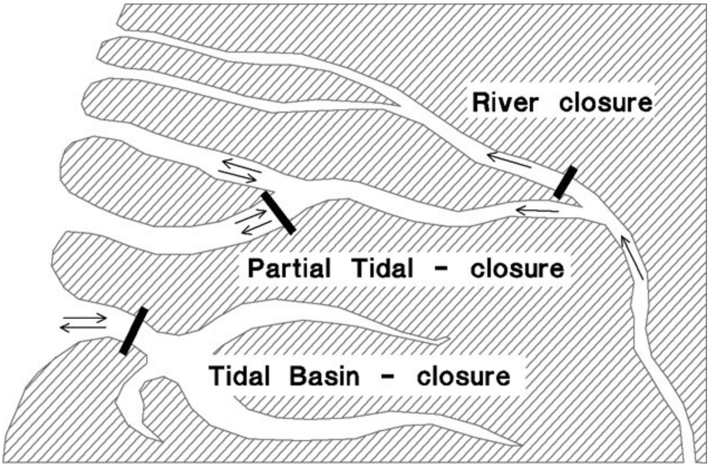


Figure 2-7 Closures named after hydrologic conditions

The *materials used*, which may vary according to the method of closure:
- Stacking-up mattresses: Closure realized by successively dropping mattresses (made of willow or bamboo 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.
- 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).

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. The method is mainly used for closing dike breaches quickly which may require strengthening afterward.
- Temporary closure is used to influence the conditions elsewhere; for instance, by stepwise reduction of the dimensions of the basin. This type of closure needs to be sufficiently strong during the required period but is easily removable afterward.

2.4 Historical breakwaters

The first breakwaters that are described in traceable sources date back to ancient Egyptian, Phoenician, Greek and Roman cultures. Most of them were simple mound structures, composed of locally found rock. As early as 2000 BC, mention was made of a stone masonry breakwater in Alexandria, Egypt (TAKAHASHI [2002]). The Greeks also constructed breakwaters (mainly rubble mound) along some parts of the Mediterranean coast. The Romans constructed true monolithic breakwaters once they had mastered the technique of making concrete. The Roman emperor Trajan (AD 53 - 117) initiated the construction of a rubble mound breakwater in Civitavecchia, which still exists today (Figure 2-8). 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])

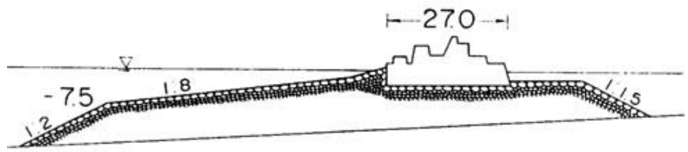


Figure 2-8 Rubble mound breakwater at Citavecchia

In modern times similar breakwaters were constructed at Cherbourg (France) (1781/1789/1830), and at Plymouth (UK) (1812/1841). In both cases, the stability of the seaward slope was insufficient and during subsequent repair operations the final slopes were between 1:8 and 1:12 (See Figure 2-9 and Figure 2-10).

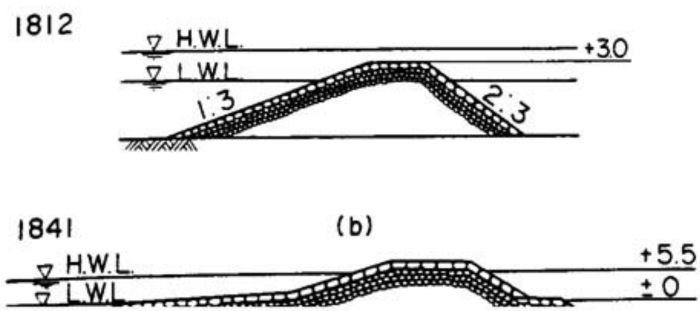


Figure 2-9 Breakwater at Plymouth

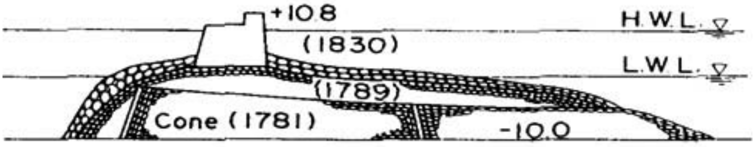


Figure 2-10 Breakwater at Cherbourg

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

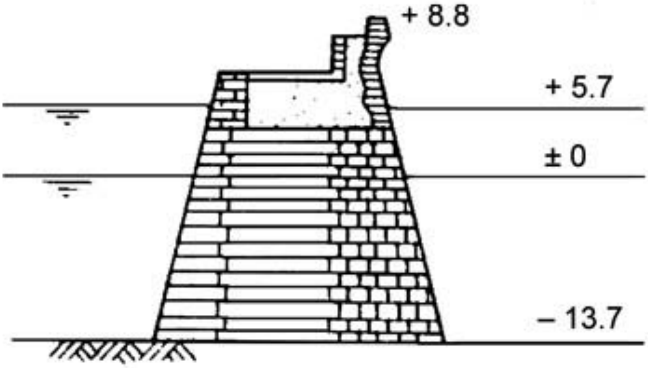


Figure 2-11 Monolithic breakwater at Dover

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 to protect an expanding fleet of vessels. British engineers, in particular, put the lessons learned from the Dover breakwater construction to use. To avoid the problems of construction in deep water, rubble mound berms were used for the foundation of monolithic superstructures, and thus the first real composite breakwaters came into existence. Here also, however, the process of trial and error took its toll. Many breakwaters had to be redesigned because the berms were originally erected too high and where subject to instability due to wave action.

In France, engineers tried to solve these stability problems by designing flatter slopes above SWL, and by applying extremely heavy (cubic and parallelepiped) concrete blocks as the armour layer. They also started to use smaller-sized stone in the core of the structure. The breakwater at Marseilles (1845) was seen as a success among French engineers just as the Dover type of breakwater was a success for the British. However, it was recognized that the Marseilles type of solution required very heavy armour units and also a lot of material in the cross-section, especially in deeper water (Figure 2-12).

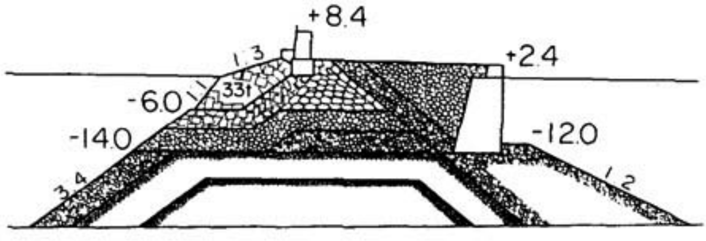


Figure 2-12 Breakwater at Marseilles

These developments made the composite breakwater the most widely used breakwater type in the early 20th century. In Italy, where many breakwaters were constructed in relatively deep water along the Mediterranean coast, the logical solution appeared to be a composite structure consisting of a berm to about half the water depth with a vertical faced wall on top of it. The wall was built of extremely large (Cyclopean) blocks, sometimes interlocking to create the monolithic effect (Figure 2-13). However, these breakwaters were not a success. The shallow berm caused waves to break and slam against the vertical wall causing high impact forces which led to the eventual failure of the breakwater itself.
Concerns over these failures led to the creation of an international association for hydraulic research (IAHR) by PIANC[3] port engineers. The failures of the vertical-wall breakwaters around the Mediterranean in the first half of the 20th century marked the end of the popularity of this type of breakwater in western Europe.
The French continued their efforts to optimize their rubble mound concept. To reduce the required weight of the armour blocks, they developed the concept of interlocking them. Thus, in 1949, P. DANEL [1953] of the Laboratoire Dauphinois d'Hydraulique (later Sogreah) designed the Tetrapod armour unit, which was the start of a long series of similar blocks. For a time, the Dolos designed in South Africa, seemed to provide the ultimate solution, until the limited mechanical strength of this block triggered a new series of mishaps. The development of special shaped blocks went on, however, resulting in two other French blocks, which are still quite successful: the Antifer cube and the Accropode. In the US, a stronger version of the Dolos unit was developed, the Core-Loc. In the Netherlands, Delta Marine Consultants created the Xbloc.

3 Permanent International Association of Navigation Congresses.

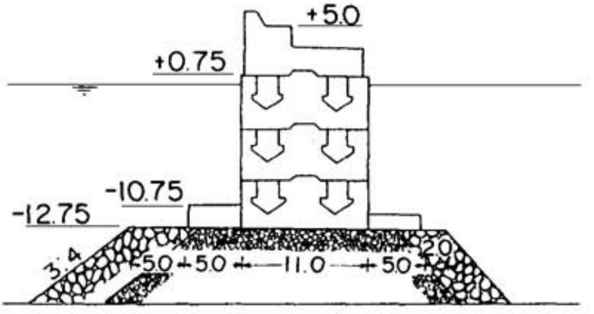


Figure 2-13 Typical breakwater along the Mediterranean coast

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

2.5 Historical closures

Closure dams have most likely been constructed since mankind started performing 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 recordings 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 remains are found today, even though they might have been quite extensive, considering that the builders were able to construct the pyramids.

The damming of the rivers Rhine and Meuse in the late Middle Ages

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

In order to prevent the river Rhine, 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 recordings give a fair idea about the old methods used. The dams had to be constructed from locally available materials that could be lifted by hand and simple equipment. These materials were typically not stable under conditions of high flow velocities. Therefore the procedure was to limit the flow velocities during the 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 from trial and error indicated the maximum area that could be closed 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) was made. When ballasted with clay this could be sunk onto the bottom. The closure was created 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 south western part of the Netherlands 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, with a local tidal range of about 4 m. By sinking mattresses, a sill was constructed up to the low water level. This sill had side slopes of 1V:1H and a crest width of 18 m. The next stage was to construct an osier revetment on top of the sill. In consequence of the added weight, the sill settled 1.80 m. In order to fabricate the wall up to high water level (at a height of 4 m above the original height of the sill), a 5.80 m high dam had to be made which took a full month to construct. Part of the final profile was made by adding a clay cover over the osier revetment.
In the cases where the construction of an osier revetment failed, an attempt was made to position a vessel in the final gap and sink it onto the sill. This was not a simple operation, as transport was done by sailing or rowing and hand winching was the only driving force. Timely ballasting and the prevention of the escalation of piping

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 Dagebüll (northwest Germany) in 1633. Clay had to be transported over a long distance by sailing vessels and it took an estimated 5500 labourers to execute the job. The closure was completed by sinking a vessel in the gap. This was then ballasted by 350 cart loads of clay.

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 were used, for which a stable railway had to be constructed. Therefore, initially, the only change in construction method 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 created ground.
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 northwest 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 newly created ground, which often subsided, and derailments frequently occurred, thus escalating the problems. Later, the work method was modified. The preceding scour was solved 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 leads to halting of the complete works. The system is less flexible than one using manual labour.

Learning how to adapt existing methods and the use of the new equipment also stimulated the development of new methods. New engines could handle heavier units and reach higher production capacities. The advantages of engine use 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 Afsluitdijk. The 32-km long dam crossed two main gully systems. During the execution of the works large deposits of boulder-clay 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 were 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. In 1932, for the damming of the enormous tidal basin, the Zuiderzee (now called IJsselmeer), the differential equations for tide-propagation had to be solved. Professor Lorentz, a Nobel Prize winner in physics, was able to achieve this. Three questions had to be answered before the job started:

- How would the tide change when the works were in progress, and would this affect the closing conditions?
- How would the tide change when the works were completed, and would this affect the design water level of the dike?
- What other design conditions would affect the profile of the dike in the 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. This action dislodged the enemy troops and opened the fairway to Antwerp for the allied army fleet. However, at the same time, it demolished the sea-defences and opened

the low-lying island to tidal penetration. Restoring the sea-defences had to be completed quickly in order that the island would not be permanently lost. Again, the mathematical basis for calculating tide-propagation improved. The four gaps in the dike, (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. Moreover, owing to the progressive erosion of gullies, the hydraulic resistance changed with time. Mathematical analysis was needed to establish the most favourable order of progress and also to ascertain risks that would arise if a different path should occur in practice.
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). In the left figure the extend of the flow is indicated (note that there is an overlap in the basins, some of the water entering the island through the gap of Westkapelle is leaving the island via the gap of Veere). In the right figure the gully formation is indicated.

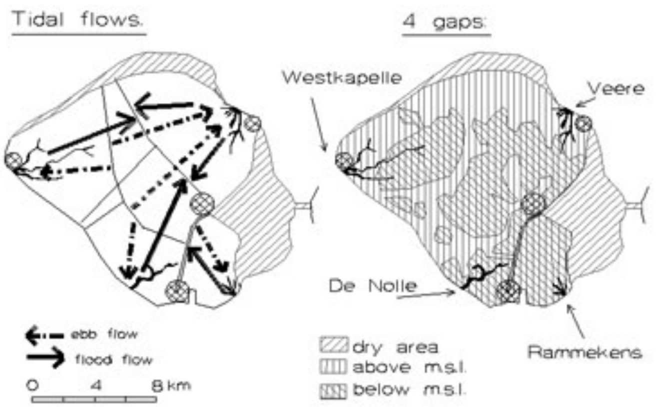


Figure 2-14 Walcheren - four gaps on one island

Due to the concurring war, there was no material or equipment available and the areas were covered with mines. In June 1945, when at last construction could start, 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 the 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 laying scour-protection in the gaps, a variety of large units, such as pontoons, caissons, concrete, 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. However, by the end of January 1946, the gaps were closed. A very good description of the difficulties encountered is given in the novel "Het verjaagde water" by A. den Doolaard.

Through this project, 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 in 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 Western Scheldt river, 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 closure and thus restrict the forces.

1953 and the Deltaworks

On February 1st, 1953, a flood disaster occurred in the southern North Sea. Storm surge, together with spring tide-high water, inundated 2000 km^2 of land in the Dutch Delta, creating 73 major dike-breaches and numerous smaller ones. Again, all available technical experience, equipment, and improvisation had to be used on many sites simultaneously to close these gaps before the next winter season. Initially, the gaps varied in degree of difficulty or dimension. However, many 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 river. While not initially a threat, this became one of the major dike breaches that occurred. The scouring process continued during the actual closure works as well. The gap increased from an initial 40 m width (on February 1st) 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 mean sea level, overtopping the dike. The unprotected inner slope of the dike slid down over a length of approximately 40 m and the top layer of the dike scoured away. However, the slope protection on the outer side remained intact up to the level of +1.70 m as it rested on century-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.

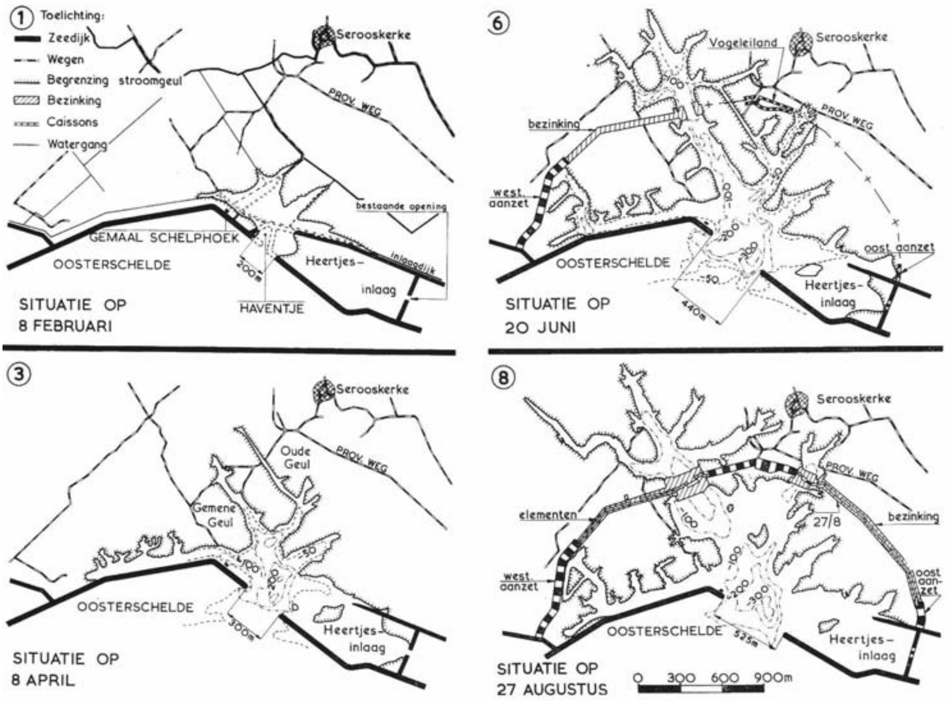


Figure 2-15 Development of erosion gullies in Schelphoek (after the breach of 1 February 1953)

These numerous difficult circumstances led to various innovative actions, which resulted in complete repair within 10 months. Table 2-2 illustrates this enormous achievement.

Once again, the experience was used in later developments of closing technology. This is shown by the following example: The principles of a temporary closure made in 1953 near Kruiningen (in the south west of the Netherlands) were 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 m^3 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.

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

The disastrous flooding in 1953, was a catalyst for a new decision making process for the reconstruction of sea defences in the Netherlands. In order to avoid strengthening 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 current experience, it was possible to develop the required new methods during the period of construction by 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.

Period after 1975

Around 1975 enhanced world views regarding ecological importance altered the design of closures. 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 constructed and the creation of the new design and its execution were parallel, many problems arose in this period. A lot of new ideas were 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.

Name of Estuary	Total length of closure dike	Tidal range (m)	Area (km2)	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.

The experience gained during the execution of the Deltaworks has been applied by other closing projects across the rest of the world. Important closure works to mention are a number of estuaries closed during the period of 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.

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

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

3.2 Abstraction level

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

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

A few examples are presented in Table 3-1.

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

	Macro level	Meso level	Micro level
General terms	System	Component	Element
Example 1[a]	Harbour in the global and regional transport chain	Harbour layout	Breakwater
Example 1[b]	Harbour layout	Breakwater	Crest block
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 design phases that in some countries are related to the general conditions of contract between employer and consultant. Therefore the phases may vary from country to country. The contractual contents of each phase are subject to modifications in the same way. A logical set of phases are:

Initiative
Formulation of the ultimate goals of the design object as part of the system.
Feasibility
Review of the system with respect to technical, economic, social and environmental consequences and feasibility. Requirements are formulated on the component level.
Preliminary design
Giving shape to the system on broad lines, including determination of the exact functionality of the components and definition of requirements at the element level.
Final design
Composition of a set of drawings and specifications for the system in which the final shape of the components is fixed and the functionality of the elements is determined.
Detailed design
Composition of a set of drawings and specifications in which the final shape of the elements is fixed.

This concept can easily be schematised in a matrix in which each row represents one of the phases and shows which activities will take place at the various levels of abstraction. The columns show how the levels of abstraction in the project become more concrete throughout the phases. The matrix also shows that working on the elements does not start before one reaches the preliminary design phase and certain decisions have been taken about the purpose and function at the system level and about the purpose at the component level.
Following this line of thought helps to ensure that the proper approach is chosen at each stage so that neither too much nor too little detail is sought.

3.4 Cyclic design

Each activity in the design process, which is represented by a cell in Table 3-2, is a cyclic process in its own right, consisting of a number of steps:

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

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

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

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

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

3.5 Consequences of systematic design

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

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

When these questions have been answered, can one start to think in more detail about specifics such as the breakwaters, starting with a rough layout and an indication of the required functions. Only in the final stage of the design process, can the actual design of the cross-section be made, including decisions about crest level, slope, and choice of materials and construction method.
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 must still be determined. One must have insight into the hydraulic system of the flow, the subsoil conditions in the area, and 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 between the purpose and functionality of the system. At the same time, it will be clear that certain details of the design need not be worked out in the early stages. It makes no sense to plan a working harbour in detail before the closure method has been chosen.

3.6 Probabilities

No construction can be designed in such a way that it will never fail. However, the probability of failure has to be very small. The probability of failure of a structure is partly a financial problem (the extra cost of lowering the probability of failure has to be lower than the capitalised cost of failure), and partly depends on non-monetary values, such as loss of life, ecological damage, etc. In case probability of failure is mainly a financial problem, the optimum probability of failure can be computed; this will be explained later. In case numerous non-monetary values are at stake (e.g. a dike protecting an urbanised area or a natural reserve), an objective optimisation is

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

3.6.1 Basics of a probabilistic analysis and the use of safety coefficients

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

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

where R is the strength and S is the load[4]. 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:

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

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

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

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:

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

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

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, such as protection of a museum or a religious site.

In the first case, one can calculate the optimal design conditions based on economic restrictions. In the second case, these values cannot be calculated but are subject to political decision making. Typically, for breakwaters, it is purely an economic problem. In case of failure there will be damage: the cost of repairing the direct

damage plus the loss of income during non-operation of the breakwater (consequential damage). The details of the economic optimization will be explained in Appendix 6 of this book.
Often such an economic optimization is not made. This is usually due to the fact that decisions on the investments for a breakwater project are not based on proper life-cycle analysis, but on the budget available or on the (short-term) rate or return on the initial investments. Therefore in practice often a political decision is made on the return period of the design storm, based on ad-hoc considerations.

3.6.3 Determination of a design storm

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

$$p = 1 - \exp\left(-f\,T_L\right)$$

in which:

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

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

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

This means that there is probability of 63% that the construction will fail during its lifetime. It is clear that this is unacceptable. More acceptable values would be between, say, 5% and 20%. The actual choice depends largely on the purpose of the structure and on the risk involved. In this book, some examples have been worked

out based on the relatively high value of 20%[5]. This must not be interpreted as a recommendation, but just as an example!
It means that the storm frequency becomes:

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

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.

[5] 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, where the system is either a port or a scheme to close a river or estuary. The breakwater or the closure dam is then an element of that system. By discussing the system, we attempt to approach our design problem from a slightly more abstract position. This refers to both the functions and requirements, and to the side effects of the project.

4.2 Functions of breakwaters and examples

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

- Protection against waves (Section 4.2.1). This can be subdivided into protection of ports and shipping and 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 (sailing with or without tugs, moored, being loaded/unloaded) or installations that are to be protected makes a big difference to what is required. In other words, one must have an idea how vulnerable the area to be protected is before deciding what degree of protection must be provided.

In general, a vessel is most vulnerable when it is moored alongside a rigid structure such as a quay, a jetty, or alongside another vessel. The acceptable wave height is related to the size of the vessel, on one hand, and the height, period and direction of the waves, on the other hand. THORESEN [2003] gives suggestions for ships at berth in head seas. These values are slightly modified in Table 4-1 according to the experience of the authors. The acceptability of the conditions refers to both damage to the vessel and damage to the structure.

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 Serviceability Limit State (SLS) that are usually different from the Ultimate Limit State (ULS), which concerns the survival of the structure under extreme conditions.

Figure 4-1 shows the layout of a harbour where the breakwater typically protects the harbour basin, including berths for loading and unloading.

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 year.

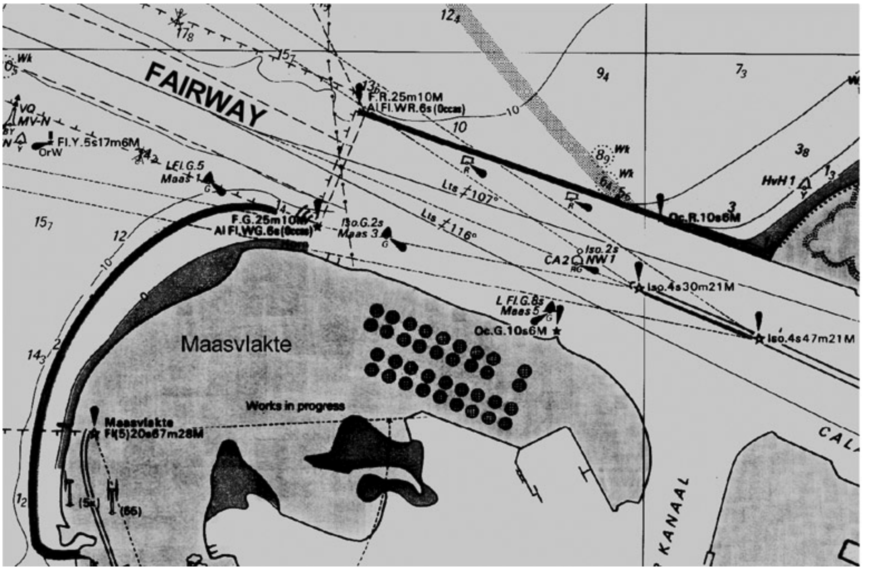


Figure 4-2 Breakwater at the Europoort entrance

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

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

The entrance to the Port of Rotterdam is a good example of an entrance where the layout of the breakwater is designed to cope with the current pattern (Figure 4-4). In this case, the function of the breakwater is twofold: it guides the current and it damps the waves to a level at which the tugs can work.

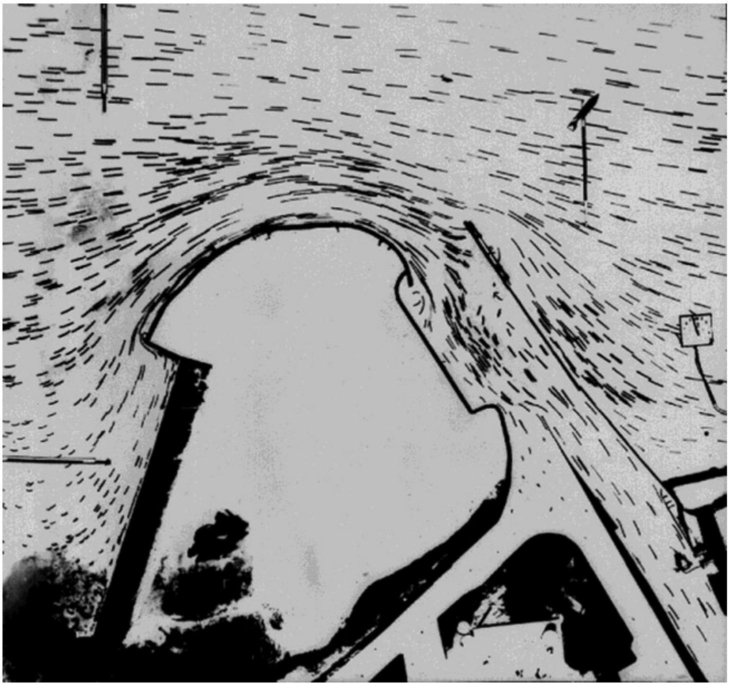


Figure 4-4 Flow pattern at the Europoort entrance

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).

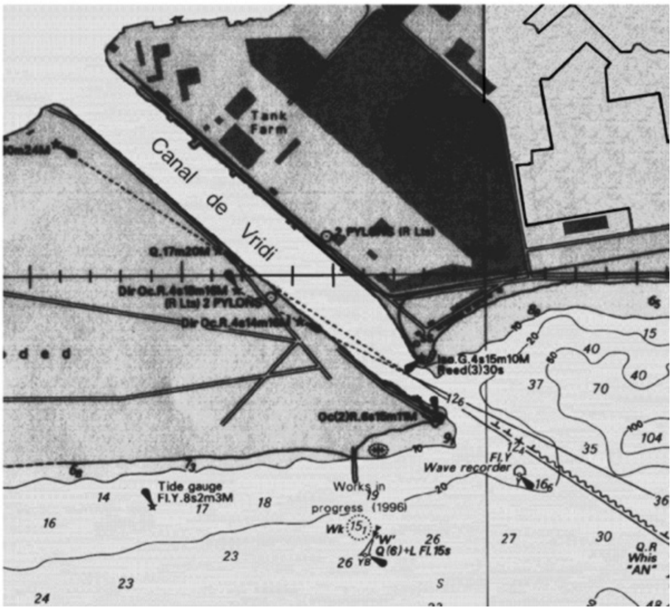


Figure 4-5 Entrance to the port of Abidjan

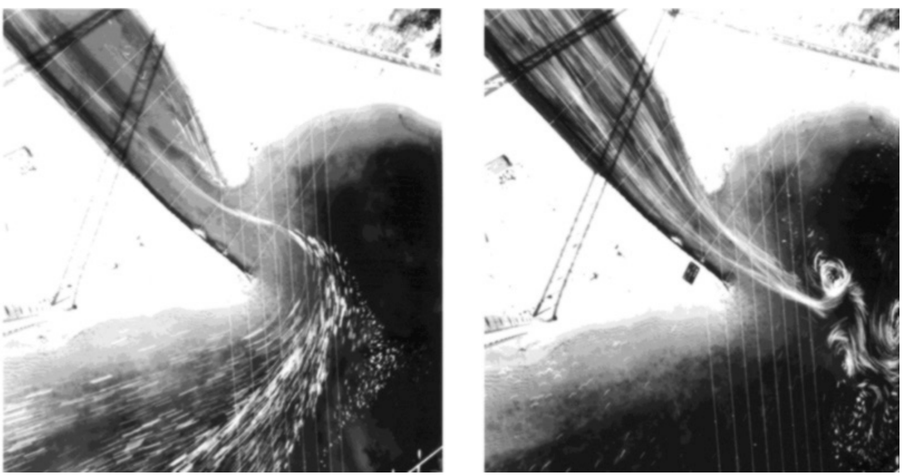


Figure 4-6 Flow pattern at the port of Abidjan

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 it's head. Dredging is difficult in such cases because of the proximity of the breakwater. An example of a breakwater that is too short is the breakwater of Paradip (India), shown in Figure 4-8.

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.

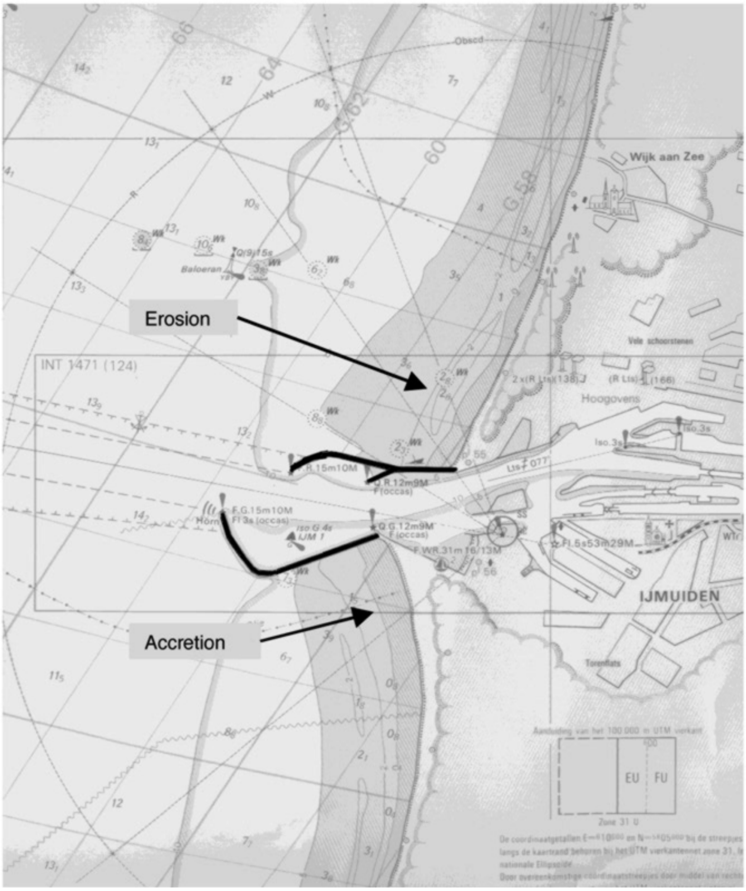


Figure 4-7 Port and breakwaters at IJmuiden

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:

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

probabilistic design (see Chapter 15)): These deficiencies may lead to unforeseen problems in operation of the port, especially when the breakwater also acts as quay wall.

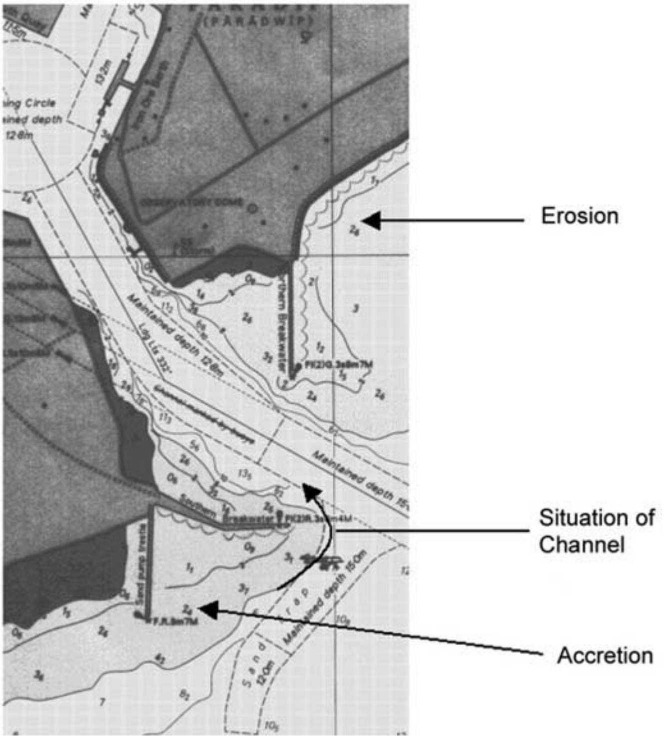


Figure 4-8 Siltation at entrance to port of Paradip

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 ignore the consequences of this phenomenon in a larger space and time frame. Accretion and erosion of the coastal zone on either side of the breakwater will most likely pose a serious threat to the community in the region and possibly to the ecosystem as well. It goes without saying that such consequences have to be assessed and quantified, and that remedial measures have to be designed, planned, and executed. In this respect, one may think of:

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

4.4 Functions of closure dams and side effects

A number of purposes and side effects are listed below. Side effects may be negative or positive. 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. This is especially he case with positive side effects.

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. Before that time, the Rhine emptied into the North Sea near Katwijk and, choked by sediment, regularly inundated the coastal area behind the dunes. In order to prevent this flooding, a closure dam was constructed 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. This indeed prevented all flooding both near Katwijk as well as near Utrecht. The flow was diverted via the Lek river-branch. Of course, this dam had unforeseen side effects. It cut off the downstream area from siltation and the outer delta at the Katwijk river mouth 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.

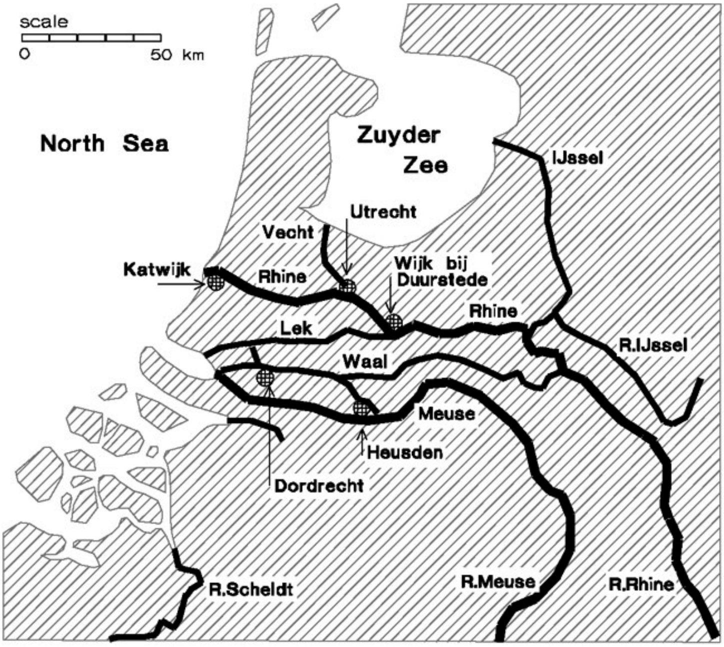
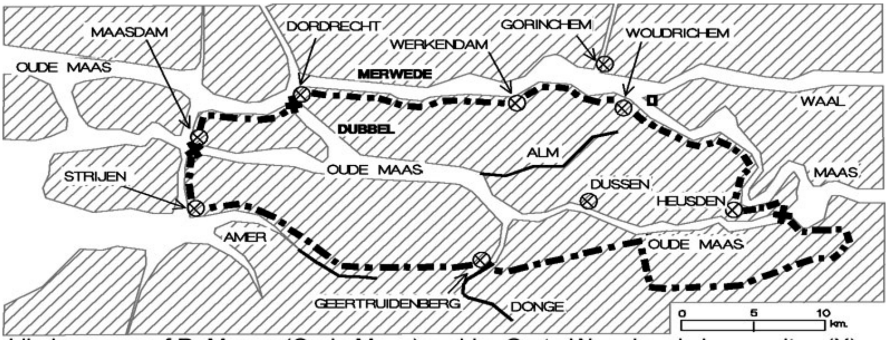


Figure 4-9 The Rhine Meuse-delta before the year 1000

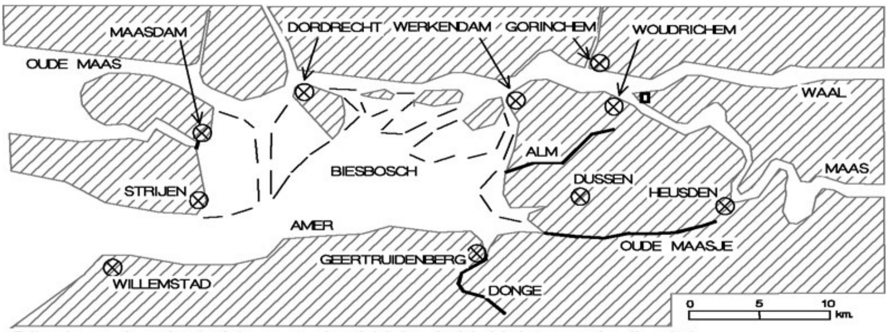
The damming of the river Meuse (Maas) followed a different scheme. The town of Dordrecht had obtained staple rights (the right to store and sell certain goods) along the river Merwede. However, the payment of toll dues could be avoided by sailing along the River Meuse (see Figure 4-10, Oude Maas).

Most likely 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.



Likely course of R. Meuse (Oude Maas), polder Grote Waard and closure sites (X)

Figure 4-10 Situation after damming the River Meuse



Dike breaches in 1421 created a 200 km2 tidal lake, gradually sitting up

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 condense. This changed the morphology of the landscape and its flora and fauna. What started as a simple water-level control system, 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 Dutch 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 has had an opportunity to show what would have happened otherwise. Natural restorative processes are 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.

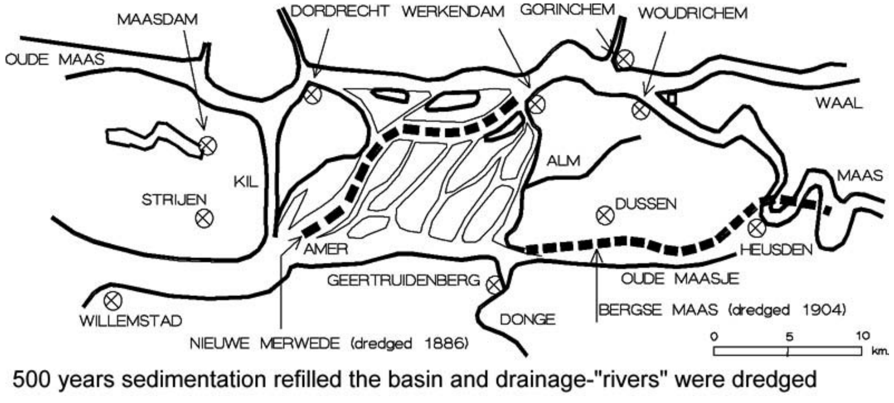


Figure 4-12 The Biesbosch area

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 Waddenzee. Due to the shallow depths of the Waddenzee, the amplitude of the tide gradually increased to more than twice the former tide with the progress of the closure. This effect was studied before the works started. However there was a more difficult question to be answered: 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 historical closures but is given because of their relevance to this book. Focus is on closures around the North Sea and in Korea.

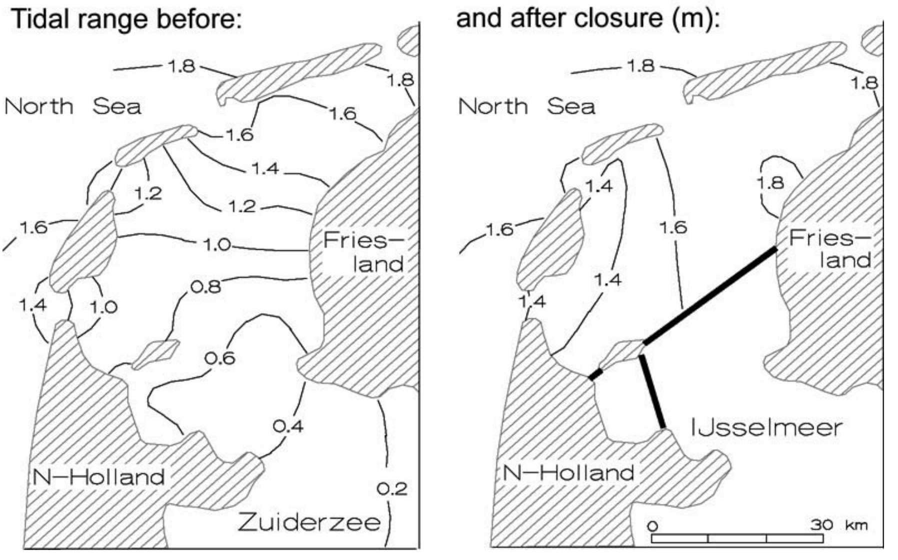


Figure 4-13 The Enclosure Dike and the tidal range

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	Oudenhoorn	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 (E = Emergency closure)