Breakwaters with Vertical and Inclined Concrete Walls

Report of Working Group 28 of the Maritime Navigation Commission

Burcharth, Hans Falk

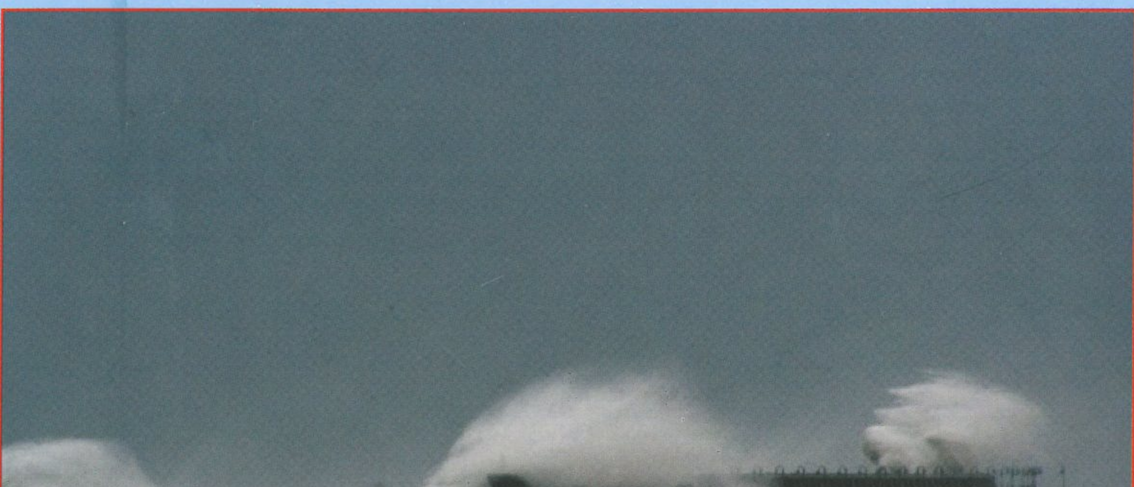
Publication date:
2003

Document Version
Publisher's PDF, also known as Version of record

INTERNATIONAL NAVIGATION ASSOCIATION

ASSOCIATION INTERNATIONALE DE NAVIGATION

Breakwaters with Vertical and Inclined Concrete Walls



MarCom
Report of WG 28
2003

INTERNATIONAL NAVIGATION ASSOCIATION

INTERNATIONAL NAVIGATION ASSOCIATION

BREAKWATERS WITH VERTICAL AND INCLINED CONCRETE WALLS

Report of Working Group 28
of the
MARITIME NAVIGATION COMMISSION

INTERNATIONAL NAVIGATION
ASSOCIATION

ASSOCIATION INTERNATIONALE
DE NAVIGATION

2003

PIANC has Technical Commissions concerned with inland waterways and ports (InCom), coastal and ocean waterways (including ports and harbours) (MarCom), environmental aspects (EnviCom) and sport and pleasure navigation (RecCom).

This Report has been produced by an international Working Group convened by the Maritime Navigation Commission (MarCom). Members of the Working Group represent several countries and are acknowledged experts in their profession.

The objective of this report is to provide information and recommendations on good practice. Conformity is not obligatory and engineering judgement should be used in its application, especially in special circumstances. This report should be seen as an expert guidance and state of the art on this particular subject. PIANC disclaims all responsibility in case this report should be presented as an official standard.

PIANC General Secretariat
Graaf de Ferraris-gebouw – 11th floor
Boulevard du Roi Albert II 20, B.3
B-1000 Brussels
BELGIUM
http://www.pianc-aipcn.org
VAT/TVA BE 408-287-945

ISBN 2-87223-139-0

Chapter 1

INTRODUCTION

1.1 Working group 28

Following the PIANC PTC II working group on Analyses of Rubble Mound Breakwaters it was, in 1991, decided to form Working Group (WG) n° 28 on "Breakwaters with vertical and inclined concrete walls".

The scope of the work was to achieve a better understanding of the overall safety aspects in the design of this important class of breakwater.

The chairmanship of Prof. H.F. Burcharth was confirmed in 1991, and all members of the WG were appointed in September 1992. Due to the foreseen start, by January 1993, of the three years duration European Community MAST II research project "Monolithic Coastal Structures" (MCS), it was proposed to run the WG 28 parallel to the MCS project in order to be able to include the relevant findings of this project in the WG 28 work. This coordination was accepted by PIANC and turned out to be very fruitful and easy as some members participated in the MCS project.

The start-up meeting was held February 1993 at University of Hannover. Further meetings were held October 1993 at CEDEX in Madrid, April 1994 at Port Authority of Genoa, April 1994 at Institution of Civil Engineers London, and February 1995 at the Technical University of Berlin. The final meeting was held in September 1996 at Delft University of Technology.

For various reasons the completion of this main report was delayed. This however, gave the opportunity to make use of the results of the European Union PROVERBS project and other recent results. Final editing of the main report was done in a meeting of the subgroup leaders in Delft, August, 2001.

The outcome of the work is the present main report, which summarises the contents of four subgroup reports, which can be purchased from PIANC.

The working group comprised the following members:

Chairman
Prof. Dr. Techn. H.F. Burcharth,
Aalborg University, Denmark

- Mr. J. Juhl, Danish Hydraulic Institute (DHI), Denmark

- Mr. O.J. Jensen, COWIconsult,
Lyngby, Denmark, substitute

Dr. J. W. van der Meer, Infram, The Netherlands

Prof. H. Ligteringen, Delft University of Technology and Royal Haskoning, Rotterdam, The Netherlands

Mr. A. G. H. Lejeune, University of Liege, Belgium

Mr. F. Ropert, Service Technique Central des Ports Maritimes et des Voies Navigables, Compiègne, France

Mr. M. Canel, SOGREAH Ingénierie, Grenoble, France

Mr. J.L.Diaz Rato, Gijon Port Authority, Spain

Mr. B. G. Madrigal, CEDEX, CEPYC, Madrid, Spain

Prof. Dr.-Ing. T. Stückrath,
Technische Universität Berlin, Germany

Prof. Dr.-Ing. H. Oumeraci,
Technische Universität Braunschweig, Germany

Prof. Dr.-Eng. Leopoldo Franco,
Politecnico di Milano, Italy

Mr. E. Brizzolara, Ing. G. Brizzolara & C., Genoa, Italy

Mr. J. Clifford, Fairwinds, Oxon, U.K.

Prof. N.W.H. Allsop, corresponding member
HR Wallingford, UK

Prof. Dr. P.A. Hedar, Gothenburg, Sweden

Mr. B. N. Sharp, corresponding member
Halcrow Consulting Engineers, London, U.K.

Dr. R.W. Whalin, WES, Vicksburg, Mississippi, USA

Mr. Y. Morin, Public Works of Canada, Ottawa, Ontario, Canada

Prof. Dr. K. Tanimoto, Saitama University, Japan

Mr. A. van Tonder, corresponding member CSIR, South Africa

Mr. A. Graauw, SOGREAH Ingénierie, Grenoble, France

Mr. M. Fedolino †, Genoa Port Authority, Italy

Dr. Osamu Kiyomiya, Wasea University, Japan.

A significant contribution to the work was given by Aalborg University, Denmark, on the probabilistic calculations, in particular by the following individuals:

Prof. dr. J. Dalsgaard Sørensen, Dr. Zhou Liu and Dr. Jan Pedersen.

The work of the group has depended upon help given by many individuals and organisations. Many of the individual helpers are listed as members of the subgroups. The many organisations that have provided data on various important structures, are evident. In our meetings in various countries, we received help and hospitality from the National Sections of PIANC as well as from the host organisations indicated in the list of meetings. To all these, the gratitude of the members of the Working Group is extended.

1.2 Terms of reference and organisation of work

The basis for the working group investigations was the PTC II (MarCom) formulated draft terms of reference (TOR):

a. Give an overview of the different types of concrete breakwaters with vertical or inclined walls, with or without wave absorbing structures.

b. Give an overview of accidents that took place with concrete breakwaters since about 1965.

c. Select for each type of structure one or more examples for which a significant store of data may be available.

d. Prepare for each such structure a synopsis of the variable inherent in the design, the construction, the environmental conditions, particularly wave impacts and subsoil, and changes that took place. The synopsis may be in tabular or matrix form, so that comparisons between different structures can be facilitated with regard to design, construction and performance. Give an overview for each breakwater of the wave climate considerations and criteria to determine the worse conditions and of the most employed calculations for the main parts of the breakwaters.

e. Prepare an analysis, seeking from the facts and recorded behaviour common factors that relate to cause and effect suitability of the breakwaters. In short, the analysis should correlate the characteristics with performance history, so as to identify those common factors in design, construction, maintenance or environment that have resulted in satisfactory breakwaters in some cases, and similarly to identify other factors common to breakwaters that have experienced greater, or even excessive, damage.

f. Give an overview of the most probable hypotheses for the origin of the accidents described.

g. Evaluate the safety (risk of failure) and propose ways of dealing with the safety problems in particular ways. This should include a check of the proposed safety guidelines against the behaviour of selected existing breakwater structures.

h. Pay special attention to the possibility of multipurpose use of different types of concrete breakwaters.

i. Give in the conclusions of the report recommendations for further investigations such as model tests or measurements in nature and indicate which questions in detail should be answered by that research. Give an estimate of the cost of such model tests or measurements. If special model or prototype measurements are undertaken, it is required that they are made without cost to PIANC.

The TOR was in principle accepted by the WG 28 and the tasks were divided into four areas of investigation to be handled by subgroups with the following terms of reference:

Subgroup A

(chairmanship H.F. Burcharth, J.W. van der Meer) Identification and evaluation of design tools (assessment of environmental loads, failure mode formulae, methods of stability calculations, empirical design rules, model testing, computational methods)

Subgroup B

(chairmanship H. Ligteringen, K. Tanimoto and L. Franco) Investigation of the design and performance of selected existing structures (items b, c, d, e and f in draft TOR)

Subgroup C

(chairman T. Stückrath) Investigations of the implication of construction aspects in the design. Performance of concretes. Identification of "hot spots" in the design and construction

Subgroup D

(chairman H.F. Burcharth) Implementation of safety in the design (item g in draft TOR)

The above given TOR for the subgroups was accepted by PTC II (MarCom) as the final basis for the work.

1.3 Summary of work

- Five types of basic structure types have been defined as well as some modified types. Besides this, some new concepts have been identified. The characteristics and the area of application of the structure types have been described

- Failure modes for the conventional structures are identified and classified into global (overall stability) and local (structural member strength) failure modes. Level of service in terms of hydraulic response (overtopping, wave transmission and wave reflection) is discussed

- Formulae for wave load estimation are discussed as well as the Japanese method of implementation of seismic loading. Ice loads are discussed briefly. Methods of calculation of earth pressures from fill as presented in various standards are described. Base plate friction coefficients based on experiments are presented. Design tools in terms of design equations and related conventional design methods are given for each global failure mode. Equations for the estimation of hydraulic responses are discussed

- The performance of conventional structures is discussed on the basis of selected examples of non-damaged and damaged structures. The failure probability of these examples have been evaluated based on the use of conventional design methods

- The influence of some construction aspects (float-out, first grounding, joints/settlements) on the design is discussed. Durability and specification of concrete are discussed, and the production of concrete blocks for blockwork breakwaters

- Methods to implement safety calculations in the design process by the use of safety factors and partial coefficients are presented and discussed. Sets of partial coefficients for global failure modes for caisson structures are developed

- Research recommendations are given concerning:
  - Introduction in codes and design recommendations of safety classes and acceptable safety levels for breakwaters
  - design procedures including the use of partial coefficients in order to assure target safety levels
  - acceptable levels of overtopping
  - slip failure calculations
  - structural, material and construction aspects influencing the safety and long-term performance of the structure.

Chapter 2

TYPES OF STRUCTURES

2.1 Conventional type

The basic structural element is usually a sand-filled caisson made of reinforced concrete, but also blockwork types made of stacked precast concrete blocks are used. Caisson breakwaters can be divided into the following types:

Conventional, i.e. the caisson is placed on a relatively thin stone bedding layer, Fig. 2-1.

Type 1: Conventional Caisson breakwater with vertical front.

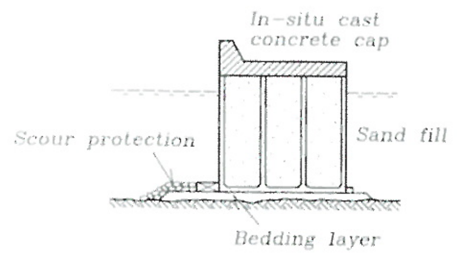


Figure 2-1: Example of conventional caisson breakwater with vertical front.

Vertical composite, i.e. the caisson is placed on a high rubble mound foundation. This type is economic in deep waters. In-situ cast caps are generally placed on shore-connected caissons, Fig. 2-2.

Type 2: Vertical composite caisson breakwater

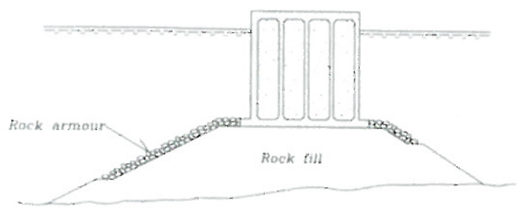


Figure 2-2: Example of vertical composite caisson breakwater.

Horizontal composite, i.e. the front of the caisson is covered by armour units. This type is used only in shallow water. The effects of the mound are reduction of wave reflection, wave impact and wave overtopping, Fig. 2-3.

Type 3: Horizontal composite caisson breakwater.

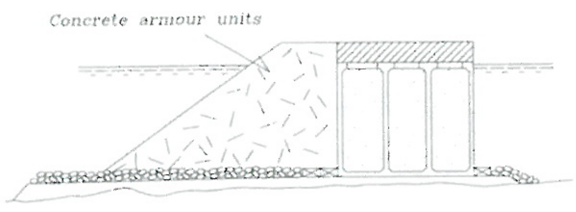


Figure 2-3: Example of horizontal composite breakwater.

Caisson breakwaters are generally less economical than rubble mound structures in the case of shallow water. Moreover, they demand stronger sea bed soils than rubble structures. Especially the "blockwork type" usually needs to be placed on rock sea beds or on very strong soils due to very high foundation loads and sensitivity to differential settlements, Fig. 2-4.

Type 4: Block work breakwater

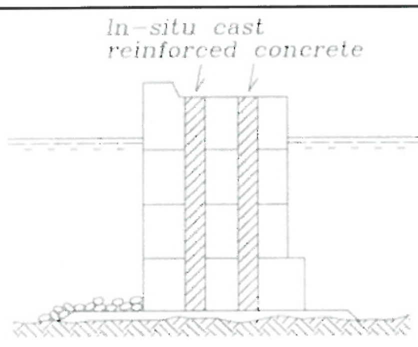


Figure 2-4: Example of block work caisson breakwater.

Piled breakwaters. Piled breakwaters consist of an inclined or vertical curtain wall mounted on pile work. The type is applicable in less severe wave climates on sites with weak and soft subsoils, Fig. 2-5.

Type 5: Piled breakwater

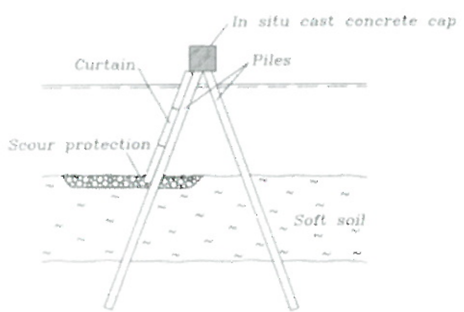


Figure 2-5: Example of piled breakwater.

Sloping top, i.e. the upper part of the front wall above still water level is given a slope with the effect of a reduction of the wave forces and a much more favourable direction of the wave forces on the sloping front. However, the overtopping is larger than for a vertical wall of equal crest level, Fig. 2-6.

Modified type: Sloping-top caisson breakwater

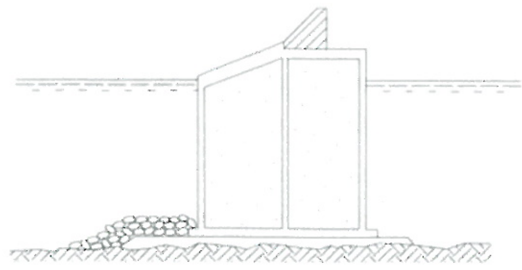


Figure 2-6: Example of sloping-top caisson breakwater.

Perforated front wall, i.e. the front wall is perforated by holes or slots with a wave chamber behind. Due to the dissipation of energy both the wave forces on the caisson and the wave reflection are reduced, Fig. 2-7.

Modified type: Perforated front wall caisson breakwater

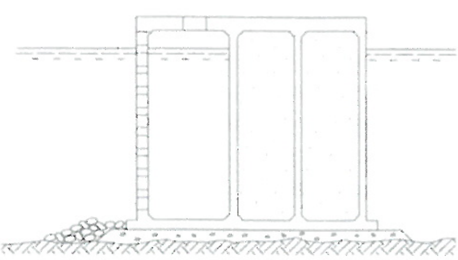


Figure 2-7: Example of perforated front wall caisson breakwater.

2.2 New concepts

There are various new concepts of vertical breakwater cross-sections. Here are introduced two types, which have been applied for practice.

A *semi-circular caisson* is well suited for shallow water situations with intensive wave breaking. Fig. 2-8 gives the cross section design at Miyazaki Port, Japan, while Fig. 2-9 shows the semi-circular caisson manufactured in a caisson yard at Miyazaki Port.

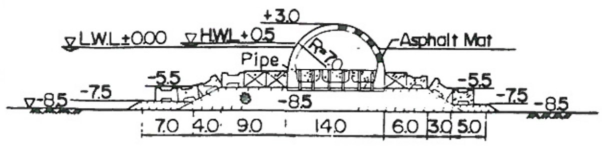


Figure 2-8: Cross-section of semi-circular caisson breakwater at Miyazaki Port, Japan (taken from Tanimoto et al. 1994).

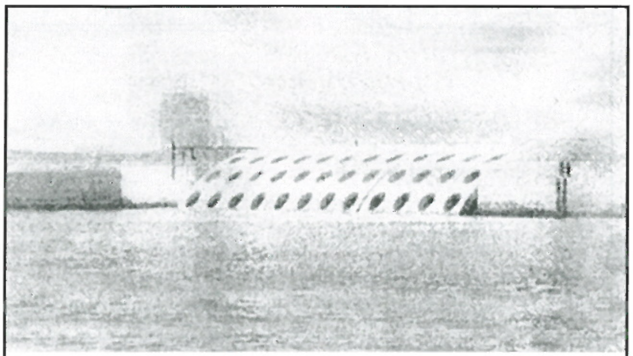


Figure 2-9: A semi-circular caisson, manufactured at a caisson yard, Miyazaki Port, Japan (taken from Tanimoto et al. 1994).

A dual cylindrical caisson is formed of an outer permeable cylinder and an inner impermeable cylinder. It is a low reflective and low permeable structure. Fig. 2.10 shows the 180m breakwater formed of dual cylindrical caisson in a marine recreational area of Nagashima Port with a water depth of 11m. The centre chamber and the lower ring chamber are filled with sand.

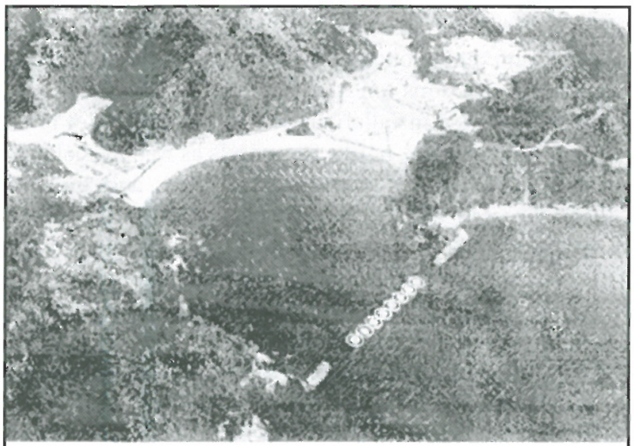


Figure 2-10: Dual cylindrical caisson breakwater at Nagashima Port, Japan (taken from Tanimoto et al. 1994).

Chapter 3

OVERVIEW OF FAILURE MODES

3.1 Terms of failure

For many people, the word "*failure*" implies a total or partial collapse of a structure, but this definition is limited and not accurate when discussing design and performance of coastal structures. In the context of design reliability, it is preferable to define failure as:

> **FAILURE:** Damage that results in performance and functionality of the structure below the minimum anticipated by design.

For example, subsidence of a breakwater protecting a harbour would be considered a failure if it results in wave heights within the harbour that exceed operational criteria.

Partial collapse of a structure may be classified as "*damage*" provided the structure still serves its original purpose at or above the minimum expected level.

Failure of coastal project elements arises from one or more of the following reasons:

- *Design failure* occurs when either the structure as a whole, including its foundation, or individual structure components cannot withstand load conditions within the design criteria. Design failure also occurs if the structure does not perform as anticipated and if design criteria are inappropriate
- *Load exceedence failure* occurs because anticipated design load conditions were exceeded
- *Construction failure* arises due to incorrect or bad construction or construction materials
- *Deterioration failure* is the result of structure deterioration and lack of project maintenance.

New or innovative coastal project design concepts are more susceptible to design failure due to lack of previous experience with similar designs. In these situations, allowances should be made for unknown design effects, and critical project elements should be extensively tested using laboratory and/or numerical model techniques before finalizing the design. Practically all projects accept some level of failure probability associated with exceedence of design load conditions, but failure probability increases at project sites where little prototype data exist on which to base the design. These cases may require a conservative factor of safety. For information on probabilistic design, see Chapter 7.

In the design process all possible failure modes given in the next section must be identified and evaluated in order to obtain a balanced design.

3.2 Types of failure modes

The failure modes can be classified as follows

Overall (global) stability failure modes of monoliths

- shoreward and seaward sliding
- foundation failure modes
  - slip surface failures
  - excess settlement
- overturning

Local stability failure modes

- hydraulic instability of rubble foundation
- hydraulic instability of rubble mound slope protection in front of caissons and breakage of blocks
- seabed scour in front of the structure
- breakage and displacement of structural elements

Local stability failure modes can trigger the overall stability failure modes.

The overall and the local stability failure modes are illustrated in Figs. 3-1 and 3-2, respectively.

Excessive wave overtopping and wave transmission are performance failures.

Failure mode limit state equations including validity ranges are given in the Subgroup A report.

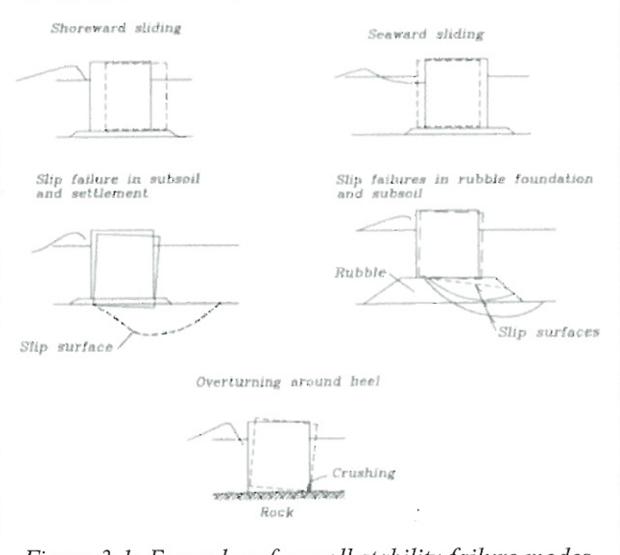


Figure 3-1: Examples of overall stability failure modes.

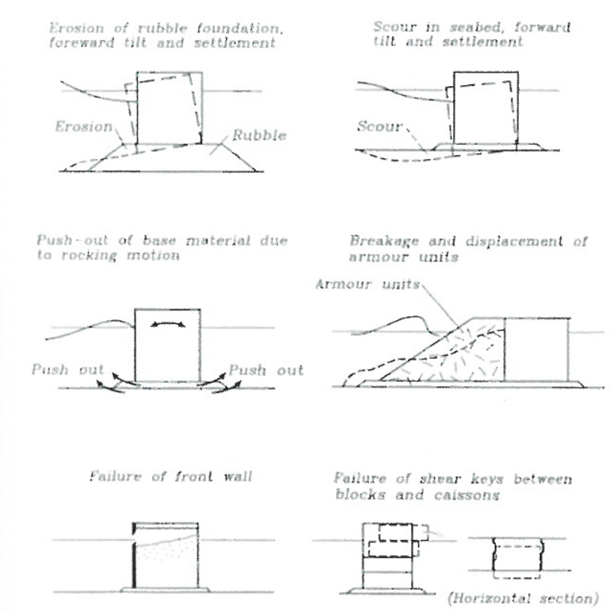


Figure 3-2: Examples of local stability failure modes.

Chapter 4

HYDRAULIC RESPONSES

4.1 Overtopping

Wave overtopping occurs when the highest run-up levels exceed the crest freeboard.

The amount of overtopping which can be accepted depends on the function of the structure. Certain functions impose restrictions on the overtopping discharge, e.g. access roads and installations placed on the crest of breakwaters and seawalls, berths for vessels as well as reclaimed areas containing roadways, storage areas and buildings located just behind the breakwater. Design criteria for overtopping should include two levels: Overtopping during normal service conditions and overtopping under extreme design conditions where some damage to permanent installations and structures might be considered. Very heavy overtopping might be allowed where a breakwater has no other functions than protection of harbour entrances and outer basins. However, significant overtopping can cause wave disturbance that can lead to damage of moored vessels. Fortunately, waves generated by overtopping usually have much shorter periods than the waves in the open sea. The overtopping discharge from wind-generated waves is very unevenly distributed in time and space as the amount varies considerably from wave to wave. The major part of the overtopping discharge during a storm is due to a small fraction of the waves.

In fact the local overtopping discharge (in m^3 per second wave per metre structure) from a single wave can be more than 100 times the time averaged overtopping discharge (in m^3 per second per metre structure) during the storm peak. Nevertheless, most information on overtopping is given as the time averaged overtopping discharge, per metre structure. However, some limited information exists on the probability distribution of the volume of overtopping water per wave.

Information from various studies is condensed in Fig. 4-1, which presents critical values of the average overtopping discharge q. The figures given in the table must be regarded only as rough guidelines because, even for the same value of q, the intensity of water hitting a specific location is very much dependent on the geometry of the structure and the distance from the front of the structure. The maximum intensities might locally be up to two orders of magnitude larger than q.

Some of the values given in Fig. 4-1 seem conservative. Research in this field within the EU-Fifth Framework project, CLASH is ongoing.

The wind can carry *spray* long distances whereas solid (green) water is practically unaffected by the wind. It is important to consider spray as it can cause damage to goods placed on storage areas and can cause over-icing of vessels in cold regions.

Formulae for average overtopping discharge and volume of overtopping individual waves are given in the Subgroup A report. Also the effect of wind on overtopping is treated there.

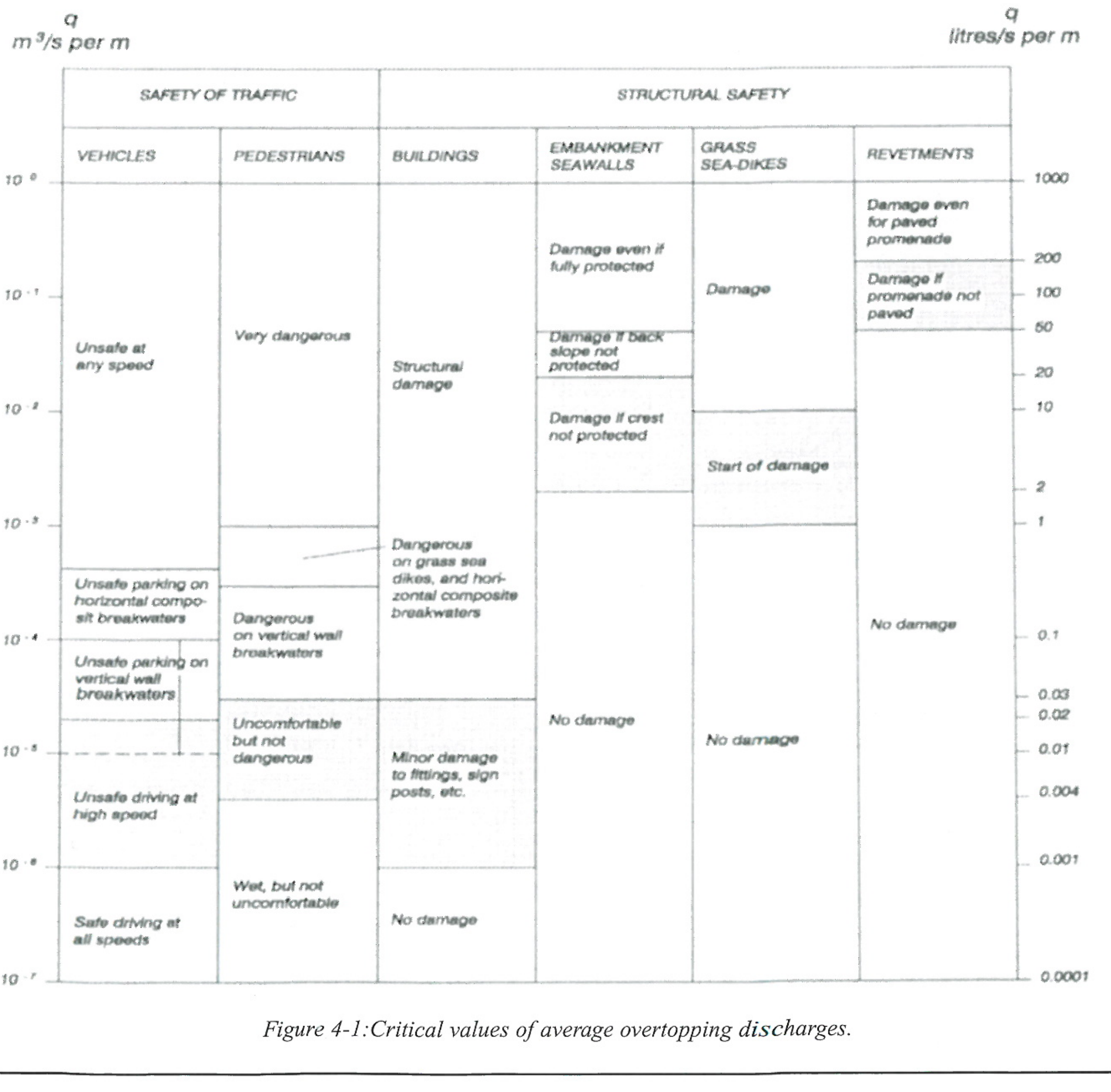


Figure 4-1:Critical values of average overtopping discharges.

In general the average overtopping discharge per unit length of the structure, q, is a function of the standard parameters:

q = function (H_s, T_{op}, σ, β, R_c, h, g, geometry of structure and near-structure sea bed)

where

H_s	= significant wave height
T_{op}	= wave periods
σ	= spreading of short crested waves
β	= angle of incidence for the waves
R_c	= freeboard
h_s	= water depth in front of structure
g	= the gravitational acceleration

Formulae for overtopping are empirical as they are fitted to hydraulic model test results for specific structure geometries. Two types of models for dimensionless formulae are dominating the literature

$Q = a \quad exp\ (-\ b\ R)$
$Q = a\ R^{-b}$

where Q is a dimensionless average discharge per metre and R is a dimensionless freeboard. The fitted coefficients a and b are specific to the front geometry of the structure.

Fig. 4-3 shows examples of model test overtopping data.

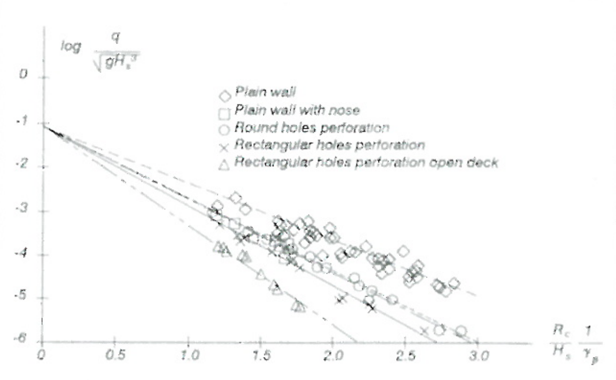


Fig. 4-3: Wave overtopping data. (Franco and Franco, 1998).

4.2 Wave reflection

Breakwaters reflect some proportion of the incident wave energy. If significant, the interaction of incident and reflected waves can create a very confused sea with very steep and often breaking waves. It is a well-known problem in many harbour entrance areas where it can cause considerable manoeuvring problems for smaller vessels. A strong reflection also increases the risk of sea bed erosion potential in front of the structure. Moreover, waves reflected from breakwaters can, in some cases, create or increase erosion of neighbouring beaches.

Non-overtopped impermeable smooth vertical walls will reflect almost all the incident wave energy while a permeable mild slope rubble mound will absorb a significant portion of the energy and is therefore well suited as a wave absorber, for example in harbour basins. The wave reflection of vertical wall breakwater can be reduced by introducing a perforated front or a sloping top, cf. Figs. 2-6 and 2-7.

In general, the energy of incident waves can be partly dissipated by wave breaking, surface resistance and porous flow, partly transmitted into harbour basins due to wave overtopping and penetration, and partly reflected back to the sea, i.e.

$E = E_d + E_t + E_r$

where E, E_d, E_t and E_r are incident, dissipated, transmitted and reflected energy, respectively.

The reflection can be quantified by the reflection coefficient

$C_r = H_{sr} / H_s = (E_r / E)^{0.5}$

where H_s and H_{sr} are the significant wave heights of the incident and reflected waves, respectively. E and E_r are the related energies.

Examples of reflection coefficients for a plain impermeable and a perforated vertical caisson breakwater are shown in Fig. 4-4.

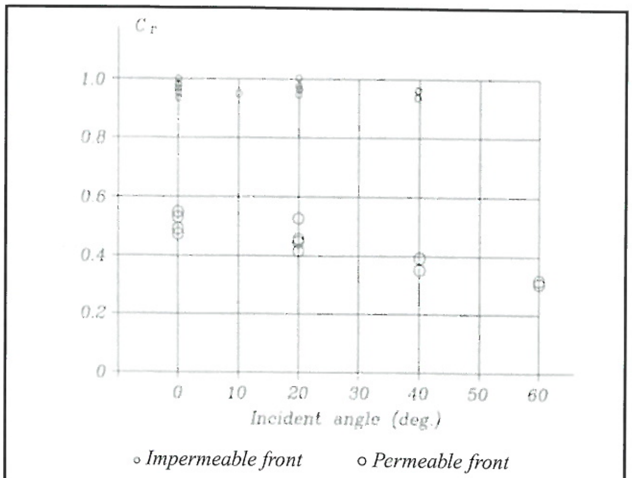


Fig. 4-4. Wave reflection from a plain impermeable and a perforated vertical caisson breakwater exposed to short crested waves (Helm-Petersen, 1998).

Reflection coefficients for types 1, 2, 3 and 4 and the modified types of breakwaters shown in Chapter 2 are given in the Subgroup A report.

4.3 Wave Transmission

Waves behind a structure can be caused by overtopping, and for permeable structures also by wave penetration. Waves generated by the falling water from overtopping tend to have shorter periods than the incident waves. Generally the periods of the transmitted waves are in the order of 0.5 of the incident waves, which means that energy is shifted to higher frequencies causing a change in the shape of the spectra.

Wave transmission can be characterized by a transmission coefficient, C_t, defined either as the ratio between transmitted and incident characteristic wave heights (e.g. H_{st} and H_s) or as the square root of the ratio between transmitted and incident time averaged wave energy (e.g. E_t and E),

$$C_t = H_{st} / H_s = (E_t / E)^{0.5}$$

Specific transmission coefficients for wave overtopping (subindex *o*) and wave penetration through the structure (subindex *p*) can be defined as follows

$$C_{tp} = H_{st}^{penet.} / H_s$$

$$C_{tp} = H_{st}^{overtop} / H_s$$

In practice it is often difficult to distinguish between $H_{st}^{overtop}$ and $H_{st}^{penet.}$ and consequently only C_t is calculated.

Values of C_t given in the literature are almost all from laboratory experiments, many of these performed to rather small scales. Significant scale effects might be present especially for the proportion of C_t stemming from wave penetration.

Wave transmission for vertical breakwaters is usely introduced by wave overtopping. Therefore the ratios of the breakwater crest height R_c to the incident wave height H_s is the most important parameter.

Fig. 4-5 shows typical relations between wave transmission coefficient and dimensionless freeboard for a plain vertical wall breakwater exposed to head-on regular waves. The most relevant range corresponds to $0.5 < R_c / H < 1.5$.

Wave transmission coefficients for the breakwater types 1, 2, 3 (cf. Figs. 2-1, 2-2, 2-3) and the modified types (cf. Figs. 2-6 and 2-7) are given in the Subgroup A report.

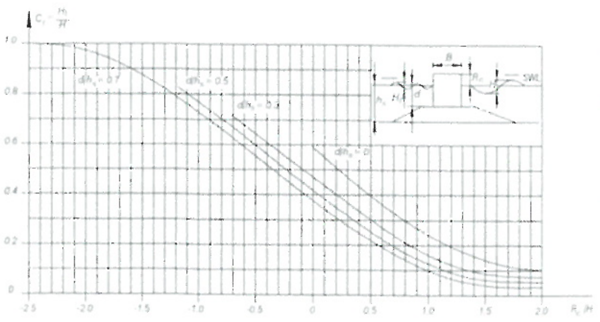


Fig. 4-5. Wave transmission coefficients for a plain vertical wall breakwater exposed to head-on regular waves. From Goda, 1969.

Chapter 5

WAVE FORCES

5.1 Introduction

Wave generated pressures are highly complicated functions of the wave conditions and the geometry of the structure. It is therefore recommended to carry out model tests, at least for the final design of important structures. For preliminary or conceptual designs the formulae presented in the Subgroup A report can be used within the stated limitations, together with consideration of the uncertainties.

Three different types of wave forces on vertical walls can be identified, as shown in Fig. 5-1.

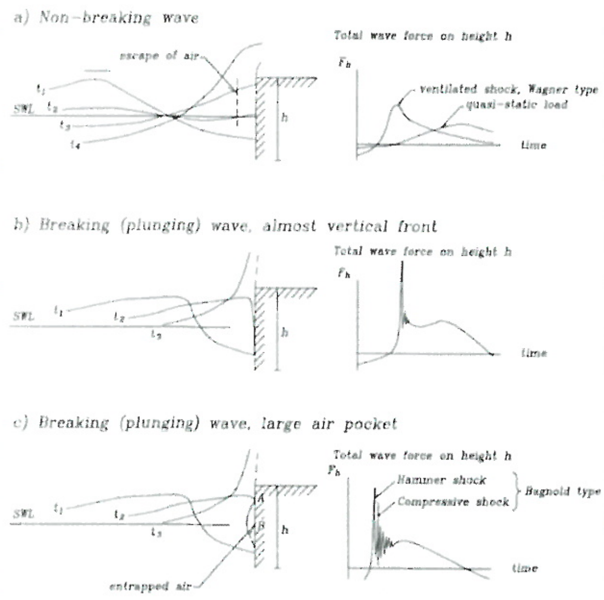


Figure 5-1: Illustration of vertical wall wave forces from non-breaking and breaking waves (Burcharth 1993).

a) Non-breaking waves: No air pocket will be entrapped on the wall. The pressure at the wall will have a relatively gentle variation in time and be almost in phase with the wave elevation. This kind of wave load might be called a *pulsating* or *quasistatic load* because the period is much larger than the natural period of oscillation of the structure including conventional caisson breakwaters (app. one order of magnitude larger). Consequently, the wave load can be treated like a static load in stability calculations, with the exception that special considerations are required if the caisson is placed on fine soils where pore pressure built up resulting in significant weakening of the soil might take place.

b) Breaking (plunging) waves with almost vertical fronts: Breaking waves of the plunging type develop an almost vertical front before they curl over. If this almost vertical front happens to appear just prior to the contact with the wall then very high but extremely short duration pressures occur. Only a negligible amount of air is entrapped, resulting in a very large *single peaked* force followed by very small force oscillations. The duration of the pressure peak is in the order of hundredths of a second. The shorter the duration, the larger the peak pressure for constant momentum.

c) Breaking (plunging) waves with large air pockets: A larger amount of air is entrapped in a pocket, resulting in a *double peaked* force followed by pronounced force oscillations. The first and largest peak is induced by the wave crest hitting the structure at point A, and is denoted a *hammer shock*. The second peak is induced by the subsequent maximum compression of the air pocket, B, and is denoted *compression shock*, (Lundgren, 1969). In the literature such wave loading is often called the Bagnold type. The force oscillations are due to the pulsation of the air pocket. The double peaks are typically spaced in the range of some hundredths of a second. The period of the force oscillations for large air pockets is in the range of tenths of a second or larger (Oumeraci et al. 1992).

In the literature, all wave generated pressures with fast rising time are generally called *impact pressures or impulsive pressures*.

Due to the extremely stochastic nature of impacts, there is no reliable formula for prediction of impulsive pressures caused by breaking waves. Determination of impact pressures from model tests is difficult because of scale effects related to the amount and size of air bubbles and size and shape of air pockets. Also the instrumentation, the data sampling and the analyses need special consideration in order to avoid bias by dynamic amplification and misinterpretations at prototype scales. Another problem related to model tests is the sensitivity of the shock loads to the shape and kinematics of the breaking waves. This calls for a very realistic and statistically correct reproduction of natural waves in models.

Impulsive loads from breaking waves are very large. Moreover, as they are of stochastic nature the extreme values increase with the number of loads. Further, impulsive loads might cause dynamic amplification of the caisson movements, cf. the discussion in the Subgroup A report. For this reason the design of the breakwater cross- section and/or the orientation of the breakwater relative to the wave direction should be such that frequent wave breaking at the structure is avoided and, if not possible, a rubble mound structure might be chosen. Alternatively, a mound of armour units might be placed in front of the vertical wall structure.

Impulsive wave loads as illustrated in Fig. 5-1, b, might be parameterised as shown in Fig. 5-2.

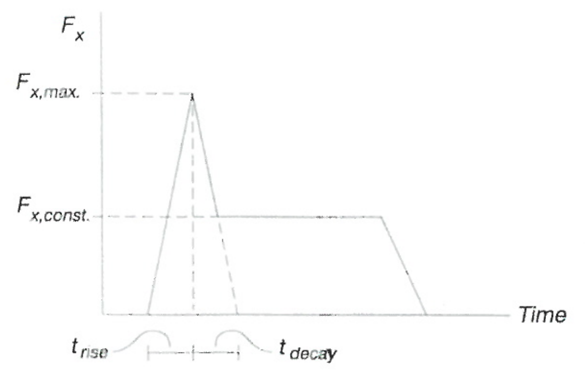
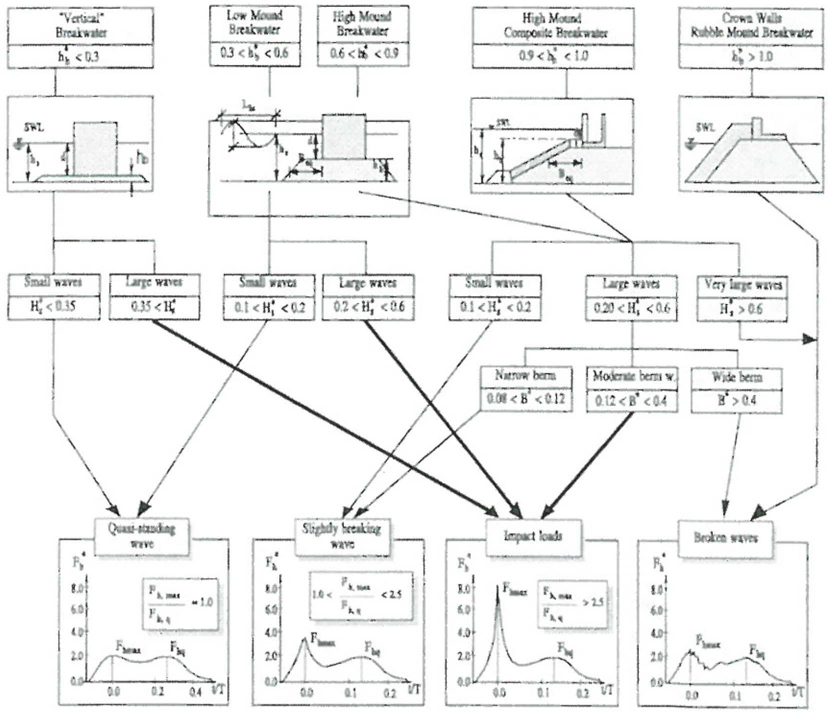


Fig. 5-2. Example of parameterization of Impulsive wave load.

Impulsive wave loads are further discussed in the Subgroup A report and PROVERBS (2001).

Fig. 5-3 shows a system for identification of types of total horizontal wave loading on the vertical front as a function of structure geometry and wave characteristics (Kortenhaus and Oumeraci 1998). The system is based on 2-dimensional model tests with irregular head-on waves. It should be noted that conditions for 3-dimensional and oblique waves are different. Note that the diagram does not cover situations where wave breaking takes place in a wider zone in front of the structure, i.e. typical shallow water situations with depth limited waves and sea beds flatter than, say, 1:50.

Also note that the slope of the seabed near the structure, which can influence the wave loading, is not included in the diagram.



with $h_b^* = \frac{h_b}{h_s}$; $H_s^* = \frac{H_{si}}{h_s}$; $B^* = \frac{B_{eq}}{L_{hs}}$; $F_h^* = \frac{F_h}{\rho \cdot g \cdot H_b^2}$

H_{si}	Incident significant wave height	h_b	Height of rubble mound
H_b	Breaking wave height	B_{eq}	Length starting from $h_b/2$ to caisson foot
L_{hs}	Wave length at water depth hs	F_h	Horizontal wave force
g	Gravitational acceleration		

Figure 5-3: Identification of types of total horizontal wave loading on vertical wall structure exposed to head-on long-crested irregular waves (Kortenhaus and Oumeraci 1998 and PROVERBS 2001). Not valid if a wider breaker zone is present in front of the structure. Influence of sea bed slope in front of the structure not included.

Frequent wave breaking at the structure causing very large impulsive forces will not take place in oblique waves with angle of incidence larger than 20°. Nor will it take place if the sea bed in front of the structure has a mild slope, say 1:50 or less, over a distance of at least some wave lengths, and no sloping foundation at the toe of the wall is present.

The use of a sloping top from about SWL to the crest is very effective in almost eliminating large impact pressures from breaking waves. Moreover, the direction of the wave forces on the sloping part (right angle to the surface) is very favourable in reducing the horizontal force and the tilting moment. Sloping top structures might be difficult to optimize where large water level variations are present. Moreover, for equal crest heights a sloping front structure allows more overtopping than a vertical wall structure.

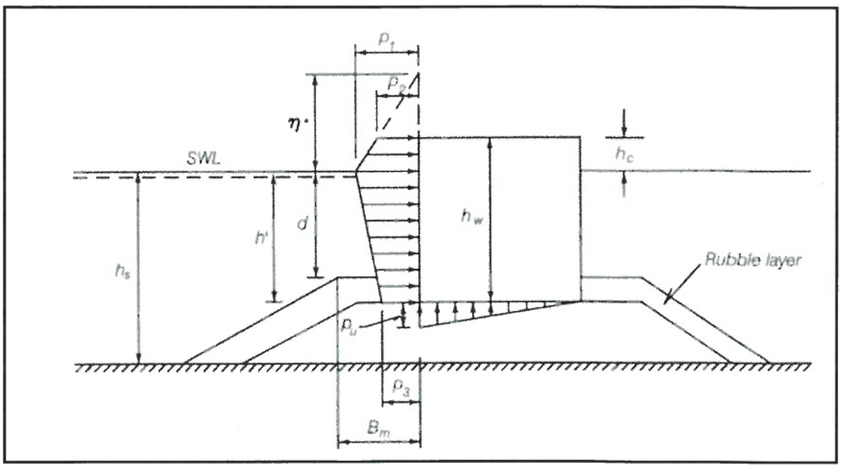
It is important to note that a semi-high slope rubble mound (e.g. a rubble protection or foundation) in front of a vertical wall *should be designed with care* because it might trigger wave breaking and thereby frequent impact loads on the wall. In Goda (2000) further advice is given on how to prevent large impulsive loadings.

5.2 Practical formulae for 2-D wave force on vertical walls

For such condition Goda (1974) presented a practical design formula for 2-D wave forces on plain, impermeable vertical walls, see Box 5-1. The formula is based on small scale model tests and experience from performance of Japanese prototype caisson breakwaters. As the formula is adjusted for design it includes some conservatism (positive bias), cf. chapter 7.2 and the Subgroup A report.

The Goda formula has been expanded by Tanimoto et al. (1976) and Takahashi et al. (1994) and others to include impermeable inclined walls and sloping top structures as well as horizontally composite structures and vertical slit walls. These formulae are given in the Subgroup A report together with the Sainflou formula for loading from standing waves.

Box 5-1. Goda formula for irregular waves (Goda 1974, Tanimoto et al. 1976).



$$\eta^* = 0.75\,(1 + \cos\beta)\,\lambda_1\, H_{design}$$

$$p_1 = 0.5\,(1 + \cos\beta)(\lambda_1\,\alpha_1 + \lambda_2\,\alpha_*\,\cos^2\beta)\rho_w\, g\, H_{design}$$

$$p_2 = \begin{cases} (1 - \dfrac{h_c}{\eta^*})p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \le h_c \end{cases}$$

$$P_3 = \alpha_3\, p_1$$

$$P_u = 0.5\,(1 + \cos\beta)\lambda_3\,\alpha_1\,\alpha_3\,\rho_w\, g\, H_{design}$$

where

β angle of incidence of waves (angle between wave crest and front of structure)

H_{design} design wave height defined as the highest wave in the design sea state at a location just in front of the breakwater. If seaward of a surf zone Goda (2000) recommends for practical design a value of 1.8 H_s to be used corresponding to the 0.15 % exceedence value for Rayleigh distributed wave heights. This corresponds to $H_{1/250}$ (mean of the heights of the waves included in 1/250 of the total number of waves, counted in descending order of height from the highest wave). Goda's recommendation includes a safety factor in terms of positive bias as discussed in the Subgroup report A. If within a surf zone, H_{design} is taken as the highest of the random breaking waves at a distance $5H_s$ seaward of the structure.

$$\alpha_* = \alpha_2$$

$$\alpha_1 = 0.6 + 0.5\left[\frac{4\pi h_s / L}{\sinh 4\pi h_s / L}\right]^2$$

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2 \text{and } \frac{2d}{H_{design}}$$

$$\alpha_3 = 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{\cosh(2\pi h_s / L)}\right]$$

L wave length at the water depth h_b, corresponding to that of the significant wave T_s = app. 1.1 T_m, where T_m is the average period.

h_b water depth at a distance of $5H_s$ seaward of the breakwater front wall.

λ_1, λ_2, and λ_3 modification factors depending on the structure type. For conventional vertical wall structures $\lambda_1 = \lambda_2 = \lambda_3 = 1$.

5.3 Effect of structure length and alignment

Diffraction at the head of the structure creates variations in wave heights along the structure, see the Subgroup A report.

The wave loading is never equally distributed along the front of the breakwater. Consequently, the instantaneous average load depends on the length of the structure. This plays a role when designing long caissons and keys between caissons. The angle of incidence of the waves not only affects the pressures (see Box 5-1) but also affects the lateral load distribution. This is illustrated in Fig. 5-4, which shows the peak-delay force reduction for fully reflected oblique non-breaking regular waves, based on Battjes (1982).

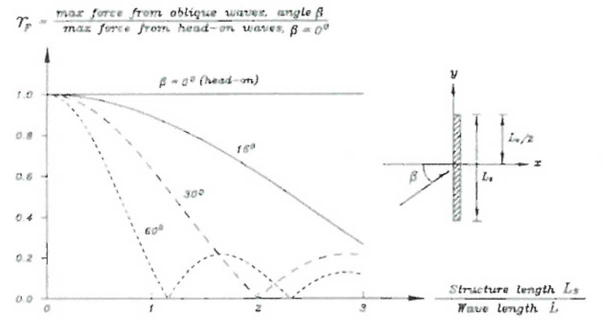


Figure 5-4: Peak-delay force reduction for fully reflected non-breaking oblique waves.

Burcharth and Liu (1998) presented formulae for peak-delay force reduction for oblique non-breaking irregular and short-crested waves. They also presented a formula for the horizontal turning moment for non-breaking regular waves. The formulae and related diagrams are given in the Subgroup A report.

Short-crested waves break in a limited area and not simultaneously along the whole caisson. This results in an even larger force reduction in comparison with non-breaking waves. Fig. 5-5 shows an example of force reduction from model tests with short-crested breaking head-on waves, where the force reduction r_F is defined as

$$r_F = \frac{F_{1/250},\ \textit{short crested wave, mean wave incident angle}\ \theta_m}{F_{1/250},\ \textit{long - crested head - on wave}}$$

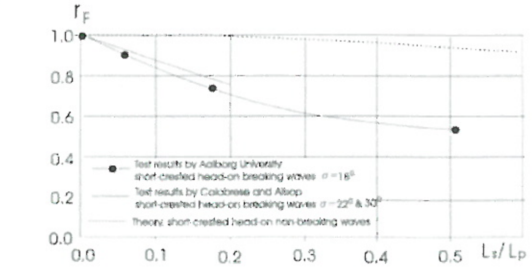


Fig. 5-5. Example of force reduction from model tests with short-crested breaking waves (Burcharth and Liu, 1998).

Chapter 6

OTHER FORCES

This chapter gives an overview of other forces than wave forces, treated in the Subgroup C report.

6.1 Earthquake

Although advances in computer techniques enable dynamic response to be analysed by finite element methods, the simple equivalent static load method is generally acceptable for breakwater structures. In many countries, and for the obvious example of Japan, the horizontal earthquake load is still calculated by multiplying the vertical dead load and surcharge by a seismic coefficient determined from a number of factors. Reference should also be made to the book produced by the PIANC Marcom Working Group 34, PIANC 2001.

6.2 Ice Pressure

Load from ice pressure on a vertical breakwater seldom exceeds the wave load. The effective pressure from ice loading decreases with structure size and there are, at present, no conclusive formulae that can be applied to large works. Therefore, in those countries where ice loading is a consideration, ice pressures are derived from local experience and judgement.

6.3 Earth Pressures for Structural Design

Earth pressure is relevant to vertical breakwaters with rubble or fill placed against them, and to the load from retained materials within caissons.

Traditional "working stress" codes recommend "active" or "at-rest" pressure coefficients to be applied to the dry or submerged soil mass, appropriate to different forms of construction. Different approaches are taken in different countries. Therefore, traditional methods still remain as an option in most codes.

New structural analysis codes and geotechnical codes now adopt limit state philosophy. Structural analysis to limit state codes requires the application of partial factors for loading cases and materials for the calculation of the ultimate and the serviceability limit state conditions.

There are two distinctive methods of applying limit state methods and partial factors to the structural design of earth retaining structures. One method derives directly from structural design. It applies the partial factors from

Eurocode 2 or similar national codes to the characteristic or serviceability limit state loading. The other method derives from geotechnical stability analysis. It applies a partial factor (or, in the case of BS 8002, a "mobilisation" factor) to a parameter, such as tan Φ.

A comparison of various national applications of partial factor methods for the calculation of structural members is given in the Subgroup C report and is illustrated by an example. The example demonstrates the range of results for calculation of the load on one side of a member in 20m depth of fill of some 1.5 to 1.

The range of factors lies between the application of the partial factors in the structural codes (i.e. 1.4 or 1.35 on dead load and 1.6 or 1.5 on live load) to the unfactored soil properties, and the less conservative loading from new USA, Japanese and older Scandinavian codes and BS 8002 and the draft Eurocode 7 depending upon interpretation (where the factor is of the order of 1.2).

6.4 Fill Pressures within Caissons

The loading within caissons is generally derived from silo theory. An example of how fill pressures calculated to various national standards compares with the "at-rest" unconfined pressure is also illustrated in the Subgroup C report. The silo pressure of submerged sand is seen to range from 30 % to 60 % of the unconfined "at-rest" pressure.

6.5 Friction

There is a surprising divergence in the various national codes between the figures used in design for friction and for a factor of safety against sliding. The coefficient of friction varies between 0.5 and 1.0 (for different cases) and the factor of safety between 1.0 to 1.75.

6.6 Handling and Float-Out Loads

Loads which can arise during construction, although transient, can be significant and must be considered carefully. The forces arising from towing can be taken from Japanese standards.

6.7 First Grounding

Severe loading cases can arise when a lowered caisson first makes contact with the prepared foundation. In most cases the caissons will never again undergo a comparable distribution of load. These dynamic loads cannot be predicted precisely, but the designer can influence and reduce the risk of indeterminate load imposition by various means, including downstand legs that predetermine the location of first grounding, see Figure 6-1.

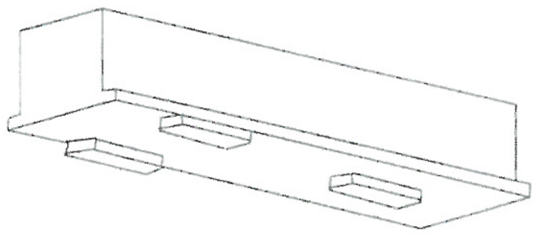


Figure 6-1: Legs, which provide a first landing on three points.

Chapter 7

DESIGN METHODS

7.1 Present design practice

Current design practice related to *overall stability of monolithic caissons* makes use of empirical formulae and model tests to prepare different conceptual designs to be compared and costed. When based on national standards or recommendations in which overall safety factors are given, the term *deterministic* design is used.

This is to distinguish from *probabilistic* design procedures in which the uncertainties on load and resistance parameters as well as on the design formulae and methods of calculation are taken into account.

The safety of a deterministic design based on overall safety factors is unknown in terms of probability of damage or failure within the lifetime of the structure.

However, the safety of a deterministic design can be estimated by the use of so called level II and level III probabilistic methods. It necessitates that the abovementioned uncertainties are estimated.

The outcome will be a certain probability of damage/failure. If larger or smaller than anticipated, the design must be changed and the procedure repeated. This would be characterized as a probabilistic design procedure.

In order to shorten this procedure and to ensure a certain minimum safety, various national codes and the Eurocode for civil engineering structures introduce so called *partial safety factors* to be applied to load and resistance parameters in the design formulae. These parameters are calibrated such that the outcome will be a structure with a safety level corresponding to experienced and accepted good long-term performance of the specific type of structure, for example for conventional buildings. The actual safety level of a structure will not be known when applying such partial safety factor systems.

Eurocode does not yet include partial safety factors suitable for coastal structure design.

Compared to the number of most conventional civil engineering structures, there are few breakwaters built and there are no generally accepted or defined safety levels. This was the reason for the development of a new partial safety factor system for rubble mound breakwaters in the PIANC PTC II (MarCom) Working Group 12. The characteristic of this method is that partial coefficients are given for various safety levels. This means that the designer can decide on a target safety level for the breakwater and subsequent, by applying the related partial safety factors, obtain a design with the target safety.

This system termed the PIANC p*artial safety factor system* has been expanded by Working Group 28 to include vertical wall breakwaters.

An overall presentation of the PIANC safety factor system for both rubble mound and vertical wall breakwaters is given in Burcharth and Sørensen (2000). This includes tabulated safety factors for the main failure modes.

The target safety levels can be chosen within the range corresponding to performance of existing breakwaters. However, it is proposed that safety classes and acceptable safety levels are defined and included in standards and recommendations. This work, which must be based also on economic optimization calculations for typical breakwaters, still remains to be done.

The presented target safety partial safety factor system has been used in the EU-MAST III PROVERBS project. This project included research on probabilistic economic optimization.

As to the design practice related to *structural analysis of concrete caissons,* see Chapter 10.4.1.

7.2 Partial safety factor system

Limit state equations

Evaluation of structural safety is always related to the structural response as defined by failure modes, cf. Figs. 3-1 and 3-2. Each failure mode response must be described by a formula or a set of equations.

As an illustration example of the concept of using partial safety factors, the limit state equation for horizontal sliding of a caisson on a rubble foundation is considered.

$$(F_G - F_U)\, f - F_H \begin{cases} \geq 0, \text{ no sliding} \\ < 0, \text{ sliding} \end{cases}$$

where

F_G = Buoyancy reduced weight of the caisson
F_U = Wave induced uplift force
F_H = Wave induced horizontal force
f = Friction coefficient for base plate on rubble stone foundation

The limit state equations including their validity ranges for all the considered failure modes are given in the Subgroup A report.

Definition of partial safety factors

The variables in the limit state equations are either load variables, X_i^{load} as for example F_H and F_U, or resistance variables, X_i^{res} as for example F_G and f.

Because the variables are uncertain parameters we apply a partial safety factor, γ_i, to characteristic values of each of them, or, if sufficient, to some of them to obtain the design values:

$$X_i^{design} = \gamma_i^{load} \cdot X_{i,ch}^{load}$$

$$X_i^{design} = \frac{X_{i,ch}^{res}}{\gamma_i^{res}}$$

The partial safety factors, γ_i, which are larger or equal to one, are uniquely related to the definition of the characteristic values of the uncertain parameters. In conventional civil engineering codes, the characteristic values of material strength parameters are taken as the lower 5 % fractile, while for load parameters characteristic values corresponding to the upper 5 % fractile are often used. Other definitions may be used, as is the case in the PIANC WG 12 and the present system. The magnitude of γ_i reflects both the target safety level, the uncertainty on the related parameter X_i, and the relative importance of X_i in the failure mode equation.

Design equations

When the partial safety factors and the characteristic values of the parameters are applied in the limit state failure mode equation, we obtain a *design equation* which in its general formulation for a sliding failure reads:

$$G = \left(\frac{F_G^{ch}}{\gamma_{FG}} - \gamma_{FU} \cdot F_U^{ch} \right) \frac{f^{ch}}{\gamma_f} - \gamma_{FH}\, F_H^{ch} \geq 0$$

This equation, which contains four partial safety factors, has been simplified as follows:

The wave load is in the present partial safety factor system assumed either calculated from the Goda formula compensated for bias, or determined from physical model tests.

In case no model tests are performed, F_H and F_U are recommended values calculated from the Goda formula, Box 5-1, applying a design wave height corresponding to the highest wave in the design sea state at a location just in front of the breakwater. In the present system it has been chosen to use the structure lifetime (T_L) return period sea state characterized by the central estimate significant wave height, i.e. $\hat{H}_S^{T}L$.

γ_{F_U} and γ_{F_H} are then substituted by one partial safety factor γ_H to be applied to $\hat{H}_S^{T}L$ in the calculation of the wave induced forces. γ_H accounts both for the uncertainty on H_S and the uncertainty of the Goda formula.

Because Goda's formula includes positive bias (as a practical design formula) some bias factors U_V and U_H must be applied to F_U and F_H respectively.

Moreover, because the uncertainty on F_G is small compared to other involved uncertainties, and γ_{F_G} as a consequence is almost equal to one, this safety factor is omitted (set equal to one). As characteristic values are used the central estimate indicated by $\wedge$. The final *design equation* then reads

$$G = (\hat{F}_G - \hat{U}_{Ver.Force} \cdot \hat{F}_U)\frac{1}{\gamma_z}\hat{f} - \hat{U}_{Hor.Force} \cdot \hat{F}_H \geq 0$$

in which $\gamma_H \cdot \hat{H}_S^{T}L$ is used as significant wave height as the basis for estimation of the maximum wave height applied in the Goda formula for calculation of $\hat{F}_U$ and $\hat{F}_H$. γ_Z is an overall resistance parameter safety factor which includes the uncertainty on $\hat{f}$.

The following design equations are given in the Subgroup D report together with the related partial coefficients:

- Stability against sliding
- Stability against overturning
- Slip failure stability of rubble foundation on sand subsoil
- Slip failure stability of rubble foundation on clay subsoil
- Hydraulic stability of toe berm rock armour
- Wave induced scour in front of roundheads

For horizontal composite structures:

- Hydraulic stability of Dolosse
- Hydraulic stability of Tetrapods
- Breakage of Dolosse
- Breakage of Tetrapods

The first four design equations involve determination of the wave load as action parameter whereas the other six design equations apply the significant wave height as action parameter.

The following factors by van der Meer et al. (1994) are used to compensate for the positive bias inherent in the Goda formula:

$\hat{U}_{Hor.Force}$	= 0.90 : bias factor to be applied to the Goda horizontal wave force
$\hat{U}_{Ver.Force}$	= 0.77 : bias factor to be applied to the Goda vertical wave force
$\hat{U}_{Hor.Moment}$	= 0,81 : bias factor to be applied to the moment from the Goda horizontal wave forces around the shoreward heel of the base plate
$\hat{U}_{Ver.Moment}$	= 0.72 : bias factor to be applied to the moment from the Goda vertical wave forces around the shoreward heel of the base plate

No bias factors are applied to central estimates of wave loads from model tests.

Determination and format of the partial safety factors

The concept of the partial safety factor system is in principle similar to the PIANC WG 12 system valid for rubble mound breakwaters, in that both systems are calibrated to obtain a target safety level of the breakwater.

The overall procedure in development of the system comprised the following steps:

- Identification of the failure modes and related equations, which give the relationship between the wave impact and the response of the structure
- Selection of the format of the partial safety factor systems. The format defines the number and type of partial safety factors and the way they are applied to the failure mode equations to obtain the design equations
- Specification of the statistical properties and the relevant range of the uncertain parameters in the design equations
- Selection of a number of typical types of structures

- Design for each type a large number of structures (failure elements) using a First Order Reliability Method (FORM)
- Select and optimize (calibrate) on this basis the partial safety factors corresponding to selected failure probabilities
- Verify the accuracy of the calibrated partial safety factors by calculation of deviations from target reliability levels
- Verify the partial safety factor system against the behavior of existing structures.

The chosen system includes the following safety factors:

- The load partial safety factor on permanent loads is set equal to one
- A load partial safety factor γ_H to be multiplied to $\hat{H}_S^{T}L$ (the central estimate of the significant wave height which in average is exceeded once every T_L years)
- A resistance partial safety factor γ_Z to be used with resistance parameters as shown in the design equations. γ_Z is divided into tangent to the mean value of the friction angle in failure modes involving friction materials like quarry rubble mounds and subsoil
- A resistance partial safety factor γ_C to be divided into the undrained shear strength of subsoil clay materials.

The partial safety factors for the vertical wall breakwaters are presented in tables with the following entrees reflecting both the target safety level and the character and quality of the available wave and wave load information:

- Design structure lifetime T_L = (20, 50 or 100 years)
- Acceptable probability of failure P_f (= 0.01, 0.05, 0.10, 0.20 or 0.40)
- Quality of wave data given by a coefficient of variation, σ'_{FH_S}, on the H_S – source data values used in the wave climate statistics. $\sigma'_{FH_S} = 0.05$ corresponding to small uncertainty (typically advanced hindcast model values), or 0.20 corresponding to large uncertainty (typically fetch diagram values)
- Deep or shallow water conditions. In the latter case the waves are depth limited which implies less increase in design wave height with increasing return period
- Wave loads determined by model tests or not. In case of model tests, the uncertainty on the wave loads is reduced.

The calibration and the subsequent verification of the partial safety factors are described in the Subgroup D report.

Uncertainties covered by the partial safety factors

- Model uncertainties related to mathematical formulae (e.g. for calculation of wave loads, bearing capacity of soils, and hydraulic stability of armour blocks) are covered. The uncertainties are evaluated on the basis of statistical analyses of experimental data or expert opinions
- The statistical uncertainty of a specific type of extreme distribution (e.g. a Weibull distribution) fitted to long-term significant wave height data is covered
- Non-biased errors related to wave height recordings and imperfect hindcast methods for estimation of significant wave heights are covered by the coefficient of variation, σ'_{FH_S}, which is one of the entrees in the safety factor tables. Estimated values of σ_{FH_S} are given in Table 7-1.
- Uncertainty related to material parameters (e.g. friction coefficients and densities) and geometrical parameters are covered.

Uncertainties not covered by the partial safety factors

- Uncertainty due to lack of knowledge about the true long-term extreme distribution for significant wave heights is not covered. The designer must try to fit different theoretical distributions to the data and select the most appropriate on the basis of best fit (focus on the tail) and maybe some conservatism. Note that it is assumed that the sample data (H_S-values) represents the statistical population to which H_S belongs. Consequently there are limits to the minimum length of the observation period and the minimum number of data
- Uncertainty due to climatologic changes is not covered. It is the designer's choice to correct for trends if identified in historical data
- Uncertainty due to imperfect physical modeling including scale effects, also related to transformation of data from model to prototype, is not covered in the case where wave loads are determined from model tests (instead of using the Goda formula which has been calibrated against prototype behavior of structures). For such case the designer must correct for possible errors in his interpretation of the model test results.

Uncertainty related to wave climate was extensively treated in the Subgroup B report of the PIANC MarCom WG 12 on rubble mound breakwaters. Burcharth (1992). Uncertainty related to environmental data and estimated extreme events.

Table 7-1. Typical variational coefficients $\sigma' = \sigma/\mu$(standard deviation over mean value) for measured and calculated wave heights (Burcharth, 1992)

Parameter	Methods of determination	Estimated typical values σ'	 Bias	Comments
Significant wave height, OFFSHORE	Accelerometer buoy, Pressure cell Vertical	0.05-0.1	~ 0	
	Horizontal radar	0.15	~ 0	
	Hindcast, num. Models	0.1-0.2	0-0.1	Very dependent on quality of weather maps.
	Hindcast, SMB method	0.15-0.2	?	Valid only for storm conditions in restricted sea basins.
	Visual observations from ships	0.2	0.05	
Significant wave height NEARSHORE determined from offshore significant wave height taking into account typical shallow water effects (refraction, diffraction, shoaling, …)	Numerical models	0.1-0.20	0.1	σ' can be much larger in some cases.
	Manual calculations	0.15-0.35		Partial safety factors

Partial safety factors

The following tables provide the partial safety factors for two important failure modes. The related design equations are also given. The equations and partial safety factors for other important failure modes, including foundation failures, are given in the Subgroup A report with Appendices A and B. The basis of the partial safety factors is explained in the Subgroup D report.

The application of partial safety factors is demonstrated in an example, presented in the Appendix to this Main Report. The width of a caisson is determined, using the partial safety factors for sliding and overturning as given in Table 7-2 and 7-3, and the probability of failure as proposed in Chapter 10. The result is compared with that of the conventional design method and the difference between the two methods is discussed.

Table 7-2. Partial safety factors for sliding failure of vertical wall caissons.

Design equatio

$$G = G(\gamma_H \hat{H}_S^{TL}, \hat{\rho}_c, \hat{U}_{Hor.Force}, \hat{U}_{Ver.Force}, \hat{\xi}, \frac{1}{\gamma_Z}\hat{f}, B)$$

$$= (\hat{F}_G - \hat{U}_{Ver.Force}\hat{F}_U)\frac{1}{\gamma_Z}\hat{f} - \hat{U}_{Hor.Force}\hat{F}_H$$

In calculation of $\hat{F}_U$ and $\hat{F}_H$ apply as wave height $\gamma_H \hat{H}_S^{TL}$.

Deep water. Design without model tests.

	$\sigma'_{F_{H_S}} = 0.05$		$\sigma'_{F_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.4	1.7	1.5	1.7
0.05	1.3	1.4	1.4	1.4
0.10	1.3	1.2	1.4	1.3
0.20	1.2	1.2	1.3	1.2
0.40	1.1	1.0	1.1	1.1

Deep water. Wave loads $\hat{F}_H$ and $\hat{F}_U$ determined by model tests. No bias factors applied.

	$\sigma'_{F_{H_S}} = 0.05$		$\sigma'_{F_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.3	1.5	1.4	1.5
0.05	1.2	1.4	1.3	1.4
0.10	1.2	1.2	1.3	1.2
0.20	1.1	1.2	1.2	1.2
0.40	1.0	1.2	1.1	1.0

Shallow water. Design without model tests.

	$\sigma'_{F_{H_S}} = 0.05$		$\sigma'_{F_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.3	1.9	1.4	1.9
0.05	1.2	1.6	1.3	1.6
0.10	1.2	1.4	1.3	1.4
0.20	1.1	1.3	1.2	1.3
0.40	1.0	1.2	1.0	1.2

Shallow water. Wave loads $\hat{F}_H$ and $\hat{F}_U$ determined by model tests. No bias factors applied.

	$\sigma'_{F_{H_S}} = 0.05$		$\sigma'_{F_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.2	1.6	1.3	1.6
0.05	1.1	1.5	1.2	1.5
0.10	1.1	1.3	1.2	1.3
0.20	1.1	1.2	1.1	1.2
0.40	1.0	1.1	1.0	1.1

$\hat{H}_S^{TL}$	significant wave height with return period TL
T_L	structure lifetime
B	width of caisson
$\hat{U}_{Hor.Force}$	0.90, bias factor to be applied to the Goda horizontal wave force
$\hat{U}_{Ver.Force}$	0.77, bias factor to be applied to the Goda vertical wave force
$\hat{U}_{Hor.Moment}$	0.81, bias factor to be applied to the moment from the Goda horizontal wave forces around the shoreward heel of the base plate
$\hat{U}_{Ver.Moment}$	0.72, bias factor to be applied to the moment from the Goda vertical wave forces around the shoreward heel of the base plate
$\hat{\rho}_c$	mass density of caisson
$\hat{F}_G$	buoyancy reduced weight of caisson
$\hat{F}_H$	horizontal wave force calculated by the Goda Formula
$\hat{F}_U$	wave induced uplift force calculated by the Goda formula
$\hat{f}$	friction coefficient
$\hat{\xi}$	tidal elevation

Table 7-3. Partial safety factors for overturning failure of vertical wall caissons.

Design equation

$$G = G(\gamma_H \hat{H}_S^{T_L}, \hat{\rho}_c, \hat{U}_{Hor.Moment}, \hat{U}_{Ver.Moment}, \hat{\xi}, B)$$

$$= (\hat{M}_G - \hat{U}_{Ver.Moment}\hat{M}_U) - \hat{U}_{Hor.Moment}\hat{M}_H$$

In calculation of $\hat{M}_U$ and $\hat{M}_H$ apply as wave height $\gamma_H \hat{H}_S^T L$.

Design without model tests.

	$\sigma'_{F_{H_S}} = 0.05$	$\sigma'_{F_{H_S}} = 0.2$
P_f	γ_H	γ_H
0.01	-	-
0.05	2.7	-
0.10	2.0	2.5
0.20	1.6	1.7
0.40	1.2	1.2

Wave load determined by model tests. No bias factors applied.

	$\sigma'_{F_{H_S}} = 0.05$	$\sigma'_{F_{H_S}} = 0.2$
P_f	γ_H	γ_H
0.01	2.1	2.3
0.05	1.7	1.9
0.10	1.4	1.6
0.20	1.3	1.4
0.40	1.1	1.2

$\hat{H}_S^{T_L}$	significant wave height with return period T_L
T_L	structure lifetime
B	width of caisson
$\hat{U}_{Hor.Force}$	0.90, bias factor to be applied to the Goda horizontal wave force
$\hat{U}_{Ver.Force}$	0.77, bias factor to be applied to the Goda vertical wave force
$\hat{U}_{Hor.Moment}$	0.81, bias factor to be applied to the moment from the Goda horizontal wave forces around the shoreward heel of the base plate
$\hat{U}_{Ver.Moment}$	0.72, bias factor to be applied to the moment from the Goda vertical wave forces around the shoreward heel of the base plate
$\hat{\rho}_c$	mass density of caisson
$\hat{M}_G$	moment of F_G around heel of caisson
$\hat{M}_H$	moment of F_H around heel of caisson
$\hat{M}_U$	moment of F_U around heel of caisson
F_G	buoyancy reduced weight of caisson
F_H	horizontal wave force calculated by the Goda Formula
F_U	Wave induced uplift force calculated by the Goda formula
-	indicates that a partial safety factor corresponding to the small P_f –values cannot be obtained due to the large inherent uncertainties
$\hat{\xi}$	tidal elevation

Chapter 8

PERFORMANCE OF EXISTING STRUCTURES

Several studies have been published in the past on failures of vertical breakwaters, presenting very interesting and useful information on the extent of the damage and providing indications as to the possible causes of failure (Lamberti et al. 1994, Tanimoto et al. 1994, Hitachi 1994 and Takahashi et al. 1994). Furthermore extensive studies were carried out in Japan on the failure of vertical breakwaters in the period between 1965 and 1982, which were published only in Japanese (Goda, 1973 and Hattori, 1984).

This chapter is based on the Working Group's Subgroup B Report which is written by Prof. Ligteringen.

8.1 Objectives and Methodology

The reason was to analyse the performance of existing structures within the framework of this Working Group and to provide a basis for the evaluation of existing design criteria and design methods, including safety levels, in line with the Working Group objectives.

The analysis has in most cases been done on a deterministic level, comparing the actual observed performance with the predicted behavior of the breakwater, based on various design formulae. For instance where sliding formed the dominant failure mechanism in most cases, the estimated horizontal resistance is compared with the calculated horizontal force (converted from wave height and period via wave pressures) and conclusions are drawn with respect to validity of the design formulae. It will be clear that the reliability of the wave conditions causing the damage is of primary importance to results obtained. For a number of cases a probabilistic analysis was carried out, allowing the uncertainties in the various parameters to be reflected in probabilities of failure.

8.2 Cases investigated

The search for relevant cases was carried out for East-Asia, Europe and North-America and resulted in a "long list" of 33 cases, covering all five types of vertical and inclined walls. The majority of these were of Type 1. For Type 2-5 some cases were identified. The subsequent collection and screening of the pertaining data resulted in the elimination of certain cases, mostly on grounds of incompleteness of the information. The final list of analysed cases is presented in Box 8-1, divided according to type.

8.3 Selected Case Summaries

For each type one case has been selected of which a summary is presented in the Main Report. For the full treatment of all cases reference is made to the Subgroup B Report.

(i)Type 1: Caisson on thin bedding layer

Sakata Detached Breakwater, Section II and III, Japan.

Original construction completed in 1973. Section II is located at a depth of CD -9.0m and consists of a concrete caisson on a 2.4m thick bedding layer. Section III is found at a depth of CD -8.5m with a caisson on a 1.9m thick bedding layer (See Fig. 8-1). Design wave conditions: H_s = 5.9m (depth limited), T = 10.5s, ß = 0°. Design water depth is 9.5 and 9.0m respectively.

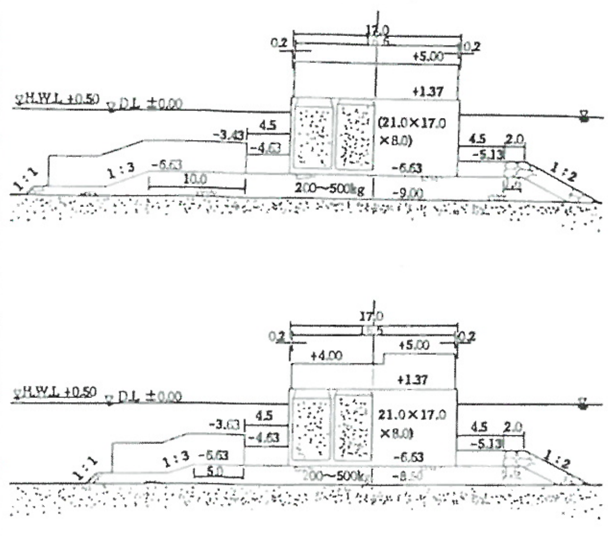
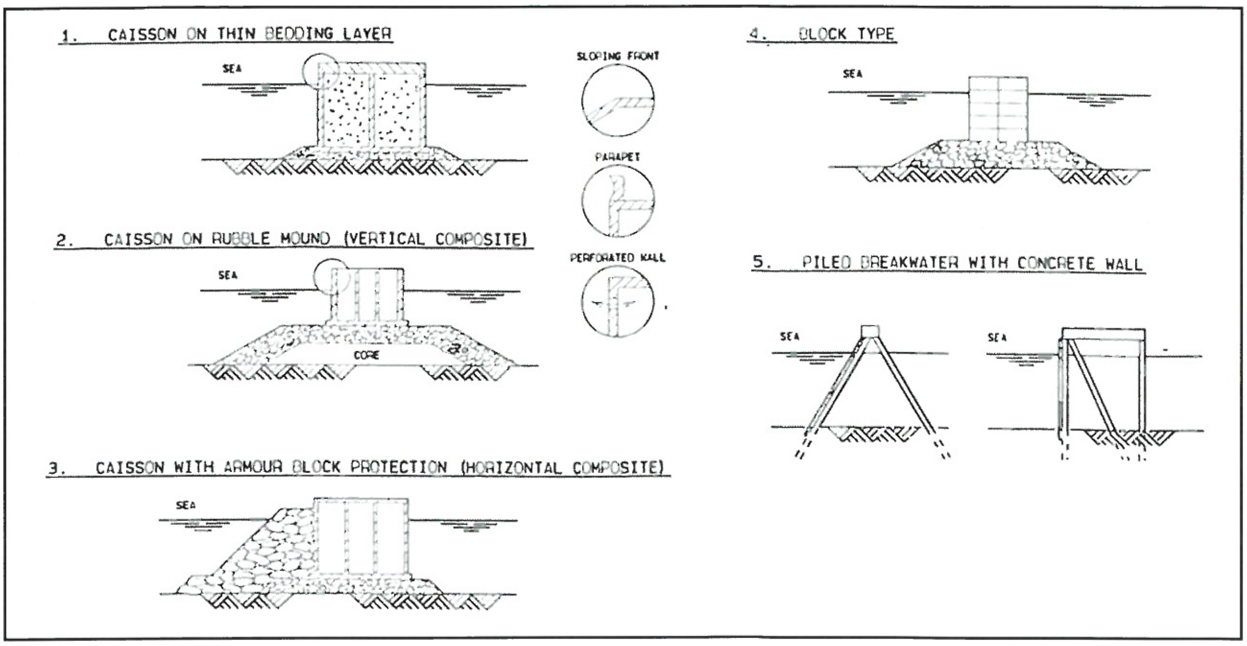


Figure 8-1: Cross Section of Section II (top) and Section III (bottom), Sakata Port, Japan (Takahashi et al., 1994).

During the winter 1973/1974 sliding of all 35 caissons occurred during consecutive storms with estimated significant wave heights at the breakwater location ranging from 4.2-6.1m and wave periods ranging from 8.8-13.8s.

Sliding distances of caissons are given in Fig. 8-3, which shows that the maximum sliding distance is about 3.8m.

Box 8-1. Types and cases investigated.



Type 1: Caissons on thin bedding layer
Detached Breakwater at Sakata Port (incl. Section: type 3)
East Breakwater Mutsu.Ogawara Port Section V (incl. Head)
West Breakwater at Niigata East Port
Second West Breakwater at Niiagata West-Port
Gela Offshore Breakwater
Roscoff-Bloscon Breakwater

Type 3: Horizontal composite breakwater
First central Breakwater at Hachinohe Port East Break-water at Mutsa-Ogawara Port (Concave Section)

Type 2: Caissons on rubble mound
South breakwater at Kashima Port (incl. Head: Type 3)
Offshore Breakwater of Napoli Harbour (incl. Sections: Type 4)
New caisson breakwater of Porto Torres Industrial Harbour
Algeciras Breakwater

Type 4: Block type breakwater
Santa Cruz de La Palma Breakwater
Los Gigantes Breakwater

Type 5: Piled breakwater
Manfredonia Breakwater

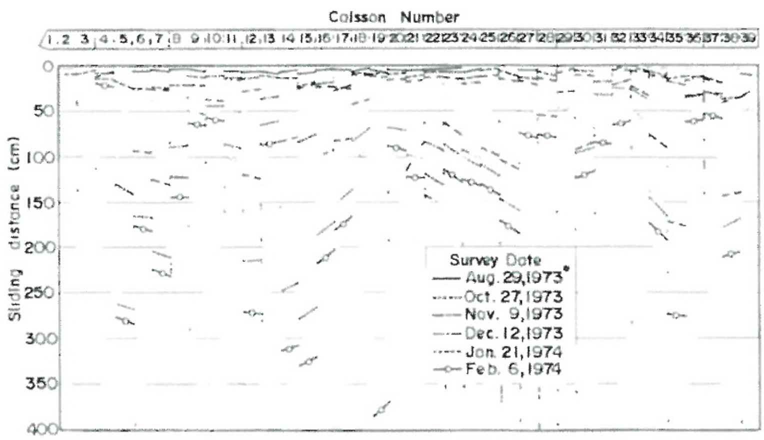


Figure 8-3: Sliding distance of caissons, Sakata Port, Japan (Takahashi et al. 1994).

After this winter the breakwater was covered to the seaward side with a pile of Tetrapods, i.e. horizontally composite breakwater, cf. Fig. 8-4. No damage has been reported since completion of this reinforcement.

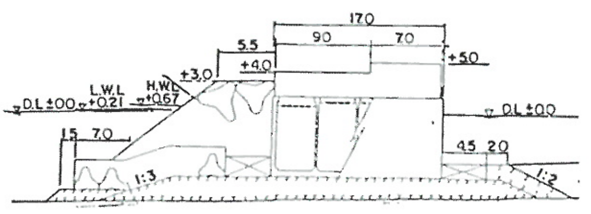


Figure 8-4: Cross-section after reparation, Sakata Port, Japan (Takahashi et al. 1994).

The analyses lead to the conclusion that sliding was caused by the occurrence of impulsive pressures due to breaking waves, which was not taken into account in the original design based on the old Japanese Technical Standards (Hiroi formula). Also the Goda formula underestimates the wave force. The Generalized Goda Formula (Goda, 2000) is a good tool to predict the impulsive pressures due to the berm configuration.

<u>(ii)Type 2: Caissons on Rubble Mound</u>

Offshore Breakwater Napoli Harbour, Italy

The breakwater at the port of Napoli is constructed in several parts, starting with a masonry blockwork section built before 1900, two sections with large concrete elements built in 1906 and between the world wars, Section D built of very small caissons in the fifties, and Section E consisting of larger caissons completed in 1982. The present analysis concerns Section D and E, which are shown in Fig. 8-5. Both are located in depths of 18-20m. Section D consists of a 13.5m high caisson on a 8-10m high mound. A high parapet wall (to about CD + 7.0m) is placed at the seaward side, a thin cap layer covers the caisson rear side. Section E consists of a 16m high caisson on a 5m high mound. Parapet wall and cap layer are more robust than in Section D.

The original design conditions and criteria are unknown. Analysis of the wave climate on the basis of 6 years wave measurements and refraction/shoaling computations results in a 50 year wave height at the breakwater location of H_s = 4.2m. Design water level during storms is assumed at CD + 0.6m.

At the peak of the storm on 11 January 1987 the last 5 caissons of Section D slid landward by 4 to 9m, tilting down the inner slope of the rubble foundation. The first two caissons of Section E adjacent to them at transitional foundation depth also slid, but for smaller distances (2.5m and 1.0m), cf. Fig. 8-6.

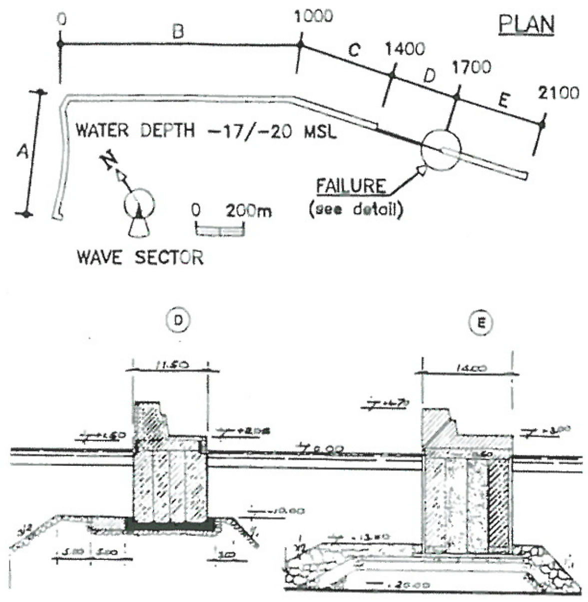


Figure 8-5: Lay-out, cross-sections of Section D and Section E, Offshore Breakwater, Napoli Harbour, Italy (Lamberti et al.1994).

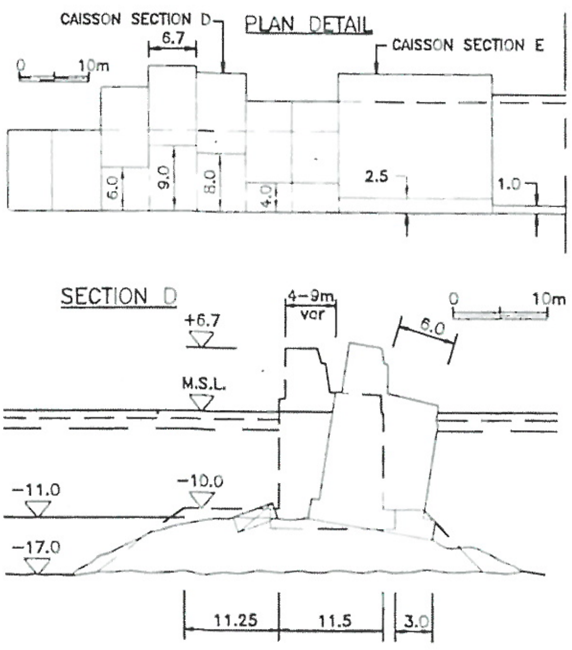


Figure 8-6: Damage of the caissons in Section D and Section E, Offshore Breakwater, Napoli Harbour, Italy (Lamberti et al.1994).

A detailed storm hind-casting was carried out, showing a peak significant wave height of $H_s = 5.14$m and $T_p = 13.8$s at the location of the breakwater and normal to it. The maximum wave height was estimated at $H_{max} = 9.35$m.

Deterministic analysis of the stability of the caissons, using Goda formula, leads to the conclusion that section D would slide at $H_{max}= 9.0$m, while Section E would reach its limiting resistance against sliding for $H_{max}=10.45$m. Uncertainties in the calculation of wave heights and their spatial distribution are believed to account for the discrepancy between predicted and actual behaviour of Section E.

(iii) Type 3: caissons with armour block protection

First Central Breakwater Hachinohe Port, Japan

The construction of this breakwater started in 1984 and a length of 1600m was completed by 1991. According to the design 14.5m high caisson are placed in a water depth of CD - 16.0m on a 4m high rubble foundation. A concrete cap brings the crest at the seaward side to CD + 5.7m. The caissons are to be protected by 50 tonnes concrete blocks, cf. Fig. 8-7.

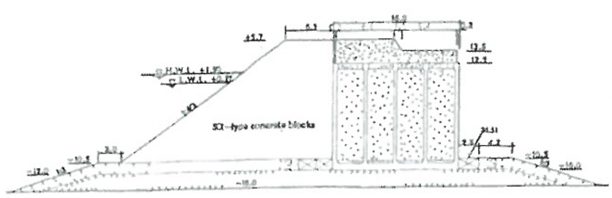


Figure 8-7: Cross-section, First Central Breakwater Hachinohe Port, Japan (Ligteringen 1996).

The design was made on the basis of present Japanese Technical Standards. Design conditions are as follows: $H_s = 6.26$m, $H_{max} = 10.94$m, $T_{1/3} = 13.0$s, $ß = 7.5°$. For the construction stage following conditions were adopted: $H_s = 5.33$m, $H_{max} = 9.02$m, $T_{1/3} = 13.0$s, $ß = 7.5°$. For both situations the design water level is taken at CD + 1.5m.

During the storm of February 1991 the estimated wave conditions were $H_s = 7.5$m, $H_{max} = 11.8$m (depth limited), $T_{1/3} = 11.4$s and $ß = 0°$. Considerable damage occurred to the outer 7 caissons (caisson no. 34-40), which were not yet fully completed: the 5 outermost caissons were not protected by the wave dissipating blocks at the seaward side and had only partial cap slabs. The sixth block was half covered, the seventh fully covered but without cap, cf. Fig. 8-8 and 8-9. This breakwater provides therefore a unique variation of conditions for investigation of failure/non failure conditions.

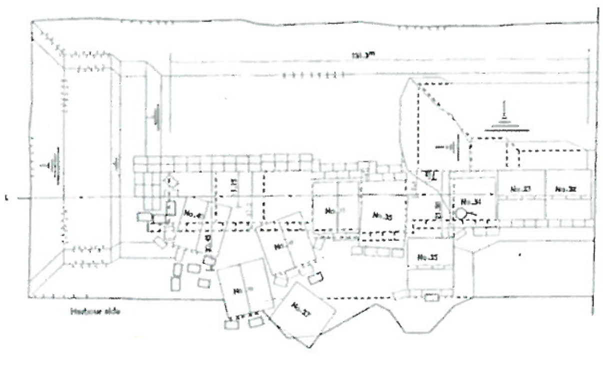


Figure 8-8: Plan-view of displacement of caissons around breakwater head, First Central Breakwater Hachinohe Port, Japan (Ligteringen 1996).

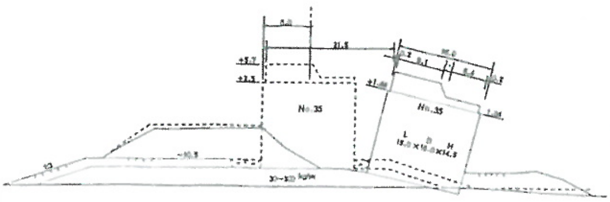


Figure 8-9: Cross-section of caisson no. 35 before and after the storm in February 1991, First Central Breakwater Hachinohe Port, Japan (Ligteringen 1996).

Analysis of the stability for the actual wave conditions using the Generalized Goda formula gives the following results:

- caissons completed and protected $SF_{sl} = 1.28$
- caissons completed and half protected $SF_{sl} = 0.6$
- caissons uncompleted and unprotected $SF_{sl} = 0.73$

The sliding of the caissons in the latter two cases and the stability in the former case are therefore explained by the analysis.

(iv) Type 4: Block Type Breakwater

Santa Cruz de la Palma Breakwater, Spain

Of the total length of this breakwater 500m were built in the fifties, 200m between 1969 and 1977, and the final 150m between 1976 and 1982.

The latter two parts consist of a block-work wall of 12m height, built on a large rubble mound foundation placed in CD -45m, and covered with a concrete cap and parapet wall, respective 2m thick and 8.5m high (see Figure 8-10).

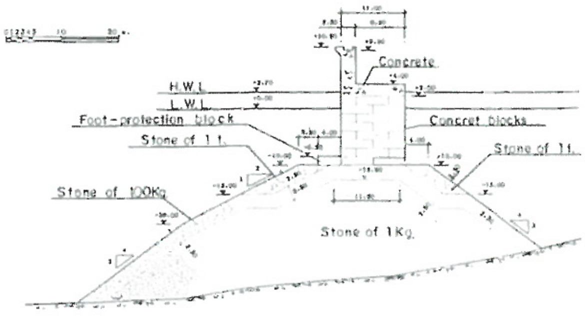


Figure 8.10: Cross-section, Santa Cruz de la Palma Breakwater, Spain (Ligteringen 1996).

The original design conditions were determined according to Iribarren's formula based on fetch length. This gave H = 5.4m, T = 12s, ß = 0°. Design water level of CD +2.7m is assumed. In the caisson design a safety factor on the wave height of 1.15 was introduced to account for uncertainties. Next Iribarren's stability formula was applied with safety factors against sliding of 1.35 and against overturning of 1.61.

The breakwater has not suffered any major damage such as dislocation of blocks. A considerable settlement of the outer portion (up to 1.8m) and some damage has been reported however (see Fig. 8-11):

- cracks in the upper concrete cap at the location of joints between portions of the blockwall, caused by differential settlement
- displacement of parts of the parapet wall
- erosion of the foundation berm

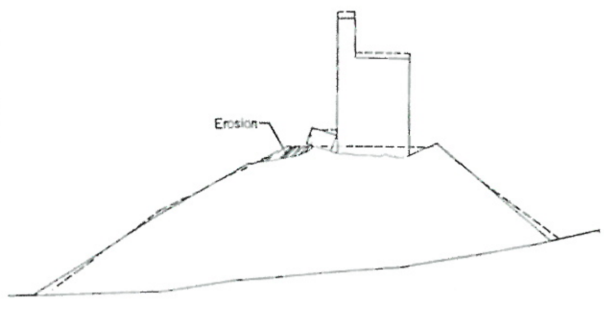


Figure 8-11: Cross-section survey by video, Santa Cruz de la Palma Breakwater, Spain (Ligteringen 1996).

A reassessment of maximum wave conditions at the breakwater location indicated H_s = 3.2m, $T_{1/3}$= 13s. The corresponding H_{max} would amount to 5.75m. Analyses of the breakwater stability for these wave conditions indicated that both Iribarren and the Goda formula allow ample margins of safety. According to Goda the breakwater will fail for H_{max} = 9.0m. The main problem of this breakwater has therefore been the large and differential settlement of the foundation mound.

(v) Type 5: Piled Breakwater

Manfredonia Breakwater, Italy

This is the only breakwater of this type reported. The structure was built in the period 1970-1972 in depth of MSL - 10.0m. A typical cross-section is given in Fig.8-12.

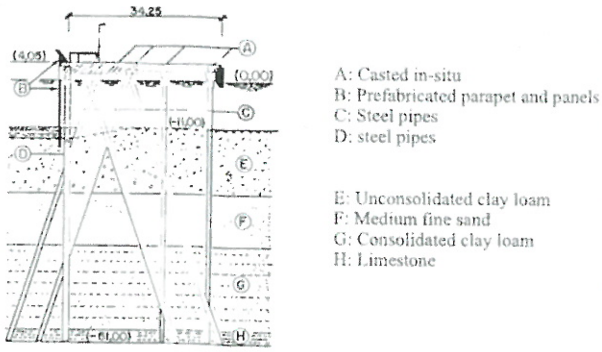


Figure 8-12: Cross-section, Manfredonia Breakwater, Italy (Ligteringen 1996).

The design conditions were derived from ship observations, applying refraction analysis. A wave height with return period of 5 years amounts to H_s = 4.5m with period T_p = 8-10s.

Design water level amounts to MSL + 1.0m. Soil conditions in the area are very poor. (reason why this type was chosen).

No detailed information on the breakwater performance could be obtained, but the general observation shows no structural damage.

Comparison of wave pressures and forces as measured in model tests with several design formulae gives this following result:

- Goda formula underpredicts measured pressures and forces by a factor of about 2 (does not take into account the impulsive impact pressures, which occur in the model)
- Hiroi formula also underpredicts pressures, but gives a reasonable approximation of the maximum force
- Minikin formula gives a severe over-prediction of measured pressures and forces.

8.4 Summary and Conclusions

8.4.1 Summary

In total 26 different cases, related to 14 breakwaters in Asia and Europe, have been investigated. The analysis of performance was concentrated on the sliding of caissons, mainly because this appeared to be the most common cause of damage. In some cases soil mechanic failure may have played a secondary role, but this was not further analysed. In other cases displacement of concrete armour units has been reported (toe blocks or wave dissipating blocks), without going into details and without analysis.

The range of conditions encountered in all these cases is comprehensive, i.e. inclusive of all combinations of extent of damage (no damage to total failure), storm wave conditions (below design wave height to well in excess of these) and structural conditions (uncompleted to almost and fully completed). The results of the analysis with respect to the appropriateness of the original design method and the applicability of the various formulae for calculating wave pressures and forces form therefore a good basis for conclusions.

The results are first summarized in two groups:

(i) Design wave heights not exceeded during service life

Damage	Sakata Section II and II Gela Second West Breakwater Niigata
No damage	Porto Torres Algeciras Santa Cruz de la Palma Los Gigantes Sakata Section I Manfredonia

Cause of damage can be attributed to inadequateness of design method. In the case of Sakata Section II and III the old Japanese Standards were applied, which do not account for impulsive wave impacts. In the case of Gela Breakwater the wave impacts on the parapet wall were probably underestimated. The case of Second West Breakwater at Niigata is complicated by the fact that the structure was not completed at the time of the storm event, that conditions are very poor and the wave direction was more at right angle than anticipated in the design. When accounting for these aspects, one finds still an SF > 1.0 according to the old Japanese Standards. But the (Generalized) Goda approach predicts sliding correctly. For Sakata Section II and III the Generalized Goda Formula does predict sliding for the storm conditions experienced.

In most of the no-damage cases the analysis indicates varying degrees of overdesign. Porto Torres Breakwater is designed on the basis of Sainflou Formula with SF_{sl} = 1.89, but appears to have still a large safety factor in Goda's approach. The Spanish breakwaters Algeciras, Santa Cruz and Los Gigantes are all designed according to Iribarren's method, with high safety factors against sliding. Goda's approach for design wave conditions gives similar or even higher values of SF_{sl} = 1.6 - 2.4. And finally Sakata Section I, which is designed with the old Japanese Standards, had apparently a high safety factor because re-analysis with Goda gives a minimum SF = 2.2 for the wave conditions very near design values.

The Manfredonia breakwater is difficult to judge, because the Generalized Goda Formula gives no increased wave pressures due to impact for this configuration, whereas the design based on model tests does take these into account. It is stated by the designers that the structure has probably some extra safety due to the fact that wave impact pressures in a small scale model are overpredicted by the inherent scale effects.

(ii) Design wave heights exceeded during service life

Damage	Mutsu Ogawara, Transient Section Mutsu Ogawara, V-Section Mutsu Ogawara, Concave Section Niigata West Kashima South Napoli, Section D and E Hachinohe First Central, Caissons 35 and 36
No damage	Hachinohe First Central, Caisson 32

The fact that design wave height has been exceeded clearly contributes to the occurrence of damage, but is not the only factor. In the case of Mutsu-Ogawara Transient and V-Section design is based on the original Goda Formula. The underprediction of (impact) pressures is evident in the Transient Section, where the Generalized Goda Formula gives SF_{sl} < 1.0. But for the V-Section the Generalized Goda Formula gives the same result as the original Goda Formula and an SF = 1.1 is found. It is believed that impulsive wave pressures have occurred at this section, but are not accounted for by the Generalized Goda Formula in its present form (compare Manfredonia). The design of Niigata West Breakwater and Kashima South Breakwater was based on the old Japanese Standards, leading to insufficient stability as we have seen in the previous category. Added to this was the fact that in both cases the construction was still not completed, in particular the concrete cap. When the actual configuration is analysed for the storm conditions experienced, the Generalized Goda Formula predicts the sliding properly. The design method for

Napoli Breakwater is unknown. Application of the original Goda Formula leads to SF < 1.0 for Section D but gives $SF_{sl} > 1.0$ for Section E. When adding the effect of impulsive pressure in the stability analysis, the sliding of the latter caisson is probably explained. The most comprehensive case investigated is the First Central Breakwater at Hachinohe. Because of the various stages of completion of this structure it presents 3 different conditions, within a stretch of 134m, at the end of the portion still under construction at the time of storm damage. Caisson no. 37 is not yet protected by the seaward armour, and has no concrete cap. Caisson no. 34 is half protected, but has a concrete cap in place.

And Caisson no. 32 is fully protected and also covered by the concrete cap. The design is based on the present Japanese Standards and the original Goda Formula underpredicts the wave pressure and forces for the unprotected caissons. The Generalized Goda Formula gives $SF_{sl} < 1.0$ for Caisson 37 and 34 applying the wave conditions as occurred, and hence predicts the sliding in prototype. It also gives SF = 1.28 for Caisson 32, which has not moved during the storm. A final result is related to the caissons with a perforated wall, the Jarlan Caisson. Roscoff-Bloscon Breakwater (Type 1) and a part of Porto Torres Breakwater (Type 2) have a perforated seaward wall. Analysis of these cases was based on the Modification Factors for vertical slit wall caisson, as incorporated in the Generalized Goda Formula (Tanimoto and Takahashi, 1994). For Porto Torres Breakwater very high safety factors are found, while for Roscoff-Bloscon the safety factors are well below unity, while in prototype very little displacement is found. The underlying conditions are too uncertain to draw conclusions regarding the applicability of the Modification Coefficient. Moreover it must be recognized that these coefficients are derived from one set of model tests for a particular geometry, which is quite different from that of the above cases.

8.4.2 Conclusions

Based on the foregoing, the following conclusions are formulated:

1. Most examined damage cases are caused by the fact that the original design was based on inadequate designs. Hiroi Formula in the old Japanese Standards underpredicted wave pressure and forces as recognized long ago. Sainflou Formula does not account for breaking wave pressures.

2. Most of the no-damage cases are considerably overdesigned. The design method of Iribarren is based on high values of SF_{sl} to start with and the structure thus designed shows even higher safety factors when analysed with Goda's Formula.

3. The result of the Generalized Goda Formula predicts prototype behaviour (sliding and no sliding) well, if applied for values of $d/h_s < 0.7$. For very low levels of the bedding layer or berm in front of the caisson, this formula gives results equal to the original Goda Formula, i.e. the impulsive pressure coefficient equals one. There is ample evidence that for such geometry impulsive wave pressure can occur, depending on the seabed slope and wave steepness in deepwater. For such cases the Generalized Goda Formula still underpredicts wave pressures and forces.

4. The applicability of the Generalized Goda Formula for a perforated caisson (Jarlan type) is unclear, partly due to the uncertainties in the investigated cases and partly because the geometry in those cases is quite different from the schematical configuration for which design coefficients are given.

Chapter 9

MATERIALS, CONSTRUCTION AND MAINTENANCE

9.1 Concrete

9.1.1 General Principles

The performance of concrete in seawater is a subject for which knowledge and guidance remains fragmented and ill-understood. The main reasons for this are that climatic and exposure conditions vary widely, different materials have been used in various countries, and that the properties of cement have changed during the century. A basic reason is, also, that deterioration can take a sufficiently long time such that it can be difficult to connect cause with effect. The mechanisms for the deterioration of concrete structures have not been adequately understood. For this reason, the subject of concrete durability is the largest single element of the report of Subgroup C.

9.1.2 Design Working Life (or Service Life)

"Design working life" is the term and definition from Eurocode 1 (European Committee for Standardization, 1994), and has three main implications for maritime structures:

- probability levels for wave return periods
- probability levels for limit state design
- time dependent factors such as corrosion and durability.

A period of operating or service life (related to operational and maintenance strategy) has to be considered by the owner of a structure and the means of achieving this has to be addressed by the designer. The definition of service life, design life and economic life require careful consideration, as there are many different definitions in use. The main categories of definition are compared in a table. For maritime structures, subject to the probability and return periods of environmental loading, the following definition is recommended in which the definition of Eurocode 1 is supplemented by the rider expressed in italics: "The assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary *within a probability appropriate to the function of the structure*".

Figures for design working lives specific to maritime structures within the classification of Eurocode 1 are given, drawn from the Spanish maritime recommendations (See Table 9.1.)

9.1.3 Processes of Deterioration

The various deterioration mechanisms which affect the durability of concrete maritime structures, the locations in which they are likely to occur, and methods of avoidance are scheduled in Table 9.2.The most widespread and critical problem is that of chloride-induced corrosion of steel reinforcement, and the sections which follow concentrate on this phenomenon. Adequate guidance on other forms of deterioration is usually given in national standards, as scheduled in the Sub-Report.

9.1.4 Exposure Classification

The most important macro-climatic factors are temperature and rainfall. Temperature controls the rate of chemical reactions and the degree of drying out of the cover concrete. Rainfall, humidity and the location of a member in relation to sea level movement control the wetness of concrete. The wetness of the concrete determines the mechanism for the penetration of chlorides and controls the penetration of oxygen to fuel the corrosion process.

Contrary to the case of structural steel, timber and masonry, plain concrete and for freeze-thaw damage, reinforcement corrosion is less severe in the regularly wetted tidal and splash zones. In cool and temperate climates, the concrete does not dry out to appreciable depth. However in the infrequently wetted and mostly dry zone above the tidal zone but subject to irregular inundation from seasonal changes in sea level, storms etc., concrete dries out to greater depth. Especially in hot-arid areas such as the Middle-East, and also where elements are sheltered from rain or in artificial climates such as in tunnels, the sporadic wetting of the dried-out concrete enables chloride-laden

Table 9.1: Design Working-Lives (Service Lives)

DESIGN WORKING-LIVES (SERVICE LIVES) DEFINED IN ROM 0.2-90 AS "MINIMUM DESIGN LIVES"* FOR WORKS OR STRUCTURES OF DEFINITIVE CHARACTER (IN YEARS)

TYPE OF WORK OR INSTALLATION	REQUIRED SECURITY LEVEL		
	LEVEL 1	LEVEL 2	LEVEL 3
GENERAL USE INFRASTRUCTURE	25	50	100
SPECIFIC INDUSTRIAL INFRASTRUCTURE	15	25	50

LEGEND

GENERAL USE INFRASTRUCTURE

General character works: not associated with the use of an industrial installation or of a mineral deposit.

SPECIFIC INDUSTRIAL INFRASTRUCTURE

Works in the service of a particular industrial installation or associated with the use of transitory natural deposits of resources (*e.g.* industry service port, loading platform for a mineral deposit, petroleum extraction platform, etc).

LEVEL 1

Works and installations of local or auxiliary interest. Small risk of loss of human life or environmental damage in case of failure.
(Defence and coastal regeneration works, works in minor ports or marinas, local outfalls, pavements, commercial installations, buildings, etc).

NB: 1. The General Use period of 25 years corresponds with Class 2 of draft Eurocode 1.

LEVEL 2

Works and installations of general interest.
Moderate risk of loss of human life or environmental damage in case of failure. (Works in large ports, outfalls of large cities, etc).
NB: 1. The General Use period of 50 years corresponds with Class 3 of draft Eurocode 1.

LEVEL 3

Works and installations for protection against inundations or international interest. Elevated risk of human loss or environmental damage in case of failure.
(Defence of urban or industrial centres, etc).
NB: 1. The General Use period of 100 years corresponds with Class 4 of draft Eurocode 1.

*Defined as Design Working Life in draft Eurocode 1.

water to be very rapidly sucked in to greater depth by absorption. The processes of absorption, capillary suction and wick-action lead to much more rapid chloride ingress than the diffusion process which operates in saturated concrete. In a wet climate the chloride concentration at depth is reduced and the penetration of oxygen is limited.

The then proposed new Eurocode exposure classification system, now incorporated in EN 206-1, (2000), is explained and new suggestions for severity ratings for concrete exposed to chloride induced corrosion expressed on a scale of 1 to 12, are illustrated as in Fig. 9.1. This Fig. demonstrates the relative severity of exposure conditions in the submerged, intertidal and splash zones in the range of macro- and micro-climatic conditions and has been included in BS 6349, 1 : 2000.

9.1.5 Influence of Cement Type

The weakness of much prescriptive advice in current codes is that guidance on mixes and associated cover thickness to reinforcement is given independently of cement type. The behaviour of the various types of cement is compared and it is concluded that :

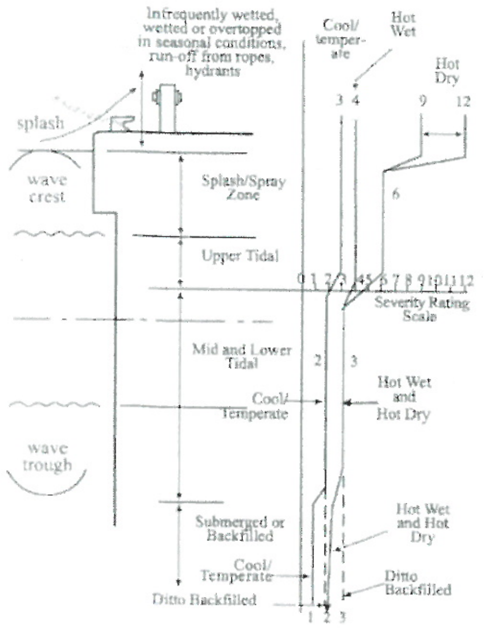


Fig. 9.1: Suggested severity ratings on a scale of 1-12.

Table 9.2: Deterioration Mechanisms for Maritime Concrete.

DETERIORATION MECHANISMS FOR MARITIME CONCRETE

Deterioration Mechanism	Locations most likely to occur	Method of Avoidance
Reinforcement corrosion (due to chlorides)	Elements wetted but subject to drying - especially hot dry climates. See Figure 13. Corners subject to increased wetting and then drying. Areas of low cover.	Analysis, design and detailing. Properly designed cover to reinforcement for specific exposure conditions and tolerances.
Sulfate attack on concrete matrix	Delayed action in seawater. Colder waters may be more critical.	Specification and tests.
Salt weathering of concrete surface	Elements subject to concentration of salts by drying - intertidal zone. Paradoxically, cements which achieve the finer pore structure and resistance to steel corrosion may be most susceptible.	Specification. Extensive water curing.
Alkali-aggregate reaction	Susceptible aggregates, pessimum reaction with mixed aggregates. Alkalis from sea water and marine aggregates. Rich mixes.	Specification and tests Petrography. Mix limitations.
"Frost" (freeze-thaw) action	In cool with freezing zones with prolonged and repeated freezing.	Specification and detailing. Air entrainment spacing factor.
Abrasion	Subject to abrasive bed movement, shingle, vessel impacts, ropes and moorings.	Higher strength concrete, detailing, extensive curing, controlled permeability formwork, permanent steel protection.
Early thermal cracking	Thick sections and massive structures built in separate pours, causing restraint to shrinkage during cooling from heat of hydration.	Design and detailing, specification, pre-cooling of mix, cooling pipes inbuilt for the hydration period.
Plastic shrinkage cracking (workmanship)	Arid climates, drying winds, low bleed mixes.	Curing and protection at casting.
Plastic settlement cracking (workmanship)	Deep sections, high bleed mixes.	Mix design, reduction of bleeding. Revibration.

- modern unblended Portland cement generally has the lowest resistance for chloride penetration and, where severe chloride exposure conditions exist, even in temperate climates, traditional thickness of cover may be inapplicable. There are exceptions in some national products and conditions. Blast furnace cements are highly recommended and have been traditionally used in some countries (originally on account of sulfate resistance) and enable more traditional thickness of cover to be used. Their tolerance to surface scaling and poor curing is, however, less than for unblended Portland cement. Other blending materials, such as fly ash and microsilica, have their benefits and limitations
- sulfate resistant Portland cements (i.e. with C_3A less than 5 %) are unlikely to be necessary in maritime concrete. A compromise solution is often reached by controlling the C3A to between 5 % and 10 % for moderate sulfate resistance. In conditions where reinforcement corrosion is not critical and, especially in colder waters, the long term effects of sulfates may lead to a need for low C_3A Portland or slag cement.

9.1.6 Influence of Cement Content

As is well known, the quality of a concrete mixture in relation to both strength and durability (as related to the pore structure) is controlled primarily by the water-cement ratio and the unit water content. The water-cement ratio is therefore more important as a parameter to be specified than is cement content. The cement content is established, mainly, by dividing the water demand for a given mix by the water-cement ratio. As it is desirable to use the lowest possible water-cement ratio to achieve durability (generally the requirement for durability may be more onerous than for strength) and to reduce water movement and shrinkage effects, the cement content is controlled by the water content required to achieve appropriate workability.

9.1.7 The Influence of Cracking

The causes and consequences of cracking have often been misunderstood. Early thermal cracking caused by restraint to shrinkage during cooling from the rise in temperature due to heat of hydration is a main cause of cracking which was previously, and erroneously, attributed to drying shrinkage.

Cracking caused in the plastic state can be prevented by good mix design, protection against drying winds and by good curing under arid conditions.

The cover to reinforcement should not be reduced for crack width control reasons.

Cracks do not significantly affect freeze-thaw damage as the scaling caused by freeze-thaw is, again, due to the effect of frost on water-filled pores. Cracking may, of course, be more significant in the case of unreinforced concrete if it reduces the mass of armour units or blocks.

9.1.8 Influence of Curing

It appears that prolonged water curing in wet and temperate climates may be of limited advantage and may even lead to adverse effects such as thermal shock. It may be essential in hot and arid climates.

As the duration of curing is inversely proportional to water-cement ratio, adoption of a low water-cement ratio enables the curing period to be reduced.

9.1.9 Monitoring and Maintenance

Inadequate guidance on this strategic topic is available in the literature and national codes, but it appears that it is, at last, receiving more attention. Regular inspections should be carried out at least once per year, most likely following the winter storm period. The principle objects of the survey are to determine :

- The integrity of armour units and elements of the structure
- indication of movement and settlement
- scour.

It is essential to record "base-line" measurements of line and level immediately on completion of construction. This should include "as constructed" measurements of cover to reinforcement and crack and damage mapping. Computers, underwater video recorders and corrosion measurement devices can now be utilized.

9.2 Materials

9.2.1 Rock and Rubble

Reference is made to the RWS Report 169 (2000)/CUR.

9.2.2 Filling and Backfilling

Current requirements are outlined, including the recommendation that measures may be necessary to increase the density of infill material.

9.2.3 Unreinforced Concrete

The factors affecting unreinforced concrete (more usually termed "mass concrete" in Europe) primarily concern deterioration of the exposed surface and include freeze-thaw, abrasion, and sulfate attack. These forms of deterioration have similar and overlapping effects, and are described in more detail. Early-thermal design is important for crack avoidance. Suggestions for the choice of water-cement ratio, and hence minimum cementitious contents and grades, for various cement types and blends and minimum dimensions of pour, for a range of exposure conditions, are tabulated.

9.2.4 Reinforced Concrete, including the Selection of Cover to Reinforcement

The main conflict point in the design and production process is the selection and practical achievement of the appropriate cover to reinforcement. As the protective power of a given concrete is broadly related to the square of the cover thickness, the provision of appropriate cover is the simplest and most positive way of reducing corrosion damage. The cover to be specified is influenced by the exposure conditions, the cement type, the mix quality as determined by the water-cement ratio and the placing tolerance that can be achieved, see Table 9.3.

The minimum cover considered necessary for corrosion protection should be regarded as a "characteristic" value and a margin of at least 10mm to 15mm should be added to the figure in order to reduce the rate of failure to achieve the characteristic value to within 5 %, by analogy with concrete strength compliance. Without this margin it is statistically impossible to achieve the necessary cover as, in practice, the variation in position of reinforcement about the mean position exceeds common perception. A nominal cover of 50mm is the lowest practicable figure and is only suitable for the lowest severity rating and using blended cements.

Nominal cover thicknesses between 75mm and 100mm have to be considered as normal. For severe exposure combined with unblended Portland cements, it may be necessary to double the cover.

Since the preparation of the Sub-Report, the new exposure classification system has been published in the European Committee for Standardization, 2000, EN 206-1. The application to various national standards, has, however, not achieved consensus for the specification of concrete classes and cover, and national applications will be covered by national application documents.

However, a development of this system specific for maritime structures in the UK has been included in BS 6349 Pt 1: 2000. This document also opens the door to analytical durability assessment as well as prescriptive limits.

9.2.5 Protective Measures such as Coatings and Cathodic Protection

In aggressive conditions only high-build, high quality coating products are dependable, which are expensive and require the concrete surface to be prepared to a high

Table 9.3: Suggested Nominal Cover for reinforced Concrete (Slater D. and Sharp B.N.,1998, Concrete in Coastal Structures, Thomas Telford, London).

SUGGESTED NOMINAL COVER FOR REINFORCED CONCRETE (BEFORE ABRASION ALLOWANCE) FOR DIFFERENT MARINE ENVIRONMENTS FOR 60 YEARS "DESIGN LIFE"
(suggested as appropriate for 50 years "design working life")
(ref Slater D and Sharp B N, scheduled for publication late 1997)

Exposure Severity Rating	Suggested Nominal Cover[1,2,3] mm			
	75% pc : 25% pfa 50% pc : 50% gbs [4,5]	70% pc : 30 pfa 30% pc : 70% gbs 90% pc : 10% ms [6,7]	100% pc w/c ratio 0.45 [8]	100% pc w/c ratio 0.40 [9]
1	50 [10]	50 [10]	75	65
2	50	50 [10]	95	85
3	65	50	120	105
4	80	60	145 [11]	130
5	95	70	170 [11]	155 [11]
6	115	85	200 [11]	180 [11]
9-12	135 [12]	100 [12]	230 [11,12]	205 [11,12]

Notes:

1 Includes an allowance of 15mm for workmanship tolerances and reduction of cover during concreting.

2 Add an extra 10mm for prestressing strand to reduce percentage non-compliance of nominal cover to minimal value in recognition of risk of pitting corrosion.

3 A combination of the suggested nominal cover plus concrete mix appropriate to higher exposure rating will provide extended service life.

4 Appropriate mix for exposure severity rating 2: Grade C35/45, minimum cementitious content 370 kg/m³, maximum water/cement ratio 0.45, 20 mm aggregate. See note 10, Table 14, for definition of Grade.

5 Assumed apparent diffusion coefficient at 20°C $3.0 \times 10^{-13} m^2 sec^{-1}$.

6 Appropriate mix for exposure severity rating 5: Minimum Grade C40/50-55/65) minimum cementitious content 400 kg/m³, maximum water/cement ratio 0.40. Appropriate mix for exposure severity rating 6-12: Minimum Grade C45/55-55/65, minimum cementitious content 425 kg/m³, maximum water/cement ratio. 0.34-0.38 20 mm aggregate. See note 10, Table 14, for definition of Grade.

7 Assumed apparent diffusion coefficient at 20°C $1.5 \times 10^{-13} m^2 sec^{-1}$.

8 Assumed apparent diffusion coefficient at 20°C $15.0 \times 10^{-13} m^2 sec^{-1}$.

9 Assumed apparent diffusion coefficient at 20°C $11.0 \times 10^{-13} m^2 sec^{-1}$.

10 Nominal cover of 50mm dictated by bond requirements with 20mm maximum aggregate size and allowing for workmanship tolerances.

11 Blended cementitious mix more suited to the exposure severity recommended.

12 Note that this Severity Rating is for hot arid conditions and infrequently wetted members. See Section 2.3.4. Extra protection may be required by means of coatings or provision for cathodic protection, depending upon application and estimated severity rating.

standard by grit blasting and other means. Cathodic protection is not regularly used for the protection of new construction, although in some cases allowance for later implementation are made by ensuring continuity of reinforcement and facility for electrical connection. There is growing experience of its use as a repair technique where simpler alternatives are not feasible.

9.2.6 Corrosion of Structural Steel

The corrosion performance of structural steel in maritime conditions is much better known than that of reinforcing steel embedded in concrete.

The corrosion rate is usually higher in the splash zone and at low astronomical tide levels, and very low in deep water. Either an extra thickness of metal as a "corrosion allowance", high duty coating or cathodic protection can be used. Successful traditional coatings, however, may no longer meet environmental and health and safety regulations for application and there is, as yet, inadequate experience with some new water-based systems.

9.3 Construction Related Criteria and Methods for Caissons

9.3.1 Sea Conditions Data and Limits for Construction Risks

Wave forecast data must be available to enable the designer and the contractor to estimate the sea state at each stage of construction, especially for float-out, grounding and filling of caissons. These data should include information on the proportion of time during a year in which certain wave heights are not surpassed and the length of windows for calm weather.

9.3.2 Construction Joints

Construction joints are an important feature of vertical breakwaters. Horizontal joints seldom give problems. Vertical joints are necessary to allow differential settlement to occur between adjoining elements, but at the same time the interconnection of elements is required in order to distribute the load from local wave attack over more than one element. Typical examples of jointing methods are shown on Figure 9.2.

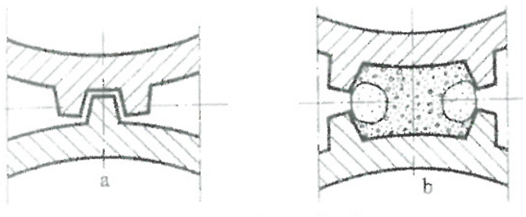


Fig. 9.2: Vertical joints between caissons with a circular horizontal cross-section

9.3.3 Settlement

The magnitude of settlement observed for vertical breakwaters is higher than for most other forms of construction. Settlement is rarely critical but the range of likely settlement and differential settlement should be anticipated, and the visible effects of settlement should be minimized by appropriate detailing features.

Examples of measured settlement are quoted, including absolute settlement of up to 1.5m and a differential of 0.2m. The contribution to settlement caused by the time-dependent consolidation of rock-fill for many years after construction is not generally appreciated. Examples are given using the logarithmic expression published by Penman and others (Penman, 1971).

Chapter 10

RECOMMENDATIONS

10.1 Design codes and design procedures

No existing design codes, standards and recommendations specify target safety levels for breakwater structures. In almost all cases are specified a sea state return period, overall safety coefficients (e.g. for caisson sliding) and some specific values of parameters like the friction coefficient between caisson base plate and rubble foundation. However, such specifications do not secure any specific safety level. In fact, application of such specification leads to unknown safety/reliability of the structures.

Consequently there is a need for new concepts in national and international design codes and recommendations.

It is recommended that such documents specify maximum probabilities of damage/failure within the structure lifetime, based on the following concept that includes classification of the breakwaters and the design limit stages.

<u>Safety class</u>

Very Low Safety Class, where failure implies no risk to human injury and very small environmental and economic consequences.

Low Safety Class, where failure implies no risk to human injury and some environmental and economic consequences.

Normal Safety Class, where failure implies risk of human injury, significant environmental pollution or high economic and political consequences.

High Safety Class, where failure implies risk of human injury, significant environmental pollution or very high economic and political consequences.

Design limit states

As a minimum the following two classes should be included:

ULS Ultimate Limit State
Examples: foundation failure, failure of significant part of caisson concrete structure

SLS Serviceability Limit State
Examples: Overtopping, settlement of foundation soil

The acceptable maximum probabilities of failure must be specified on the basis of a systematic cost-benefit investigation for typical breakwaters and/or on the basis of what is generally accepted safety levels for breakwaters. The values given in Table 10-1 are indicative estimates on reasonable acceptable failure probabilities, which ensure designs not far from existing practice.

Table 10-1: Indicative values of acceptable (maximum) probabilities of failure within structure lifetime

Limit State	Safety class			
	Very low	Low	Normal	High
SLS	0.4	0.2	0.1	0.05
ULS	0.2	0.1	0.05	0.01

Having defined such safety classes, design limit states and related acceptable failure probabilities, then the design can be performed on the basis of the developed partial coefficients given in Chapter 7 and in the Subgroup A and D reports. The list of partial coefficients can be expanded to cover also other failure modes than those presented in the report.

10.2 Overtopping

Ranges for admissible average overtopping discharge are provided in Chapter 4.1 together with a formula for estimation of overtopping volumes of individual waves.

However, investigations of the damaging effect of overtopping water on various structures and installations are very limited and not sufficient for proper design of structures and installations in the zone of falling water. More research is needed in this field.

The still more frequent use of breakwaters for promenades makes it important to study means of reducing overtopping without increase of the height of the structure, which in many cases must be kept relative low for economic reasons.

10.3 Slip failure calculations

Geotechnical failure of the rubble foundation is generally analysed as 2-dimensional limit state failure, assuming simultaneous development of failure all along the failure surfaces, and static forces, i.e. negligence of inertia forces and possible dynamic amplification. In reality the failure of a fairly short caisson will be 3-dimensional, for which case 2-dimensional calculation will be on the safe side. Subsoil pore pressure built-up, which might lead to liquefaction, should be considered. Also dynamic amplification, which takes place when the natural frequency of the structure-soil system is close to the wave load raise time, should be considered.

There is a need for development of more precise calculation methods that reflects better the strength and deformation characteristics of the rubble material.

10.4 Structural, material and construction aspects

10.4.1 Structural Analysis and Change to Probabilistic Approach

Structural analysis of caissons can be carried out by the traditional approach, in which the structure is split into sets of beams and slabs, guidance on which is amply given in national codes. Computer methods are likely to be used for two-dimensional frame analysis. For detailed final design it is more likely that full three dimensional model analysis will be used, using finite element analysis.

In implementing finite element models, the main problem may be the modelling of soil behaviour, i.e. the definition of stress-strain relationship. The simplest approach assumes a linear unconnected spring relationship, as per Winkler.

This simplistic assumption disregards the inter-connection of the soil elements, and these can either be modelled as well or the simpler method used with sensitivity tests on the soil elasticity parameters. It is suggested that as a complex soil model is critically dependent on soil testing and interpretation, as well as its comparison with the stiffness of the structure, it is sensible to test the design against local reductions of ground support.

It is emphasised that caution must be exercised in making the transition from traditional working stress design methods to limit state methods which are now general, worldwide. It is not simply a case of adapting partial factors from one national code to another, because the underlying principles of reinforced concrete design may be different. The recommended partial factors in most national codes were derived for land-based building and bridges and relate to broad probabilities of failure drawn from historical precedent. These factors are not necessarily applicable to maritime structures in which the main loading cases are caused by environmental loads that have to be derived from a probabilistic approach.

10.4.2 Earth Pressure Analysis for the Design of Structural Elements

Similar problems to those outlined in 10.4.1 relate to earth pressure loading and to establishing compatibility between recent geotechnical codes and structural codes. The geotechnical codes introduce the concept of limit states to be used in the calculation of geotechnical stability. Structural codes, however, give partial factors appropriate for general structural analysis of bridges and buildings. Adjustments to these partial factors can be appropriate for the design of maritime structures, and are under current debate or are already adopted in some national codes.

Reference – Andrew Beeby and Brian Simpson (2001), discussion of clarifications to BS 8110 and BS 8002 on the treatment of soil loads.

10.4.3 Concrete Durability

Durability of concrete and other constructural materials is not simply a question of materials, i.e. a product related matter. As well as 'product' one has to consider 'process', which includes conceptual and detailed design, plus the full consideration of the type and method of construction, and maintenance and management in service.

At the time of preparation of the concrete durability sections of this work the early drafting of the new European Standard for concrete (as a product) was in hand, and a new approach to classifying exposure conditions according to the various separate deterioration mechanisms for concrete, was being developed. This standard has now been completed and ratified as EN 206 : 2000. However, it proved difficult to standardize all approaches to durability in the various countries of Europe, due to differing materials, climates and experience. Consequently, although the principles and bases of comparisons of conditions and materials are the same, there will also be extensive national complementary standards for concrete.

However, the approach in these standards is, generally, for concrete as a product. This is not the case when considering the specific case of the design of maritime structures. In aggressive situations and under environmental loading, design cannot be isolated from materials and a total approach to design and construction is essential.

Durability, itself, is not a limit state but the means by which the structural limit states are maintained for the operational life of the structure.

Current codes and standards, even the latest EN 206, do not provide a rational framework to design concrete for specific life periods, but deal with durability only by prescriptive rules. The principal risk is that of corrosion of reinforcement. The limits leading to failure in either the serviceability or the ultimate limit state may involve:

- The onset of corrosion
- the rate of propagation of corrosion
- cracking resulting from corrosion
- spalling and/or loss of steel and/or concrete section.

These conditions require assessment of the consequence of deterioration to the type of structure and the capability for repair or replacement. The acceptable risks and the assessment of reliability or probability of failure require to be introduced in a similar way to the considerations for reliability of the design for overall stability and for structural safety. There may be not one simple solution, but a range depending on the risks accepted.

At the present time, analytical methods of durability design, including the considerations of risk and probability, are being introduced. At this stage, there is insufficient agreement of the models appropriate for analytical design. For example the British National Standard for maritime structures, BS 6349 Part 1 (2000), opens the door to the use of analytical methods and builds upon the methods of EN 206 for the specific case of maritime structures.

REFERENCES

Battjes, J.A. (1982). *Effects of short-crestedness on wave loads on long structures*. Applied Ocean Research, Vol. 4, No. 3.

Beeby, A. and Simpson, B. (2001). *A proposal for partial factors for soil pressures in BS 8110*. The Structural Engineer, Volume 79, No 8, London.

British Standards Institution, (1994). BS8002 *Code of practice for earth retaining structures, London*.

British Standards Institution, (2000). BS6349, Maritime structures – Part 1 : Code of practice for general criteria, London.

Burcharth, H. F. (1992). Uncertainty related to environmental data and estimated extreme events. Final Report of PIANC Working Group 12, Subgroup D. PIANC, Bruxelles.

Burcharth, H.F. (1993). *The Design of Breakwaters*. Department of Civil Engineering, Aalborg University, Denmark.

Burcharth, H.F. and Liu, Z. (1998). *Force reduction of short-crested non-breaking waves on caissons*. Section 4.3, Chapter 4, Class II Report of MAST II Project: PROVERBS, Edited by Technical University of Braunschweig, Germany.

Burcharth, H. F. and Sørensen, J. D. (2000). *The PIANC safety factor system for breakwaters*. Proc. Int. Conf. Coastal Structures '99, Santander, Spain. Balkema.

CUR/RWS Report 169 (2000). *Manual on the use of Rock in Hydraulic Engineering*. Balkema. Rotterdam/ Brookfield.

European Committee for Standardisation, (1991). EUROCODE 2 – ENV, 1992-1-1: 1991. *Design of concrete structures, Part 1. General rules and rules for buildings*.

European Committee for Standardisation, (1993). EUROCODE 7, ENV1997-1: 1993 – *Geotechnical design, Part 1.General rules*.

European Committee for Standardisation, (1994). EUROCODE 1 – ENV, 1991-1: 1994. *European pre-standard. Basis of design and actions on structures, Part 1. Basis of design*.

European Committee for Standardisation, (2000). EN206-1: Concrete – Part 1: *Specification, performance, production and conformity*.

Goda, Y. (1969). *Reanalysis of laboratory data on wave transmission over breakwaters*. Report of Port and Harbour Research Institute, Vol.8, No.3, pp 3-18, Japan.

Goda, Y. (1974). *A new method of wave pressure calculation for the design of composite breakwaters*. Proc. 14th Conf. Coastal Eng., ASCE, Copenhagen.

Goda, Y. (2000). *Random seas and design of maritime structures*. Advanced Series on Ocean Engineering, Volume 15, World Science.

Helm-Petersen, J. (1998). *Estimation of wave disturbance in harbours*. Ph.D. thesis, Department of Civil Engineering, Aalborg University, Denmark.

Kortenhaus, A. and Oumeraci, H. (1998). *Classification of Wave Loading on Monolithic Coastal Structures*. Proc. of the 26th Int. Coastal Engineering Conference, Vol. 1, pp 867-880.

Lundgren, H. (1969). *Wave shock forces: An analysis of deformations and forces in the wave and in the foundation*. Proc. Symposium: Research on Wave Action, Vol. 2. Delft, The Netherlands.

Ministry of Public Works, Madrid, (1990). ROM *(Recommendations de obras maritimas)*. Maritime works recommendations ROM 0.2-90, *Actions in the design of maritime and harbour works (General Criteria – in English)*.

Oumeraci, H., Partenscky, H. –W. and Tautenhain, E. (1992). *Breaking wave loads on vertical gravity structures*. Proc. 2nd International Conf. Offshore & Polar Eng., ISOPE`92, Vol. 3, San Francisco, pp.532-539.

Overseas Coastal Area Development Institute of Japan, (1991). *Technical standards for port and harbour facilities in Japan*. Bureau of Ports and Harbours, Ministry of Transport, Tokyo.

Penman A. D. M., (1971). *Rockfill, Building Research Station Current Paper 15/71*. Building Research Establishment, Garston, U.K.

PIANC PTC II Working Group 12, Sub-group B, final report June 1992. Published by PIANC.

PIANC (2001). *Seismic Design Guidelines for Port Structures*. A. A. Balkema Publishers.

PROVERBS (2001). Report from the EU-MAST III *project Probabilistic Design Tools for Vertical Breakwaters*. Ed. Kortenhaus and Voortman, Balkema.

Slater D. and Sharp B. N., (1998). C Slater D. and Sharp B. N., (1998). *Chapters 4-6 in design guides, specifications and design.* Allen, RTL (editor), Concrete in Coastal Structures. Thomas Telford, London.

Tanimoto, K., Moto, K., Ishizuka, S. and Goda, Y. (1976). *An investigation on design wave force formulae of composite type breakwaters.* Proc. 23rd Japanese Conf. on Coastal Eng. (In Japanese).

Tanimoto, K. and Takahashi, S. (1994). *Japanese experience on composite breakwaters.* Proc. Int. Workshop on Wave Barriers in Deepwaters, Port and Harbour Research Institute, Yokosuka, Japan.

Takahashi, S., Tanimoto, K. and Shimosako, K. (1994). *A proposal of impulsive pressure coefficient for design of composite breakwaters.* Proc. Int. Conf. on Hydro-technical Engineering for Port and Harbour Construction. Port and Harbour Research Institute, Yokosuka, Japan.

Van der Meer, J. W., d'Angremond, K. and Juhl, J. (1994). *Probabilistic calculations of wave forces on vertical structures.* Proc. 24th Int. Conf. Coastal Eng., Kobe, Japan.

APPENDIX

The following example demonstrates the use of partial safety factors in the design of a caisson breakwater. The result is also compared with that of the conventional deterministic design for the same case.

(i) Design process

The design process is simplified to the determination of the width of the caisson, necessary to achieve stability for sliding and overturning. The overall geometry of the vertical composite breakwater is already defined and shown in Fig.1. Other failure mechanisms, such as slip failure through the mound and instability of the rubble mound foundation, are not considered in this example.

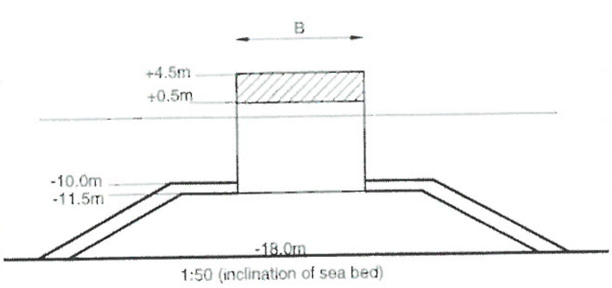


Fig. 1: Basic geometry of the vertical composite breakwater.

(ii) Design data

The service life time for the structure is set to 50 years. The structure is considered to belong to the normal safety class, cf. Table 10-1, which implies a safety level corresponding to a probability of serious failures, cf. $P_f = 0.05$ (ULS). The concept of the presented partial safety factor system implies that in the calculations the return period for the design wave conditions is set equal to the structure lifetime, i.e. 50 years in this example.

The design wave conditions for 50 year return period are determined on the basis of wave records at the location of the breakwater and read as follows:

H_s	=	6.45 m
T_p	=	11.0 s
β	=	10 deg

The breakwater is located outside the surf zone and no impulsive breaking wave forces are to be expected.

Other geometrical parameters in Goda's formula can be derived from Fig.1, as follows:

h_b	=	18.65 m (at distance 5xH_s)
d	=	10.00 m
h'	=	11.50 m
h_c	=	4.50 m

The specific weight of the caisson material is given as:

- concrete cap
  w = 23 kN/m^3
- caisson above water
  w = 21 kN/m^3
- caisson below water
  w = 11 kN/m^3 (incl. Buoyancy effect)

(iii) Design based on Partial Safety Factors

The relevant design equations in this example are (see Ch.7.2):

Sliding:

$$G = (\hat{F}_G - U_V \cdot F_U)\gamma_Z^{-1} \cdot \hat{f} - U_H \cdot F_H \geq 0$$

$$G = (\hat{M}_G - U_{VM} \cdot M_U) - \hat{U}_{HM} \cdot M_H > 0$$

in which F_H, F_U, M_H, and M_U are calculated on the basis of:

$$H_s = \gamma_H \cdot \hat{H}_s^{TL}$$

The partial safety factors are obtained from Tables 7-2 and 7-3 respectively, taking the values for $\sigma'_{FHs} = 0.05$ (reliable wave climate) and $P_f = 0.05$ (see Ch.10, normal safety class and ULS). This gives:

- for sliding $\gamma_H = 1.3$, $\gamma_Z = 1.4$
- for overturning $\gamma_H = 1.7$

a. Sliding

The wave loads F_H and F_U are calculated using Goda's formula, with $H_{max} = 1.8 \cdot 1.3 \cdot 6.45 = 15.09$ m. For the friction coefficient the characteristic value $f = 0.6$ is taken.

Applying the bias factors U_H and U_V as given in Ch.7 the design equation reads:

$(22.9\ B \cdot 10^4 \cdot 3.30\ B \cdot 10^4)\ 1.4 - 1 \cdot 0.6 - 0.9 \cdot 233.1 \cdot 10^4 > 0$

from which follows: $B \geq 25.9$ m

b. Overturning

The wave induced moments M_H and M_U are calculated with Goda's formula, now with $H_{max} = 1.8 \cdot 1.7 \cdot 6.45 = 19.74$ m.

Applying the bias factors U_{HM} and U_{VM} as given in Ch. 7 the design equation reads:

$(22.9\ B \cdot 0.5\ B - 0.72 \cdot 6.63\ B \cdot 0.67\ B) \cdot 0.81 \cdot 302.9 \cdot 7.94 > 0$

from which follows: $B \geq 15.3$ m.

The caisson width is determined by sliding stability at the value $B = 25.9$ m.

(iv) Deterministic design

The deterministic design gives for the same conditions and a standard safety factor $SF = 1.2$ a caisson width $B = 17.9$ m , also with sliding stability being determining. The return period of the design wave condition is taken at 50 years, but the actual failure probability of this design is not known. It is interesting to assess this P_f value, using the partial safety factor approach. This is done by substituting the width in the design equation, as follows:

$G = (22.9 \cdot 17.9 - 0.77 \cdot 4.29 \cdot 17.9)\ \gamma_Z^{-1} \cdot 0.6 - 0.9 \cdot \gamma_H \cdot 233.1 > 0$

$210 \cdot \gamma_Z^{-1} - 210\ \gamma_H > 0$

By fitting the γ_H ,γ_Z values from Table 7-2 in this expression one finds that the failure probability is in any case higher than 1.4. If we assume $P_f = 0.50$ (with $\gamma_H = 1.0$ and $\gamma_Z = 1.0$), this means P_f is about 10 times higher than what is aimed for in the design based on partial safety factors.

(v) Discussion

A first reaction to the above result could be that the application of partial safety factors leads to larger caisson width and thus to higher costs. This would be a wrong conclusion. What the example does point out is that the conventional design method with a safety factor of 1.2 has a probability of failure that is higher than what is recommended in Chapter 10 of this report. This is not surprising when one realizes that the 50 years wave height has a probability of occurence during a 50 year economic lifetime of P = 0.64, which is very high. With conventional design we do not know how this figure is translated into the failure probability, while the presented system of partial safety factors allows the designer to make a quantitive assessment of this. Based on this assessment the designer will have to decide which level of failure probability is chosen in his or her project, also taking into account the costs related to loss of function due to failure and the costs of repair.

INTERNATIONAL NAVIGATION
ASSOCIATION

ASSOCIATION INTERNATIONALE
DE NAVIGATION

PIANC General Secretariat
Graaf de Ferraris-gebouw – 11th floor
Boulevard du Roi Albert II 20, B.3
B-1000 Brussels
BELGIUM
http://www.pianc-aipcn.org
VAT/TVA BE 408.287.945

Front c
The violece of a storm puts enormous pres
on breakwaters and other maritime infrastru

ISBN 2-87223-139-0