FINAL REPORT

VOLUME IIc
STRUCTURAL ASPECTS

Edited by R.S. Crouch
April 1999

co-sponsored by
Commission of the
European Union
Directorate General XII
under
MAST contract MAS3-CT95-0041
(1996-1999)
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CHAPTER 1: INTRODUCTION

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This volume is part of the final report of the MAST III project PROVERBS, PRObabilistic
design tools for VERtical BreakwaterS (February 1996 – January 1999) under contract
no. MAS3-CT95-0041. The various parts of the final report are as follows (this volume in
bold letters):

- **Volume I**

- **Volume IIa**
  ALLSOP, N.W.H. (ed) (1999): Probabilistic design tools for vertical break-
Universität Braunschweig, Braunschweig, Germany, 400 pp.

- **Volume IIb**
breakwaters – Geotechnical aspects. *MAST III – PROVERBS – project*. Technische
Universität Braunschweig, Braunschweig, Germany, 250 pp.

- **Volume IIc**
  CROUCH, R. (ed) (1999): Probabilistic design tools for vertical break-
waters – Structural aspects. *MAST III – PROVERBS – project*. Technische
Universität Braunschweig, Braunschweig, Germany, 140 pp.

- **Volume IIId**
  VRIJLING, J.K.(ed) (1999): Probabilistic design tools for vertical breakwater-
s – Probabilistic aspects. *MAST III – PROVERBS – project*. Technische
Universität Braunschweig, Braunschweig, Germany, 170 pp.
Vertical breakwaters constructed from cellular reinforced concrete caissons can provide excellent performance and long service as part of a coastal structure, provided care is taken not only in the design and construction phases but also in the development of a properly managed maintenance plan. Therefore, within this volume the following issues are addressed:

- identification of the limitations of existing design methods when applied to vertical breakwaters and identification of the basis for a unified European design approach;
- development of improved methods for the specifications of loads and structural response during transportation and placing of caissons;
- development of improved methods of analysis to determine the structural response of reinforced concrete caissons under extreme wave impact loading;
- development of improved methods of analysis for the structural response to assess the resistance to long-term fatigue and durability issues related to reinforced concrete structures in a marine environment.

The following four Chapters provide a synthesis of observations pertinent to the structural design and maintenance of the reinforced concrete caissons. They have been produced by a team of five institutes from 3 European countries who altogether formed the Task 3 group of PROVERBS. The results are summarised in Chapter 4 of Volume I.

**Chapter 2** reports on the problems currently faced by engineers when designing a reinforced concrete caisson structure. An overview of some existing design codes is given and it is revealed that no single code of practice covers all aspects relevant to the sizing of structural elements in a marine environment under severe wave impact. Four codes are examined in some detail and omissions highlighted. In particular, it is shown that no clear guidance exists for determining appropriate wave heights when checking for serviceability and ultimate limit states in the structural members. An example calculation is given to determine the quantity of steel required to reinforce a perforated caisson using two alternative design criteria. It is shown that inconsistencies arise in manner in which the loads should be factored. This Chapter goes on to suggest a possible framework for a new code (based on the Eurocode philosophy) specifically for breakwater structures. Some practical observations are also made on constructability and placing of the caisson.

**Chapter 3** proposes a series of methods which may be used to analyse the response of a cellular caisson during the float-out and sink-down construction phases. Both the floating stability and damage caused by global flexure are considered and limit state equations proposed for a range of circumstances. The analysis methods include the use of Finite Element approaches to model the structure and the means of accounting for the stochastic variations of wind and wave loadings are given. The effects of un-even foundation bed preparation are explored and the pressures acting on caisson box structures during towing examined. This part of the report represents the first attempt at providing a systematic approach to treating the behaviour of caissons prior to placing. It should be remembered that this phase probably subjects the structure to far greater distress (in terms of loading) than it under-goes in later stages of its life.
Chapter 4 starts with a general review of the stages involved in the structural design of a multi-celled caisson and then goes on to describe some of the key elements along with a description of their load-transfer role. A series of possible failure mechanisms are identified next. Because each caisson structure is unique, it is difficult to provide useful generalisations on the structural response. Nevertheless, a series of highly simplified models are offered as preliminary design tools for the practising engineer. Although geotechnical engineers are justified in treating the breakwater essentially as a rigid body when examining its susceptibility to sliding or rotation, the structural is forced to quantify the deflections in the walls such that the section thicknesses and percentage of reinforcement may be properly designed. This Chapter shows how the maximum moments and shear forces acting in the front wall of a caisson breakwater may be determined on the basis of an equivalent static analysis. A 3-degree of freedom transient dynamic model which includes deflection of the front wall is described and the algorithm given. This forms the simplest idealisation for a dynamic model. The Chapter goes on to describe the benefits of using a FE layered shell formulation when analysing the cellular structure and finally a full three-dimensional continuum approach. The latter includes a discussion on how the fluid domain may be coupled to account for the added mass and damping effects. The manner in which fracturing in the concrete and yielding of the steel reinforcement is described. In this respect, more research work is required before truly robust, accurate and efficient constitutive models are found in mainstream FE codes. The Chapter reports on some new methods of modelling concrete right up to the point of total collapse. The Chapter also describes a novel treatment for the dynamic far-field to allow accurate modelling of the radiation damping condition. This method is based not on the use of infinite elements, transmitting boundaries or boundary elements but a highly accurate cloning approach.

Through the use of FE approaches, the degree of realism offered by the simplified techniques may be assessed although more work is required on gathering full scale field trials to confirm that the physics has been properly captured.

Chapter 5 reports in the problems associated with the long-term performance of reinforced concrete structures operating in an aggressive marine environment. The mechanisms of chloride penetration and carbonation (leading to corrosion of reinforcement) are described. It is stressed that degradation is a progressive process and careful diagnosis should be made prior to making decisions about remedial actions. This last Chapter goes on to describe the basic repair strategies, including patch repair and crack stitching, application of coatings and sealants and the use of electro-chemical techniques. In this Chapter an overview of patch repair materials (both cementitious based and epoxy based) is given. The electro-chemical methods discussed cover not just cathodic protection but also re-alkalisation and de-salination as well as the use of migratory corrosion inhibitors.

It is hoped that these Chapters provide coastal engineers with sufficiently practical information (as well as a taste of recent advances in State-of-the-Art modelling techniques) to
assist in the design and maintenance of caisson structures. Each Chapter has a list of references which point the reader to more detailed information.
CHAPTER 2: STRUCTURAL DESIGN OF VERTICAL BREAKWATERS
LIMITATION OF CURRENT PRACTICE AND EXISTING DESIGN CODES

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1. REINFORCED CONCRETE IN THE MARINE ENVIRONMENT

Vertical breakwaters involving the use of reinforced concrete appeared at the beginning of the 20th century (Franco, 1994; Tanimoto & Takahashi, 1994), more than half a century after the first application. The search for fast, economical construction methods that enabled contractors to avoid the contingencies of ocean and weather conditions naturally led to a preference for breakwaters built in prefabricated sections, and techniques came to be developed for using reinforced concrete caissons, particularly in Italy, Spain and Japan.

The most impressive applications in the area of marine construction are probably the giant offshore platforms used in the oil industry, of which the “Ekofisk Center” built between 1971 and 1973 was the pioneer, with 75 000 m³ of concrete; the “Heidrun” platform, held by tensioned cables, and the semi-submersible “Troll Olje” platform in Norway. More recently, the concrete barge “Nkossa” built in France, involved high-performance concrete with a compressive strength of 70 MPa and over.

Thus, after more than a century and with millions of cubic metres of concrete having been used in all kinds of contemporary structures, considerable knowledge has been amassed, notably since the first systematic experiments carried out by Pier-Luigi Nervi (Nervi P.-L., 1951). This experience has led to the formulation of many specific codes, regulations and standards adapted to the field they intend to cover and corresponding to the traditions and requirements of individual countries, with successive editions clearly showing the changes brought about by improved knowledge and greater international exchange.

Codes and regulations are generally concerned with:
- The ways in which the material is to be used, on the basis of past experience,
- The actions on the structures, depending on their intended use,
The effects of actions on the materials, depending on the level of safety required.

There is no particular difficulty involved in applying the verification procedures described above to most structures built on land. But the task is not so easy for marine structures.

2. **CHANGES IN CODES VERIFICATION FORMATS**

As the design of a structure is always part of a contractual link between a designer and an owner, specific codes, regulations and standards set down to a certain extent the principles and methods with which the designer is required to comply. The consequence for the designer is that he is almost always obliged to enter into a prescribed verification format that directs him with varying degrees of flexibility towards the model that he must use to forecast the effects (E) produced by actions (F) and compare them with the response capability (R) of the material that forms each member, so that

\[ E(\sum \gamma_f F) \leq R \]

allowing for a predetermined safety margin for all foreseeable situations.

In Europe, a major new development began at the end of the 1970s, with the progressive substitution of the traditional “permissible stress” methods by semi-probabilistic methods in the rules for checking structural safety. The principle of this method, which is recommended most notably in the Eurocodes, is to show that the combinations of actions and likely design stresses do not result in the structure or any of its parts reaching a Limit State, i.e. one of the phenomena that one wishes to avoid.

For example, in the case of a material such as reinforced concrete, which itself consists of two associated materials, concrete and steel, the characteristic strengths of which are “f_y” and “f_c28”, and by introducing the safety factors “\( \gamma_s \)” and “\( \gamma_b \)” (both > 1), the previous inequality becomes:

\[ E(\sum \gamma_f F) \leq R(f_{c28}/\gamma_b, f_y/\gamma_s) \]

in which:

- “\( \gamma_f \)” is a safety factor assigned to the actions themselves, and
- “\( \gamma_s \)” and “\( \gamma_b \)” are safety factors applying to the materials, which the designer cannot alter, as he does not know their individual origins.

The following main distinction is made:
1) ULTIMATE LIMIT STATES (ULS), which, if exceeded, would result in the destruction of the structure, through loss of static equilibrium, mechanical strength, shape stability, etc.

2) SERVICEABILITY LIMIT STATES (SLS), which, if exceeded, would result in a malfunction that would jeopardise the intended use of the structure from the point of view of strength, sensitivity to the ambient medium, strain levels reached, etc.

These various Limit States are represented by the following:

a) A set of combinations of actions, each weighted by one or more safety factors specific for the Limit State under consideration.

For example, in ULS:  
\[
S(\Sigma q_i A) = \gamma_{Gm} \cdot G_{\text{max}} + \gamma_{Gm} \cdot G_{\text{min}} + g_{Q1} \cdot Q_1 + \Sigma \gamma_{Qi} \cdot \varphi_{Qi} \cdot Q_i
\]

in which:
- \(G_{\text{max}}\) is the set of unfavourable permanent actions
- \(G_{\text{min}}\) is the set of favourable permanent actions
- \(Q_1\) is a basic variable action
- \(Q_2, Q_3, ..., Q_i\) are other so-called “accompanying” variable actions

b) A set of design properties for the materials being used, each one weighted by one or more specific safety factors for the limit state under consideration.

For example:
- \(f_{c28}\) (concrete) weighted by \(\gamma_b=1.15\) for accidental actions or 1.50 for other cases;
- \(f_y\) (steel) weighted by \(\gamma_s=1.00\) for accidental actions or 1.15 for other cases.

c) Individual specifications for the calculation models used, based on a comparison of mechanical stresses and resultant displacements with the values specified in the reference standards.

For example, in ULS:  
\[
\{ \sigma_{bc} \leq 0.85 \cdot f_{c28} / \gamma_b ; \varepsilon_s \leq 10^\circ/\text{mm} \}
\]

in which:
- \(\sigma_{bc}\) is the compressive stress of the concrete
- \(\varepsilon_s\) is the elongation of the steel

Let us remind that the response of a reinforced concrete structure under the effect of the various actions is generally carried out by designers in two stages for each Limit State:

1. Firstly, they perform an analysis of the entire structure assuming reinforced concrete as an homogeneous material, in order to determine the distribution of forces and first-order by using a linear behaviour theory.
2. Secondly, using the first step results, they check each component of the structure, section by section, in order to determine the strength at each point, but taking into consideration a non-linear behaviour for the reinforced concrete (simplified stress/strain diagrams of rectangular or parabolic-rectangular shape).

3. LIMITATIONS FOR COASTAL STRUCTURES OF EXISTING CODES

Applying the verification procedures described above to marine structures is not so easy as:

1. There appears to be no document giving a consistent overview of all the parameters required to design coastal structures made of reinforced concrete,

For example, to cover all the aspects of a reinforced concrete coastal structure with reasonable safety, the design rules adopted for the Port d’Hercule at Monaco (Isnard, J.-L., 1995) were drawn from the following regulations:

- French: Fascicule 62-V of the Technical Specifications for French State contracts for the foundation works; AFPS 90 for seismic activity; BAEL 91 for the reinforced concrete; BPEL 91 for the prestressed concrete and Bureau Véritas rules for the maritime aspects.
- American: API RP2A - LRFD for the foundations
- Norwegian: Standard NS 3473 E “Concrete structures” for the reinforced concrete; DnV Classification Notes no. 30.4 – “Foundations” for the foundations.

2. The existing recommendations for designing coastal structures give no precise indications concerning the characteristic values of the hydrodynamic actions to be introduced into each Limit State.

By default, many publications relating to hydrodynamic actions were compiled but this did not provide any usable information since these works generally concentrate on evaluating extreme phenomena, and the importance of these is quite relative when sizing reinforced concrete structures, as will be seen later.

3. Even if the partial coefficients recommended in the codes and regulations are calibrated on a probabilistic basis with reference to the working life, this is implicitly “indeterminate” as, with the exception of fatigue of the various assemblies, no coefficient or limit depends on a working life and even when a working life is specified, this does not alter the general design rules. Certain modern regulations (ROM, BSI) indeed refer to the working life but no factor ever includes it (with the exception of fatigue).
Thus, the Durability Limit State introduced by certain recommendations as one of the Serviceability Limit States is for the most part impossible to calculate and the Fatigue Limit State often mentioned as one of the Ultimate Limit States is rarely considered owing to the lack of precise recommendations adapted to coastal reinforced concrete structures. When occasional checks are carried out, they use Miner’s cumulative damage theory (subject to a few approximations), which is widespread in the offshore industry but scarcely known in the area of coastal structures.

4. PRACTITIONERS CONCERNS FOR A DURABLE DESIGN

A marine structure built in the open sea is by definition in an environment that produces permanent dynamic stresses, particularly of a cyclic nature (weak but regular stresses, say a cycle of varying intensity every 4-6 seconds to give a rough idea). At certain seasons, there may be a few brief series of rapid, high-intensity impacts. Simultaneously, this same environment has a chemical action on the materials forming the structure, extremely pronounced in the case of both concrete and steel reinforcements, and this type of action gets worse with time. Thus, practitioners have to prevent against mechanical and chemical deterioration linked with exposure time.

It is in fact the constructional arrangements or “good engineering practice”, to use the normal English expression, that become of paramount importance. Their drawback is that they are essentially empirical and difficult to express as a series of formulae.

The more mechanical aspect of durability, in the strength of materials sense, is that reinforced concrete functions by definition in a so-called “cracked” state, since the reinforcements are only there to absorb the traction that the concrete cannot withstand. This cracking under loading must therefore be kept to a minimum so as not to expose the reinforcements to corrosion. In practice, this means that cracking is limited to a conventional value depending on the country in question, or that traction in the steel reinforcements is limited to a determined value, which is tantamount to the same thing. Another means of controlling cracking is of course to avoid bending and to absorb forces as much as possible by compression.

In vertical face breakwaters made of cellular sand-filled caissons, bending of the front wall is avoided by having it bear on the inner walls, and in particular by filling the cells with a powdery material that has a high internal friction angle. In absorbent breakwaters, as indeed in concrete oil platforms, arc-shaped forms are used as much as possible in the horizontal plane in order to limit bending effects, as the geometry of an arc transmits a large proportion of the loads that it supports in the plane in the form of compression forces in the sections under most stress.
Without going into detail about the purely physico-chemical aspects of durability, which have been analysed in a remarkable work by Comité Euro-International du Béton (CEB, 1992), and govern mainly the composition of reinforced concrete for marine use, it should be borne in mind that attempts to achieve this type of durability nearly always lead to higher mechanical resistance than for “on-land” concrete, owing to the compactness being sought, and another resultant mechanical effect is the need to cover the steel reinforcements with about twice as much concrete as in the case of “on-land” structures.

Thus, even if the concept of design working life is hard to define explicitly, durability, and consequently the associated Fatigue and Durability Limit States, can be controlled essentially by:

- a suitable concrete mix,
- proper constructional arrangements (assemblies),
- limitation of cracking under loads.

5. PREVAILING PHENOMENON: DURABILITY OR EXTREME FORCES?

From the point of view of structural design calculations, the consequence of the above considerations is that when limiting cracks opening is an essential criterion, and unless the loads taken into account in each case differ to a considerable extent, the sections of each member will not be determined by an extreme event (or a resistance Ultimate Limit State) but by a more weak and repetitive action (part of a Serviceability Limit State).

For example, let us consider the particular case of the front wall of an absorbent caisson, such as that shown in the following Figure 1, when it is subjected to wave attack. Wave-induced pressures are calculated using Goda’s theory, in which a reflection coefficient is introduced on the basis of scale-model tests. Various height/period pairs were defined for the project site in question, corresponding to return periods of between 1 and 100 years (1, 5, 10, 20, 50, 100). The Goda pressures were then calculated, as shown in Figure 2.
Figure 1: Wave absorbent, reinforced concrete caisson
Figure 2: Design wave pressure

The corresponding forces, calculated for a vertical section 1 m wide, are:

Table 1: Design wave forces for a vertical section 1 m wide

<table>
<thead>
<tr>
<th>Return period (Years)</th>
<th>Horizontal force (x10^4N)</th>
<th>Under pressure (x10^4N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>66,8</td>
<td>30,7</td>
</tr>
<tr>
<td>5</td>
<td>111,3</td>
<td>54,7</td>
</tr>
<tr>
<td>10</td>
<td>152,5</td>
<td>77,8</td>
</tr>
<tr>
<td>20</td>
<td>161,7</td>
<td>83,0</td>
</tr>
<tr>
<td>50</td>
<td>206,3</td>
<td>104,3</td>
</tr>
<tr>
<td>100</td>
<td>232,5</td>
<td>118,8</td>
</tr>
</tbody>
</table>

In this particular case, the wave considered for the Serviceability Limit State is a 10-year wave and that considered for the Ultimate Limit State is the 100-year wave. These choices are quite arbitrary but were discussed at length with the owner, and a very serious cracks opening criterion was of course adopted.
The loads involved in the calculations and the various combinations used for each Limit State will not be discussed in detail here. One must simply ask the following question:

What is the wave return period that determined the size of the front wall:
1. that associated with the Serviceability Limit State “SLS(10years)” or
2. that associated with the Ultimate Limit State “ULS(100years)”?

For the needs of this work, while at the same time keeping the thickness of the front wall constant, we repeated all the reinforcement calculations introducing successively:

1. the 1-year, 5-year, 10-year and 20-year waves in all the SLS combinations and
2. the 50-year and 100-year waves in all the ULS combinations,

Because of the complexity of the stiffening system and perforations provided to absorb part of the wave energy, the calculations were run with a finite-element program featuring a reinforced concrete post-processor capable of processing load combinations in SLS and ULS formats and containing algorithms for calculating sections in Limit States. Provided no change is made in any of the parameters governing the stiffness matrix, which is inverted and stored once and for all, and modifying only the loading, many simulations were carried out very quickly. In order to apply the reinforced concrete calculation format at the Limit States, we introduced into the load combinations the wave forces factored by:

1. $\gamma_{Q1} = 1.0$ at SLS and
2. $\gamma_{Q1}=1.5$ at ULS.

In terms of the quantity of steel produced for a constant wall thickness, this first set of calculations drive to the results given in Table 2:
Table 2: Wave forces ratios versus rebars quantities ratios

<table>
<thead>
<tr>
<th>Wave return period (years)</th>
<th>Limit State</th>
<th>( \gamma_{Q1} )</th>
<th>Wave Force Ratio (WFR) (Wave force divided by the 10 year wave force)</th>
<th>( \gamma_{Q1} \times \text{WFR} )</th>
<th>Rebars Qty. Ratio (RQR) (Required rebars qty. divided by the SLS(_{10\text{years}}) rebars qty)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SLS</td>
<td>1.00</td>
<td>0.44</td>
<td>0.44</td>
<td>0.4</td>
</tr>
<tr>
<td>5</td>
<td>SLS</td>
<td>1.00</td>
<td>0.73</td>
<td>0.73</td>
<td>0.7</td>
</tr>
<tr>
<td>10</td>
<td>SLS</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.0</td>
</tr>
<tr>
<td>20</td>
<td>ULS</td>
<td>1.50</td>
<td>1.06</td>
<td>1.06</td>
<td>1.1</td>
</tr>
<tr>
<td>50</td>
<td>ULS</td>
<td>1.50</td>
<td>1.35</td>
<td>2.03</td>
<td>0.5</td>
</tr>
<tr>
<td>100</td>
<td>ULS</td>
<td>1.50</td>
<td>1.52</td>
<td>2.28</td>
<td>0.9</td>
</tr>
</tbody>
</table>

It can easily be deduced that for an Ultimate Limit State with a 100-year wave:

1. the overall applied force is \( \gamma_{Q1} \times \text{WFR} = 1.50 \times 1.52 = 2.28 \) times greater than the force produced by a 10-year wave introduced in a Serviceability Limit State \( (\gamma_{Q1} \times \text{WFR} = 1.00 \times 1.00 = 1.00) \),

2. but that it requires only \( \text{RQR} = 90\% \) of the rebars calculated for this SLS.

Which can been otherwise expressed as: “The 100 year wave force is 2.28 greater that the 10 year one, but requires 10\% less reinforcement (at constant thickness)”

Determining sections of reinforced concrete from a force generated by a 100-year wave multiplied by 1.50 has little obvious connection with reality, since the event likely to generate such a force has a return period of much more than 100 years. But as this figure generates less rebars quantity than the SLS\(_{5\text{years}}\) condition, it is evident that factoring the ULS\(_{100\text{years}}\) condition by \( \gamma_{Q1} = 1.0 \) instead of 1.5 will produce even less reinforcement for the same extreme event.

It is thus clear that, for this example, the reinforced concrete front wall is not sized for an extreme event associated with a resistance criterion, but for a more frequent event associated with a durability criterion, since in the hypothesis that the 100-year wave is introduced into a resistance Ultimate Limit State, the “equal-sizing” wave that would be required in the Serviceability Limit State with a serious cracks opening criterion would have a return period of between 5 and 10 years.
Without generalising the above result, of course, it is certain that this is a subject for careful research to evaluate the probabilities of cumulative damage caused by cracking under wave loading, in order to recommend members that are compatible with the semi-probabilistic structural verification format.

6. OTHER POORLY DEFINED PARAMETERS

In certain Limit States, the condition to be checked is a maximum displacement (or deformation) under a determined set of loads.

For example, what is the tolerable horizontal displacement for a caisson subject to a 100-year wave?

In the case of a 15 m wide breakwater (which has no other function and has water on either side), and assuming that the caissons are not connected to one another, what is the loss of performance generated by a simple 0.25 m horizontal displacement of one of the caissons under an extreme wave attack? Probably none, and it is for this reason that in the case quoted here, the owner accepted that sliding was tolerable within a limit of 2% of the width for the juxtaposed unconnected caissons, simply for aesthetic and psychological reasons.

If displacement is acceptable under extreme wave conditions, then a few modifications should be made in the conventional rules of stability, which determine the weight of a caisson only on the basis of sliding and overturning criteria on the assumption that it is independent of its neighbours.

If the caisson is connected by vertical keys to its neighbours, the same conventional rules are no longer meaningful, as the entire breakwater reacts and its stability cannot be reduced to what happens in the vertical plane without introducing a force equivalent to the support offered to each member by its neighbours.

Normal practice often involves providing keys that transmit horizontal forces from one member to another, so that the actual horizontal layout of some structures gives them a certain overall cohesion enabling them to distribute forces along their axes. Although this linking is very often found in finished structures, it seems to be largely ignored since the overall stability is very often reduced to that of a single isolated caisson or of a single vertical “section” in the corresponding design calculations.

A simple calculation model, similar to that used for a continuous beam resting on an elastic medium, should be enough to evaluate the degree of continuity or overall cohesion of structures under a live load moving along their axes. In this respect, the overall Stability Limit
State becomes a three-dimensional problem of fluid-soil-structure interaction. However, while wave action in the form of pressures on a vertical facing is relatively well documented, pressure distribution along a breakwater is less easy to determine when attempting to define three-dimensional loading to evaluate the overall response of the structure.

We saw above that the distribution of forces in a vertical plane is calculated using Goda’s theory (possibly confirmed by physical model tests). Implicitly, this means that the force is constant perpendicular to the computational section (see Figure 4). This assumption is probably extremely conservative when sizing many structures, the centre line of which is not parallel to the wave front.

![Figure 4:](image)

For example, let us imagine a breakwater with a curved layout in the horizontal plane (see Figure 5). Intuitively, it is unrealistic to consider that the entire structure will support the same pressure along its centre line. In fact the distribution is at least out of phase, depending on the angle between the structure and the orthogonal to the direction of wave propagation.
Figure 5:

How far is it possible to define a set of spatial pressure fields corresponding to time steps of one or more wave periods, as is done in designing oil platforms?

7. EXAMINATION OF MOST RELEVANT EXISTING CODES

7.1. Expectations of designers when using codes

It is normally the Engineer's duty to qualify and whenever possible quantify the degree of uncertainty in designing, constructing and exploiting structures, in association with Clients, in order to provide them with technical decision-making instruments. These are to be combined with economic, sociological and political ones, so as to reach decisions that are motivated as far as possible by a concern for human welfare and the preservation of nature. This being so, and in order to keep actions possible, the principle of precaution may be attenuated in this way:
- be careful : inform oneself, search for scientific and technical information and, in case of doubt, wait whenever possible until the “running-in” period and necessary experimentation have been completed,

- be vigilant : develop evaluation procedures and associated means : feedback, benchmarks, comparisons,

- be flexible : favour rapid reactions in organisations and arrange possibilities of backspacing, in order to benefit from new knowledge and be able to modify the project and works as far as possible.

Any Code should give emphasis on above steps and provide as far as possible à consistent set of parameters to help the Engineer in his task.

7.2. Codes examined in the context of the “PROVERBS” project

Examining the existing codes is an essential step in any attempt to determine the consistency of design methods. This is why the tasks undertaken in the context of the MAST “PROVERBS” project aim to evaluate the contents of a number of codes of practice and recommendations, not only to investigate the bases of the methodology that they propose individually and to draw on the experience gained in the different formulations suggested, but also to put forward results in a form that is directly compatible with established practices.

Five of the codes suggested were studied with the aim of identifying the way in which they deal with the specific features of reinforced concrete at sea. Reinforced concrete is virtually the only material used in constructing vertical face caisson breakwaters. Three Codes deal more particularly with the material and safety verification formats, and two recommendations specific to coastal structures fix criteria for evaluating sections. Four are European documents and the United States “ACI” is examined on account of its international use for reinforced concrete in many “export” projects. For reference, these codes are listed below.

1. Building Code Requirements for Structural Concrete (ACI 318-95), U.S.A.
2. CEB-FIP - Model code for concrete structures - 1978
4. BS 6349 - British Standard Code of practice for Maritime structures, Pts 1, 2 & 7 (U.K)
5. Maritime Works Recommendations - Actions in the Design of Maritime and Harbour Works (ROM 0.2-90 - Spain)
7.3. The three different levels of a code

Probabilistic techniques for the purpose of formulating a design/verification code usually consist in:

- Defining the system being studied (in itself, especially its desired functions and its composition, and with respect to its environment),
- Identifying parameters governing the behaviour of the system, from experimental knowledge as most of following steps,
- Separating these into parameters that are favourable to the safety of the system (in general, resistance) and those that are unfavourable (in general, actions),
- Separating relevant parameters from less relevant ones and uncertain parameters from less uncertain ones,
- Identifying interactions effects between parts of the system and attempting to quantify interactions that contribute to both greater and reduced safety,
- Identifying risks connected with use (and construction, as the case may be),
- Building model that are as accurate as possible to represent the behaviour of the system (or parts of it) when in risk situations and quantifying model errors in terms of bias and dispersion related to a selected confidence level,
- Determining standard and a priori uncertainties of parameters as above,
- Assuming a given degree of workmanship and in-service inspection,
- Assuming a target level of safety,
- Determining achieved levels of safety,
- Confirming assumed influences of parameters and, if needed, modifying models,
- Calibrating with respect to existing recognised non-probabilistic codes of safety and defining appropriate verification formats,
- Highlight areas where uncertainties must be reduced in order to fulfil safety criteria and quantifying necessary gains in certainty,
- Obtaining new knowledge (for methods) and data (for parameters) if simple modifications cannot be made to particular features of projects to fulfil safety criteria and, if needed, defining insurance requirements beyond those (mainly legal) which correspond to normal compliance with a recognised code. The first alternative is often the only one possible when requalifying existing systems which cannot be economically upgraded.

In practice, this results in following three levels, of which the first one may remain implicit:

1. **Background level**
   - target risk (individual / societal)
   - uncertainties (biases / dispersions)
   - design / verification format (level 1 = LRFD)
- calibration process

2. Risk analysis level
   - situations
   - limit-states
   - combinations

3. Engineering level
   - actions
   - models (methods)
   - resistance

7.4. Summary of contents of each code

After the general considerations given above, the following tables set out a summary of the detailed analysis work performed on each Code, which we hope is sufficiently concise. The detailed analysis may be consulted by contacting the Coordinator of the MAST “PROVERBS” project.

<table>
<thead>
<tr>
<th>TARGET RISK</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
</tr>
<tr>
<td>CEB-FIP</td>
</tr>
<tr>
<td>Ecs</td>
</tr>
<tr>
<td>BS</td>
</tr>
<tr>
<td>ROM</td>
</tr>
</tbody>
</table>
### SITUATIONS

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>No explicit consideration of situations (all assumed permanent if earthquake is excluded)</td>
</tr>
<tr>
<td>CEB-FIP</td>
<td>This code defines the following kinds of situation: permanent, temporary, transient and accidental</td>
</tr>
<tr>
<td>ECs</td>
<td>These Codes account for the following kinds of situation: persistent, transient, accidental and seismic</td>
</tr>
<tr>
<td>BS</td>
<td>Not explicitly considered</td>
</tr>
<tr>
<td>ROM</td>
<td>No detailed consideration but only a differentiation between Construction and Service Phases other than Service Phases in Exceptional Conditions</td>
</tr>
</tbody>
</table>

### RESISTANCES (CONCRETE)

#### Obtaining Parameters (Shear Strength)

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>The concrete shear strength is the square root of compressive strength (both in psi)</td>
</tr>
<tr>
<td>CEB-FIP</td>
<td>The concrete shear strength is not specifically indicated but may be considered as 0.25 x tensile strength</td>
</tr>
<tr>
<td>ECs</td>
<td>EC2 indicates 0.25 x tensile strength</td>
</tr>
<tr>
<td>BS</td>
<td>Expected in British concrete code</td>
</tr>
<tr>
<td>ROM</td>
<td>Reference to Spanish concrete code</td>
</tr>
</tbody>
</table>

#### Workmanship (Concrete Cover)

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>In this code, it depends on the method of casting the concrete, the type of exposure, the size of rebars, the type of structural components, with a maximum of 2 in., even 3 in.</td>
</tr>
<tr>
<td>CEB-FIP</td>
<td>Three exposure levels: slightly, moderately, highly aggressive with basic values of cover, respectively: 15, 25, 35mm, with a maximum of 40mm, except for sea-structures</td>
</tr>
<tr>
<td>ECs</td>
<td>Considers five classes of exposure: dry, humid, humid + frost and de-icing salts, seawater, chemically aggressive</td>
</tr>
<tr>
<td>BS</td>
<td>Not in the code (expected in British concrete code)</td>
</tr>
<tr>
<td>RESISTANCES (CONCRETE)</td>
<td></td>
</tr>
<tr>
<td>------------------------</td>
<td></td>
</tr>
<tr>
<td>ROM</td>
<td>Not in the code (expected in Spanish concrete code)</td>
</tr>
</tbody>
</table>

**ACTIONS**

**Maritime aspects**

<table>
<thead>
<tr>
<th>ACI</th>
<th>Topic not addressed</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEB-FIP</td>
<td>This code gives a methodology for determining the characteristic values of variable actions. Even though wind actions are considered and swell is mentioned as a dynamic event, the code is not specific on maritime actions.</td>
</tr>
<tr>
<td>ECs</td>
<td>Topic not addressed</td>
</tr>
<tr>
<td>BS</td>
<td>Extensively discussed in part 1: meteorology, climatology, etc.</td>
</tr>
<tr>
<td>ROM</td>
<td>This Code gives an extensive list of maritime actions to be accounted for but does not indicate the characteristic values to be considered for variable environmental loads. The methodology for obtaining these parameters for waves and wind are expected in documents ROM 0.3 and 0.4, not published yet</td>
</tr>
</tbody>
</table>

**Typology**

<table>
<thead>
<tr>
<th>ACI</th>
<th>No specific typology</th>
</tr>
</thead>
</table>
| CEB-FIP | This Code makes difference between the following aspects:  
- variation in time: permanent, variable, accidental  
- variation in space: fixed, free  
- nature: static, dynamic  
- value: characteristic, service, nominal, combination, frequent |
| ECs | Specific, well defined and detailed typology in this Code which follows CEB-FIP on this topic except with regard to the nominal value concept. A particular feature: prestressing is quoted as a permanent action. |
| ROM | Specific, well defined and detailed typology in this Code which follows CEB-FIP on this topic except with regard to the nominal value concept. |
| BS | No specific typology |

**Design Life / Return Period**

<table>
<thead>
<tr>
<th>ACI</th>
<th>Does not deal with these concepts</th>
</tr>
</thead>
<tbody>
<tr>
<td>ECs</td>
<td>Only informative and methodological</td>
</tr>
</tbody>
</table>
### RESISTANCES (CONCRETE)

<table>
<thead>
<tr>
<th>Standard</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEB-FIP</td>
<td>DESIGN LIFE: indicates 5 years for temporary works, 50 years for a normal construction (reference) and 500 years for a monumental construction. RETURN PERIOD: gives an outline methodology difficult to use in practice. It considers as general: 125 - 200 years, with a maximum of 500 years, except for wind: 1000 to 10000 years.</td>
</tr>
<tr>
<td>BS</td>
<td>DESIGN LIFE: recommends 100 years for flood protection works; 60 years for shore protection work, breakwaters and quay walls; 45 years for drydocks and open jetties; 30 years for superstructure works. RETURN PERIOD: with probability of failure of 0.2, specifies 90 years and 1000 years if lower.</td>
</tr>
<tr>
<td>ROM</td>
<td>DESIGN LIFE: produces a table giving the design life, scaling from 15 to 100 years according to the safety level (3 levels) and type of installation (general / specific use). RETURN PERIOD: No specific indication on how to determine return periods for variable loads and remains methodological on this topic. More specific information expected in ROM 0.3 - 0.4.</td>
</tr>
</tbody>
</table>

### MODELS

#### Maritime aspects

<table>
<thead>
<tr>
<th>Standard</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>Not addressed</td>
</tr>
<tr>
<td>CEB-FIP</td>
<td>Not addressed</td>
</tr>
<tr>
<td>Ecs</td>
<td>Not addressed</td>
</tr>
<tr>
<td>BS</td>
<td>Very detailed material with reference to model tests and to Goda's studies</td>
</tr>
<tr>
<td>ROM</td>
<td>Not addressed, expected in ROM 0.3 - 0.4</td>
</tr>
</tbody>
</table>

#### Overall structural models

<table>
<thead>
<tr>
<th>Standard</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>Considers the following types of structural behaviour: elastic or plastic. Considered actions include static or impact loads on beams, rafts, walls, footings and shells</td>
</tr>
<tr>
<td>CEB-FIP</td>
<td>Considers the following types of structural behaviour: elastic, elastic with redistribution, plastic, second-order effects. Considered actions include static or dynamic loads on beams, slabs and plane shells.</td>
</tr>
</tbody>
</table>
### MODELS

<table>
<thead>
<tr>
<th>Models</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ecs</td>
<td>Just presents the “appropriate model” and the “established engineering theory” concepts without any indication concerning the nature of models but stresses that the method used should be verified experimentally if necessary.</td>
</tr>
<tr>
<td>BS</td>
<td>No indication concerning types of structural behaviour (expected in British concrete code) but typical constructional arrangements given.</td>
</tr>
<tr>
<td>ROM</td>
<td>No indication concerning types of structural behaviour (expected in Spanish concrete code). Mentions static, dynamic, impact and vibratory loadings but no indication concerning relevant components.</td>
</tr>
</tbody>
</table>

#### Local Structural Models

<table>
<thead>
<tr>
<th>Models</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>No explicit limit-state, but depending on local failure mechanisms: bending and axial loads effects, shear, torsion, cracking.</td>
</tr>
</tbody>
</table>
| CEB-FIP| Presents the following local failure modes:  
- **ULS**: axial loads effects (including re-bars splices); shear; torsion; punching shear or buckling  
- **As other limit-states**: cracking or deformation |
| BS     | No explicit limit-states, nor failure mechanisms. Expected to be included in British concrete code. |
| ROM    | Presents the following local failure modes:  
- **ULS**: loss of equilibrium; breakage or yield; second-order instability; fatigue; progressive collapse; cumulative deformation  
- **SLS**: lack of durability; deformation; vibration; permanent damage; permeability |
| Ecs    | Presents the following limit-states:  
- **ULS**: loss of equilibrium; failure by excessive deformation / transformation into a mechanism; fatigue or time-dependent effects,  
- **SLS**: deformations and displacements; vibrations; appearance or durability of the structure; cracking, |

### 8. KEY ISSUES WHEN USING EXISTING CODES

From the above presentation it can be seen that none of the codes examined encompasses all aspects of a caisson design. In particular no code simultaneously addresses concrete (structural) design and wave action. The gap is to be filled by formulating recommendations.
for maritime structure and by harmonising safety levels. However, simply compiling all the
most specific parts from the different codes would not result in a good code because of the
lack of homogeneity. The combination of existing rules can lead to unreasonable results in
any case.

To avoid putting forward a plethora of recommendations, it is perhaps interesting to consider
adapting what already exists. The task is not an easy one, of course, as can be seen by looking
at a few aspects.

A. Differences in the representative wave action parameters

A discrepancy was noted concerning the representative value of wave action (H). The
maximum wave height is sometimes considered for ULS, whereas for SLS there do not
seem to be clear recommendations: significant wave height, one-tenth highest wave
height, maximum wave height ? Can it be different according to the nature of the limit
state ?

B. General rules do not specify the frequency of the event to consider

The first possibility is to adopt a single event (e.g. a return period of 50 years) to be
defined with regard to the service life. The partial factor is applied to the load “H” (e.g.
the significant wave height) and will be different according to the limit state considered
(e.g. $\gamma_h \approx 1.50$ for ULS, 1.00 for SLS). This possibility represents the general safety format
for most structure design codes. However it does not seem relevant for harbour or coastal
maritime structures since :

- local propagation conditions greatly influence wave parameters ; it is questionable
  whether a single multiplication coefficient “$\gamma_h$” can allow for local conditions,

- a standard value of 1.50 applied to wave height may lead to unrealistic waves ; it is
  therefore useful to recall that 1.50 is the product of two factors dealing with different
  uncertainties. Basically it is possible to write $1.50 = 1.33 \times 1.125$ where 1.33
  represents the uncertainty inherent to the value of the variable action and 1.125 is a
  general illustration of the model uncertainty for structural limit states.

Better solutions would then be :

- either to determine of two different events for SLS and for ULS (i.e. direct
  assumption of both characteristic and design values),

- or to calibrate “$\gamma_h$” with regard to wave uncertainties only.
The same problem arises for fatigue: what Wave Fatigue Load Model (W-FLM) is to be used for designing the concrete members of a caisson?

C. The cracking model is to be adapted

With regard to cracking effects in aggressive environmental conditions during service life, how many times can the structure sustain the SLS event loading without impairing its durability too much? Should crack opening be considered as producing reversible or irreversible effects? Should the different parts of the structure be submitted to the most stringent overall consideration or not?

D. Practitioners wait for additional recommendations for structural analysis

A caisson may be modelled as a whole, comprising the bottom slab, the front wall, the partition walls, the stiffeners, ... Modelling the members separately calls for simplifying hypotheses concerning bearing conditions, which may entail significant errors. In some cases, the sign of the forces may be reversed.

When the structure of the vertical breakwater provides 3D continuity and resists the wave forces as a continuous beam resting on soil, a simplified 3D structural model with a 3D wave pressure field for various time steps will obviously be more accurate.

9. A TENTATIVE WAY TO HARMONISE THE FORMATS

It seems to be a sensible strategy to look for consistent design rules adapted to the various parts of a given structure. For instance, attention should be paid to the consistency with PIANC recommendations in the design of composite (rubble mound + vertical concrete face) breakwaters. Attention should also be paid also to the possible future development of cases “B” and “C” of the EUROCODES system and to the development of an unified case of partial factors.

Shall the designer, then:

- use different characteristic (or design) waves for structural and foundation limit states?
- use different “γ” values according to each limit state?
- use the same representative values whatever the ULS (except fatigue)?

A way to achieve consistency between different structures without going about the titanic task of redefining new codes for each of them is to separate the treatment of the uncertainties.
Therefore would it not be interesting to make some minor adaptations to the semi-probabilistic way of thinking, which considers the uncertainties comprehensively for a given limit state function?

9.1. **Representative values of parameters to be clearly stated.**

The “at source partial factors” are related to actions and materials. They should only allow for intrinsic parameter uncertainties, without any further consideration as to the limit state function. Their values are taken mainly from existing codes or regulations without proper use of scientific calibration procedures.

The “at source factors” are applied to the relevant parameter directly at the beginning of the calculation process: the input parameters of the model are the factored parameters.

Rules for determining representative concrete values are already given in EUROCODE 2, with partial factors $\gamma_{b} = 1.50$ for concrete and $\gamma_{s} = 1.15$ for steel. Recommendations for determining representative soil parameter values are already given in EUROCODE 7.

As far as waves are concerned, the designer will:

- either determine the characteristic wave from wave data and then calculate the design wave force by using a specific partial factor,
- or determine directly the characteristic and design values from available wave data.

It seems more appropriate for the statistical uncertainty (number of waves) to be included when determining the characteristic value of the wave parameter.

9.2. **The model factors should be developed**

The “model factors” are introduced in the limit state function at the last stage in the verification process. They differentiate between safety levels according to the limit state and allow for:

- the discrepancy between model and reality,
- the required safety level,
- the design working life.

The model factors are to be calibrated once the at-source factors are given, using probabilistic procedures. Their values depend on predetermined safety levels assessed by National
Regulation Authorities. According to the EUROCODES, the model factor can be split into an “action model factor” $\gamma_{sd}$ and a “resistance model factor” $\gamma_{rd}$. However, for the sake of simplicity, it is proposed here to merge them into one factor $\gamma_d$. The canonical expression of the limit state function, which contains no exception for vertical breakwaters, could then be written:

$$\gamma_d \cdot E(\gamma_g G_k + \gamma_h H_k) \leq R(X_k / \gamma_M)$$

where:

- $\gamma_d$ : model factor of the limit state
- $\gamma_g$ : predetermined at-source factor for permanent actions
- $\gamma_h$ : predetermined at-source factor on wave
- $\gamma_M$ : predetermined at-source factor on material
- $G_k$ : characteristic value of permanent action
- $H_k$ : characteristic value of wave
- $X_k$ : characteristic value of material parameter (concrete, soil)
- $E(.)$ : effect of action (solicitation)
- $R(.)$ : structural (or foundation) resistance

10. FRAMEWORK FOR THE DESIGN OF VERTICAL BREAKWATERS

10.1. Introduction

After having identified the main inconsistencies between previously analysed design codes, we propose hereafter an unified framework of a code of practice for the design of solid and perforated vertical breakwaters.

This work is intended to identify the key items to be added to existing codes or regulations. Based on the general Eurocode’s format, this document obviously needs to be completed by adequate references to applicable national Concrete and Soil Regulations.

Two columns are introduced in the following tables: proposal of the main drafting items and examples, proposals of writing and references to Eurocodes.
### 1. GENERAL
(Matters concerning rubble mound armour and protection, though part of some vertical breakwaters, are not dealt with here.)

### DESIGN WORKING LIFE
The design working life of vertical breakwaters must be stated for the verification of durability and fatigue limit states. It can also be taken into account in the determination of the characteristic values of the environmental actions.

The design working life can be generally taken equal to 100 years.

### DESIGN SITUATIONS
Following design situations are generally defined:
- One permanent situation referring to the normal exploitation of the breakwater under various environmental conditions,
- as many transient situations as deemed necessary by the construction stages; for instance: transportation and towing, lifting of precast caissons, sinking, stability of rubble bedlayers ...
- some accidental situations according to the local conditions; for instance: earthquake (when not a variable action), tsunami, ship collision, accidental scour, accidental wave load ...

Design situation may be multiplied when taking into account:
- soil behaviour (long term and short term resistance, consolidation of hydraulic inner fill ...),
- geometrical properties influenced by erosion and scour (slope angle),
- flow and ebb water levels (for tidal sites),
- current conditions.

### SAFETY LEVELS
The required safety level, including durability, is ensured by:
- calculations using limit states conditions, characteristic values and partial factors,
- adequate workmanship and specific constructions arrangements.

For vertical breakwaters the second item proves as important as the first one.

<table>
<thead>
<tr>
<th>Headings and main drafting items</th>
<th>Examples Proposals References</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIGN WORKING LIFE</td>
<td>The design working life can be generally taken equal to 100 years.</td>
</tr>
<tr>
<td>DESIGN SITUATIONS</td>
<td></td>
</tr>
<tr>
<td>SAFETY LEVELS</td>
<td>For Ultimate Limit States, $\beta = 1.5$ to $3$</td>
</tr>
<tr>
<td></td>
<td>For Serviceability Limit States, $\beta = 0.5$ to $1.5$</td>
</tr>
<tr>
<td></td>
<td>$\beta$ is calculated over</td>
</tr>
<tr>
<td>Headings and main drafting items</td>
<td>Examples Proposals References</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>--------------------------------</td>
</tr>
<tr>
<td>When probabilistic design methods are used, the actual safety index $\beta$ must be assessed and compared with the target safety index.</td>
<td>the design working life.</td>
</tr>
<tr>
<td>The partial factors proposed hereunder are calibrated with reference to some traditional design practice [ref: Kovarik 98].</td>
<td></td>
</tr>
<tr>
<td>The target safety index depends also on the inspectability and repairability of the structure.</td>
<td></td>
</tr>
</tbody>
</table>

**LOAD CASES**
An unique load case is defined in the permanent situations. It is represented by:

$$DL (\text{dead loads, self weight of concrete}) + W (\text{quasi-static load controlled by water levels: hydrostatic pressure and uplift pressure}) + H (\text{wave load}) + C (\text{current load})$$

For structures with an inner fill, $DL$ covers also the self weight of the inner fill. For the verification of structural stability limit states, the pressure due to the fill inside the caissons is introduced in the load case.

**PARTIAL FACTORS**
The semi-probabilistic format uses a set of partial factors based on the Eurocode's format (*future EN 1990 Basis of design*). Partial factors are divided into:

- **at source factors**, which apply to the basic variables, noted $\gamma_f$, $\gamma_M$ and $\gamma_R$.
- **model factors**, which relate to the load and resistance uncertainties, noted $\gamma_{sd}$ and $\gamma_{rd}$.
- **importance factors**, which allow for reliability differentiation, noted $\gamma_n$.

The general limit states condition reads:

$$\gamma_n \cdot \gamma_{sd} \cdot E(\gamma_f \cdot F_k) \leq R_d / \gamma_{rd}$$

Where, according to the resistance parameter:

$$R_d = R(X_k / \gamma_M) \text{ or } R_d = R(X_k) / \gamma_R$$

For the sake of simplicity, we consider here an unique model factor

$$\gamma_d = \gamma_n \cdot \gamma_{sd} \cdot \gamma_{rd}$$

see ENV 1991-1
PROVISIONS
The items listed hereafter are non-calculating conditions which must be fulfilled for a safe design.

RESPONSABILITY OF THE CONTRACTOR

The construction methods and choices for the works or part of them by land or by sea are under Contractor responsibility. The Contractor manages the works keeping them free from sea damages during storms. He takes all necessary arrangements to temporary protect works parts under construction and people and equipment withdrawal to safe areas when necessary.

Caissons precast, storage and handling methods and choices are left to the Contractor's initiative and remain under Contractor responsibility. In case of sea transportation or towage, the Contractor ascertains a sufficient and safe meteorological period from starting operations and final secure installation. The Contractor takes all the necessary arrangements to ascertain a continuously uniform contact of caisson bottom on its foundation all along its life.

INSPECTION

The Owner’s engineer performs a detailed inspection of every caisson before handling, flooding, launching, towage or transportation. In case of land transportation or sea-transportation on barges or other floating device, the Owner’s engineer has the option to reinspect the caissons to check the integrity after transportation.

DEFECTS

The Owner’s engineer has the option to require the Contractor to correct or repair defects or damaged concrete. The extent of damages must not endanger the caissons integrity and life duration. If the damages nature or extent is thought to be no reparable or seriously endanger the caissons integrity or life duration in the Owner’s engineer opinion, the Contractor does not to use the caisson in the works and rejects it.
### ACTIONS

#### DEAD LOADS

**CHARACTERISTIC VALUES**
The dead loads are evaluated with the geometrical values taken from the project’s sketches. The characteristic value of the unit weight of the reinforced concrete is 25 kN/m³. The characteristic value of the unit weight of the inner fill is to be assessed for the project.

**DESIGN VALUES**
For ULS the partial factor is 1.20 if the action is unfavourable or 0.90 if the action is favourable.

#### WATER LEVELS AND WAVE ACTION
(pulsating load and impact load)

**LOAD MODEL**
The basic parameters are the water level, the wave height, period and direction. In tidal sites a couple of water levels are defined. Horizontal water pressure and uplift pressure are to be determined consistently according to the appropriate model (Goda, Sainflou, Miché ...) involving the adequate wave parameter. For perforated breakwaters, the distribution of pressures in the caissons is determined with an appropriate model.

**CHARACTERISTIC VALUES**
The characteristic values of the water levels and the wave height are defined with reference to a return period.

**DESIGN VALUES**
The design values of the water levels and the wave height are defined with reference to a return period.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height ($H_s$)</td>
<td>represented by $H_s$</td>
</tr>
<tr>
<td>Period</td>
<td>peak period</td>
</tr>
<tr>
<td>Direction</td>
<td>allows for local wave climate</td>
</tr>
<tr>
<td>Return period</td>
<td>10 years, taking into account deterministic tide and stochastic surges and drops.</td>
</tr>
<tr>
<td>Return period</td>
<td>100 years.</td>
</tr>
<tr>
<td>Partial factor ($\gamma$)</td>
<td>$1.00$ to $1.20$</td>
</tr>
</tbody>
</table>
A partial factor can be applied to the wave height to allow for uncertainties in the determination of the characteristic value (accuracy of the data, use of transfer function, number of waves used for statistics ...)

**CURRENT**

**LOAD MODEL**
The basic parameters are the current velocity and direction. In some sites different current situations can be defined unless the detection of the most unfavourable one is obvious. Generally only tidal currents provide a significant load on the structure. The current action may however be prominent during construction situations. Horizontal pressure is to be determined according to an appropriate model.

**CHARACTERISTIC VALUES**
The characteristic values of the current velocity is to be assessed directly in the construction situation according to the sensitivity of the means of transportation.
In the permanent situation, the characteristic value is the maximum velocity under the worst tidal conditions.

**DESIGN VALUES**
For ULS the partial factor is 1.20 if the action is unfavourable or 0.00 if the action is favourable.

**INNER FILL PRESSURE**

**LOAD MODEL**
The pressures exerted by the inner fill to the walls of the caisson are evaluated according to an appropriate model.

**CHARACTERISTIC VALUES**
The characteristic values of the fill pressure is calculated with the characteristic values of the basic soil properties.

<table>
<thead>
<tr>
<th>Active pressure $K_a$, at rest pressure $K_0$, hydraulic fill ...</th>
</tr>
</thead>
<tbody>
<tr>
<td>see ENV 1997-1</td>
</tr>
</tbody>
</table>
### DESIGN VALUES
The design values of the fill pressure is calculated either with the design values of the basic soil properties or with partial factor 1.20 applied to the pressure coefficient (if favourable).

### LOAD COMBINATIONS
For the fundamental and the characteristic combinations, a reduced return period of the non dominating environmental parameters is to be taken into account, the return period of the dominating environmental parameter being equal to its design value (fundamental combination) or to its characteristic value (characteristic combination).

The frequent value of the environmental parameters is not considered for vertical breakwaters. The quasi-permanent values of the environmental parameters are:

- water levels: mean level,
- wave parameters: reduced return period,
- current: $\psi_2 = 0.00$

The return period is 50 years (fundamental combination) and 5 years (characteristic combination).

The return period is 1 year.

### MATERIALS AND RESISTANCES

#### REINFORCED CONCRETE
The general specifications of Eurocode 2 are applicable.
Material factors for ULS are $\gamma_M = 1.50$ on the resistance of concrete and 1.15 on the yield point of steel reinforcements.

See ENV 1992

#### GEOTECHNICAL PARAMETERS
The geotechnical parameters deal with the inner fill and the foundation soil layers. The general specifications of Eurocode 7 are applicable. Material factors for ULS are $\gamma_M = 1.20$ on the drained cohesion, 1.20 on the drained tangent of the internal angle of friction, 1.40 on the undrained cohesion, 1.40 on the pressuremeter results.

The bearing capacity is calculated either with Terzaghi’s model based on lab tests or with Ménard’s model based on in place tests.

The design value of the bearing capacity is calculated either with the design values of the basic soil properties or with partial factor 1.40 applied to its characteristic value. The model factor of the bearing...
capacity limit state may differ with the chosen option.

**FRICTION PARAMETERS**

The friction parameters slab / rubble layer involved in the sliding limit state are evaluated according to the materials. For ULS a partial factor $\gamma_M = 1.20$ is applied to the characteristic values of the friction and the adhesion.

The friction parameter wall / inner fill involved in the calculation of the pressure exerted by the inner fill is evaluated with allowance for the roughness of the wall and the shear resistance of the inner fill.

**ANALYSIS**

**STATIC EQUILIBRIUM**

The static equilibrium of the vertical breakwater is determined assuming a rigid caisson. Unless some provisions are taken to ensure the resistance of the junctions between the caissons, the calculations are carried out assuming 2D modelling. See ENV 1992

**STRUCTURAL ANALYSIS**

- The structural analysis may be carried out with simple calculations taking into account the support conditions of the front wall, the partition wall, the back wall, the base slab, as well as the actions of the inner fill. Finite Element Analysis may also be carried out. Different mechanical modelling may be used: elastic, elastic with redistribution, plastic, second order effects. See ENV 1992

**LIMIT STATES**

[ref: Bonnet 97, Marchais 97, Carrère et al. 97]

**ULTIMATE LIMIT STATES**

**BEARING CAPACITY**

The general provisions of ECe 7 apply. The limit state condition reads: $\gamma_d \cdot q_{ref} \leq q_u$

$\gamma_d = 1.20$ to $1.50$

Global factor is: $F = 2.50$
### SLIDING
The general provisions of Eurocode 7 apply. The limit state condition reads:

\[ \gamma_d \cdot H \leq V \cdot \tan(\phi_a) + S' \cdot c_a \]

where \( S' \) is the area of the foundation in contact with the soil, taking into account the eccentricity of the load.

| \( \gamma_d \) | 1.00 to 1.10 |
| Global factor is: | F = 1.50 |

### OVERTURNING OR STATIC EQUILIBRIUM
The general provisions of Eurocode 7 apply. The limit state condition reads:

\[ \gamma_d \cdot M_{destab} \leq M_{stab} \]

| \( \gamma_d \) | 1.20 |
| Global factor is: | F = 1.50 to 2.00 |

### RESISTANCE OF CONCRETE
The general provisions of Eurocode 2 apply. The limit state condition states that an equilibrium be found with reduced material properties and design stress multiplied by model factor \( \gamma_d = 1.125 \).

The limit states are:
- shearing failure,
- torsion failure,
- punching shear failure,
- buckling failure,
- breakage or yield,
- second order instability,
- fatigue,
- transformation into a mechanism.

[ref: CEB 75]

see ENV 1992

### SERVICEABILITY LIMIT STATES

### DURABILITY OF CONCRETE
The general provisions of Eurocode 2 apply. The characteristic combination is relevant. The limit states are:
- Permeability,
- appearance,
- cracking,
- deformations.

see ENV 1992

### SETTLEMENT
The general provisions of Eurocode 7 apply. The quasi-permanent combination is relevant. The limit state condition reads: \( \frac{s_{\text{calculated}}}{s_{\text{limit}}} \leq 1 \)

**TRANQUILLITY**

The limit state condition reads: \( H_{s,\text{transmitted}} \leq H_{s,\text{max}} \)

The fundamental and the characteristic combinations are relevant. Two criteria must be defined, taking into account the nature of the port operations.

<table>
<thead>
<tr>
<th>Limit State Condition</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s_{\text{limit}} = 20 \text{ cm or } 1/50 ) (differential settlement)</td>
<td>marinas (example): ( H_{s,\text{max}} = 0.50 \text{ m} ) (characteristic) ( H_{s,\text{max}} = 0.80 \text{ m} ) (fundamental)</td>
</tr>
</tbody>
</table>

### OTHER REQUIREMENTS

#### CONCRETE COVER

For exposure classes 4a and 4b, the concrete cover should be between 50 mm and 70 mm.

#### MATERIAL SPECIFICATIONS

- cement properties, aggregates properties
- admixtures
- grout
- rock / stones

see ENV 1992

#### CONSTRUCTION ARRANGEMENTS

**PRECAST**

Precast is managed in order to avoid caissons handling within a delay of 15 days from starting their fabrication, unless a specific strength study shows that a safe handling is possible in a shorter delay. Caissons precast area is a concreted plane slab able to carry the caissons and handling equipment applied loads. Caissons forms should not be removed within a period of 48 hours from pouring start; handling is not allowed before a period of 15 days from pouring start. Transport and final installation is done 28 days minimum after starting their fabrication, unless a specific strength study shows that a safe transport and installation is possible in a shorter delay.
### JOINTS
Where caisson top surfaces receive other concrete structures such as crown walls and slabs, the contact surfaces is treated as construction joints.

### FOUNDATION
The top of the caisson foundation is horizontally leveled to the theoretical caissons bottom slab level specified in drawings. Levelling is done with an horizontal straight beam perpendicular to the caissons sea-defence line and sliding on two rails correctly leveled or by other Engineer’s approved method. Immediately before caissons placement, the Contractor cleans the surface to remove deleterious materials if necessary.

### IN SITU BALLASTING OF CAISSONS
Dropping ballast material from the freewater surface is forbidden. The Contractor adopts a ballasting method which does not endanger the integrity of the caissons.

### TOLERANCES
Caissons structures and parts are brought to their final position by suited means, in order to get regular alignment with cross-sections and vertical elevations entering within the specified limits and tolerances.

#### IN THE VERTICAL DIRECTION
After final installation of the caissons and accounting for initial and final settlements, every point of the top surface of the caissons should remain within the following tolerances:
- Main breakwater caissons: \( z_p + 0.20 \) meters
- Quay caissons: \( z_p + 0.10 \) meters
where \( z_p \) is the theoretical vertical position of the caisson top horizontal surface defined in Contractor’s final drawings.

#### IN A PLAN VIEW
Individual tolerances when considering a caisson alone:
The caisson final position in a plan-view should remain between the

<table>
<thead>
<tr>
<th>JOINTS</th>
<th>FOUNDATION</th>
<th>IN SITU BALLASTING OF CAISSONS</th>
<th>TOLERANCES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Where caisson top surfaces receive other concrete structures such as crown walls and slabs, the contact surfaces is treated as construction joints.</td>
<td>The top of the caisson foundation is horizontally leveled to the theoretical caissons bottom slab level specified in drawings. Levelling is done with an horizontal straight beam perpendicular to the caissons sea-defence line and sliding on two rails correctly leveled or by other Engineer’s approved method. Immediately before caissons placement, the Contractor cleans the surface to remove deleterious materials if necessary.</td>
<td>Dropping ballast material from the freewater surface is forbidden. The Contractor adopts a ballasting method which does not endanger the integrity of the caissons.</td>
<td>Caissons structures and parts are brought to their final position by suited means, in order to get regular alignment with cross-sections and vertical elevations entering within the specified limits and tolerances.</td>
</tr>
</tbody>
</table>

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- 34 -
following limits with respect to the theoretical Contractor’s final drawings specified values:

- **Main breakwater caissons**:
  - Perpendicularly to the corresponding theoretical plan-view axis: \( \pm 0.20 \) m,
  - along the corresponding theoretical plan-view axis: \( \pm 0.20 \) m.

- **Quay caissons**:
  - Perpendicularly to the corresponding theoretical plan-view axis: \( \pm 0.10 \) m,
  - along the corresponding theoretical plan-view axis: \( \pm 0.10 \) m.

**Tolerances for an entire multi-caisson structure**:
Added individual caisson tolerances should not produce a total tolerance for the entire multi-caisson structure outside the values hereafter specified. This tolerance is defined relatively to the theoretical length of the plan-view longitudinal axis of the structure.

- **Main breakwater**: \([-0.00; +5.00]\) meters
- **Quay structures**: \([-0.00, +1.00]\) meters

**GAP BETWEEN TWO ADJACENT CAISSONS**
Caissons should as far as possible fully and uniformly keep in contact with each other, complying with the hereabove tolerances.

### 11. CONCLUSIONS

From the above presentation it can be verified that none of the examined codes encompasses all aspects of a caisson design. In particular no code addresses simultaneously concrete (structural) design and wave action. One of the main shortcomings of these Codes concerns the choice of physical conditions (waves, water levels, etc...) to be used for each of the loading combinations to be considered for structural design. For those countries accepting the principles of Limit States design, there is a lack of guidance for the determination of the design wave load to be introduced at both Ultimate Limit State (materials resistance) and Serviceability Limit State (materials durability).

Simply compiling all the most specific parts from the different codes would not result in a good approach because of the lack of homogeneity. The combination of existing rules can lead to unreasonable results in any case.
The gap is to be filled by developing recommendations for maritime structure and by harmonising safety levels. To avoid putting forward a plethora of recommendations, it is perhaps interesting to consider adapting what already exists in the EUROCODES semi-probabilistic limit states format, with the following guidelines:

- use of predetermined “γ” values for actions and materials, depending on the parameter itself and differentiated with statistical uncertainty (“at source factors”),

- calibration of “γd” “model factors”, specific to the structures and to the limit state under consideration, differentiated with the safety level and the design life duration.

The development of a consistent code for the design of vertical breakwaters implies to consider all aspects of the safety which are intercorrelated. It was not tried here to rewrite existing codes but to focus the attention on some additions necessary to ensure compliance with them. Compatibility with existing codes is mainly achieved using the concept of model factor.

ACKNOWLEDGEMENT

This work was partly supported by the Commission of the European Community within the research program MAST III “PROVERBS” (MAST contract MAS3-CT95-0041).

REFERENCES

ACI (American Concrete Institute), 1995. ACI 318-95 - Building Code Requirements for Structural Concrete and Commentary (ACI 318R-95)


CHAPTER 2  STRUCTURAL DESIGN OF VERTICAL BREAKWATERS

CEN (European Committee for Standardization), 1991. EUROCODE 1- Basis of design and actions on structures - ENV 1991-1 Part 1 Basis of design
KOVARIK J.B. De l’application des Eurocodes aux ouvrages maritimes et fluviaux, Revue Française de Génie Civil (à paraître en 1998).
MOPU (Ministerio de Obras Publicas, Spain), 1990. Maritime Works Recommendations - ROM 0.2 90 - Actions in the Design of Maritime and Harbour Works
Nervi, P.-L., 1951. Ferrocement - Its characteristics and potentialities. Ingegnerie, ANIAI
CHAPTER 3: CAISSON RELIABILITY DURING TRANSPORT AND PLACING

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ABSTRACT

This report was prepared within the Probabilistic Design of Vertical Breakwaters Project (PROVERBS). The construction stages of caisson breakwaters and corresponding design aspects are discussed. Special attention is paid to the reliability of operations as well as cracking and collapse of concrete elements. Several mechanical models varying from hand calculation to FEM models are considered. The work may serve as a technical basis for drafting of design guidelines and as a framework for future research for which some recommendations are given.

1. INTRODUCTION

Caissons to be used as parts of vertical breakwaters are usually built in building docks. This phase of the fabrication looks like the fabrication of any reinforced concrete structure. The next stage is the flooding of the dock and the floating of the caisson. As soon as the caisson becomes floating, the problem of its stability starts to be important. Additionally the caisson is loaded by hydrostatic forces which may lead to cracking and or collapse.

Next the caisson, empty or ballasted with water or sand, is transported to the building site. The distance depends on the local circumstances. During transport the caisson is loaded by waves and the forces from the tugging operation. Wave loading in general will be limited as transports will normally be postponed if the weather forecast predicts too high wind speeds. The criteria to be used for such a cancelling should be determined in the design stage. Chapter 2 formulates a probabilistic procedure for this design task.

The placing on the caisson is in majority the most critical phase of all operations. The foundation bed may prove to be unstable and/or uneven, leading to misalignment and various stresses inside the caisson. Some of these aspects will be discussed in Chapter 3.
An important risk during the transport and placing operation is impact due to collisions with other floating or standing objects. This item will not be discussed in this report. The strategy should be that these collisions have to be omitted as far as possible. It does not make much sense to design the structure for it.

2. RELIABILITY VERIFICATION OF THE TRANSPORT STAGE

2.1. Limit states to be considered

As the reliability verification will be based on the limit state approach, the first step is to define the limit states and corresponding loading conditions that should be considered. As limit states for the transport stage are proposed:

- instability due the various motions
- cracking of the concrete, as this may effect the durability of the caisson
- failure of the caisson due the outside water pressure

During transportation the structure is subjected to the following loads:

- gravity loads
- shrinkage and temperature
- hydrostatic pressure (buoyancy)
- wave loads
- wind loads
- towing

The wave, wind and towing loads generally have a dynamic character and cause motions, of which the heave, roll and pitch are the most important.

The limit state functions for the above limit states, in their most basic form, are presented by:

(1) Uncontrolled sinking of the caisson (see figure 1):

\[ g(X) = h_e + Z - \eta_s - h_0 \]

\[ h_e = \text{emerged height of caisson} = h_c - h_d \]
\[ h_c = \text{caisson height} \]
\[ h_d = \text{caisson draught} \]
\[ Z = \text{instantaneous vertical position of the caisson} \]
\eta_s = \text{instantaneous elevation of water surface above still water level on the seaward side of the caisson:}

h_0 = B_c \theta / 2

B_c = \text{width of caisson}

\theta = \text{angle of inclination}

(2) Cracking:

\[ g(X) = m_{ct} - m_t = 0.16 f_{ct} d^2 - m_t \]  \hspace{1cm} (2)

- \text{m}_{ct} = \text{plate moment at which cracking occurs}
- \text{m}_t = \text{bending plate moment due to the external loading [kN/m]}
- f_{ct} = \text{the tensile strength of concrete}
- d = \text{wall thickness}

(3) Plate collapse:

\[ g(X) = m_r - m_a \]  \hspace{1cm} (3)

- \text{m}_r = \text{resistant moment in wall or bottom}
- \text{m}_a = \text{loading moment in wall or bottom}

If the normal forces play a non dominant role the resistance moment can be calculated from:

\[ m_r = 0.9 \rho_s f_{ys} d^2 \]  \hspace{1cm} (4)

where \( \rho_s = \text{the reinforcement ratio, } f_{ys} \text{ is the steel yield strength and } d \text{ is the wall thickness. In a reliability analysis a model factor might be added.}

The loading side of the limit state functions will be elaborated in sections 2.5 and 2.6.
2.2. Basic variable modelling

2.2.1. Introduction

In this section the probabilistic models as defined until now are presented. Wave elevation and wind velocity are considered as random processes. Other random entities will be considered as random variables for the time being. Later on in the project some variables may be added or deleted.

2.2.2. Random processes of sea motion and wind speed

The random processes of sea motion and wind speed can be modelled as Gaussian stationary processes.

The random surface elevation is described by a zero mean and the Pierson-Moskowitz spectrum:

\[ S_{\eta}(\omega) = \alpha \cdot \omega^{-5} \exp\left(-\beta / \omega^{4}\right) \quad (5) \]

with coefficient \( \alpha \) and \( \beta \) given by:
\[ \alpha = \frac{H_s^2 (2\pi)^4}{4\pi} \]  

(6)

\[ \beta = 1.25 \cdot \omega_p^4 \]  

(7)

$H_s$ = significant wave height  
$T_z$ = zero crossing wave period  
$\omega_p$ = peak energy frequency ($T_p = 1.4 T_z$)

As an example the spectrum parameters have been calculated for $H_s = 2.5$ m and $T_p = 7$ s (corresponding to a water depth of 20 m, a fetch length of 100 km and a wind speed of 55 km/hour):

![Figure 2: Spectral density of surface elevation for: $H_s = 2.5$ m, $T_p = 7.0$ s]

Where relevant, a directional spreading spectrum may be used according to the cosine-squared distribution:

\[ S_{\eta\eta}(\omega, \theta) = S_{\eta\eta}(\omega) \cdot D(\theta) \]  

(8)

\[ D(\theta) = \frac{2}{\pi} \cos^2 \theta \quad |\theta| \leq \pi/2 \]  

(9)

In most cases, however, a one dimensional spectrum can be used as a conservative but fair approximation.

For wind loading the wind spectrum by Von Karman will be used:
$$S_{gg}(\omega) = \frac{1}{\omega} I^2 v_m^2 \cdot \frac{4 \frac{\omega L_w}{2 \pi v_m} \left[ 1 + 70 \left( \frac{\omega L_w}{2 \pi v_m} \right)^2 \right]^{-\frac{5}{6}}}{\omega}$$

\(\omega\) = gust frequency
\(v_m\) = mean wind velocity
\(L_w\) = length parameter \(\approx 220\) m
\(I\) = turbulence intensity

As an example see Figure 3.

![Figure 3: Spectral density of gust velocity, mean wind velocity \(v_m = 8\) m/s](image)

2.2.3. Random variables

The following table gives an overview of the selected random variables and their tentative modelling (\(\text{cov} = \text{coefficient of variation and sd = standard deviation}\)):

<table>
<thead>
<tr>
<th>Random variable</th>
<th>X</th>
<th>mean</th>
<th>variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height</td>
<td>(H_s) prediction</td>
<td></td>
<td>(\text{cov} = 0.20)</td>
</tr>
<tr>
<td>mean wind speed</td>
<td>(v_m) prediction</td>
<td></td>
<td>(\text{cov} = 0.10)</td>
</tr>
<tr>
<td>Significant wave height due to tugging</td>
<td>(H_{s\text{ tug}}) 0.3m</td>
<td></td>
<td>(\text{cov} = 0.30)</td>
</tr>
<tr>
<td>Towing velocity</td>
<td>(v_t) 3 m/s</td>
<td></td>
<td>(\text{cov} = 0.20)</td>
</tr>
<tr>
<td>drag coefficient</td>
<td>(C_D) 1.5</td>
<td></td>
<td>(\text{cov} = 0.10)</td>
</tr>
<tr>
<td>Direction of towing</td>
<td>(\delta) 0 (\pm 100) deg</td>
<td></td>
<td>(\text{sd} = 10) deg</td>
</tr>
</tbody>
</table>
### Random variable

<table>
<thead>
<tr>
<th>Random variable</th>
<th>X</th>
<th>mean</th>
<th>variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance towing line to swl</td>
<td>e</td>
<td>1-5 m</td>
<td>sd = 0.5 m</td>
</tr>
<tr>
<td>Caisson draft deviation</td>
<td>Δhₙ₀</td>
<td>0</td>
<td>sd = 0.10 m</td>
</tr>
<tr>
<td>initial caisson rotation</td>
<td>Δθ₀</td>
<td>0</td>
<td>sd = 1.0 deg</td>
</tr>
<tr>
<td>Righting moment</td>
<td>Mᵣ</td>
<td>nominal</td>
<td>Cov = 0.02</td>
</tr>
<tr>
<td>Tensile strength of concrete</td>
<td>fₜₐₘ</td>
<td>2.7 MPa</td>
<td>sd = 0.5 MPa</td>
</tr>
<tr>
<td>time of operation:</td>
<td>tₒₜₚ</td>
<td>24 hours</td>
<td>Cov = 0.15</td>
</tr>
<tr>
<td>steel strength</td>
<td>fᵣ</td>
<td>nominal</td>
<td>Cov = 0.08</td>
</tr>
<tr>
<td>Cover</td>
<td>c</td>
<td>nominal</td>
<td>Cov = 0.15</td>
</tr>
<tr>
<td>Model factor</td>
<td>m</td>
<td>1.0</td>
<td>Cov = 0.10</td>
</tr>
</tbody>
</table>

Some clarification:

**Significant wave height and mean wind speed**

The significant wave height and the mean wind velocity are modelled as normal random variables with the mean values equal to some value as predicted by the weather forecast and a scatter representing the uncertainty in this prediction. The weather conditions leading to the minimum acceptable values of the reliability should be regarded as the limits where transport can be permitted. In a more advanced model one might want to make the prediction accuracy dependent on the total transportation time. For the time being, fixed values have been chosen. For more information about optimisation the weather condition see [38].

**Towing action**

The wave height due to the operation with tugs may be estimated [30] as about Hₜᵤᵍ = 0.5 m at a distance of 50 m from the tug. The significant wave Hₛₜᵤᵍ height is therefore estimated as 0.30 m. A relative large coefficient of variation for this value has been chosen, representing the large uncertainty.

The force due to towing depends on: the towing velocity, the drag coefficient, the position of the towing point, and the cross-section of under water part of the caisson [56]. Every variable may be considered as random. The coefficients of variation are estimated on the basis of engineering judgement. For the notation of the towing parameters, see figure 4.
Figure 4: Notation of towing parameters

Imperfections

The uncertainties of the restoring moment, caisson weight and mass distribution of the immovable ballast depend on the workmanship. Some reduction is possible by measuring and correcting draught and initial rotation after floating.

Resistance properties

The tensile strength of concrete is the main random variable as far as cracking is concerned [21]. For the bending strength in collapse the reinforcement yield stress and geometric position are the most dominating variables. Additionally some model factor may be important.

2.3. Reliability Requirements

The probability of exceeding a limit state during the transport stage must be smaller than a prescribed value. The value depends on the type of limit state. For the limit states in this project the following targets are recommended:

- instability $\beta \geq 3.0 \quad p \leq 0.001$
- cracking $\beta \geq 1.5 \quad p \leq 0.07$
- collapse $\beta \geq 3.0 \quad p \leq 0.001$
The reliability index $\beta$ is defined as $\Phi^{-1}(P)$, with $P$ equal to the acceptable failure probability and $\Phi$ is the distribution function of the standard normal distribution.

It is proposed that in the design procedure only member verifications or verifications of individual cross sections will be carried out. This means that system behaviour is reflected in the target values given.

### 2.4. Reliability verification methods

#### 2.4.1. Full probabilistic analysis

In a full probabilistic verification procedure the failure probability $P_f$ should be calculated and it should be proved to be less than the target defined in 2.3.3.

Given some limit state function $g(X)$ the failure probability can be expresses as:

$$P_f = P(g(X)<0)$$

$P_f$ = failure probability

$X_i$ = vector of random variables

A quite general formulation for the limit state function $g(X)$ in the present field of application is given by:

$$g(X) = \tau_{\text{limit}} - \tau_{\text{max}}$$

$\tau$ = response variable (e.g. stress)

$\tau_{\text{limit}}$ = limit value of the response variable (e.g. strength)

$\tau_{\text{max}}$ = maximum value of the response variable during the operation period $t_{op}$

Assuming that the response variable fluctuates as a Gaussian process, the maximum response can be written as $\tau_{\text{max}} = \tau_{\text{average}} + r_{\text{max}} \sigma_{\tau}$, leading to:

$$g(X) = \tau_{\text{limit}} - \tau_{\text{aver}} - r_{\text{max}} \sigma_{\tau}$$

$\tau_{\text{aver}}$ = mean value of the response variable (e.g. stress due to self weight)

$\sigma_{\tau}$ = standard deviation of fluctuating part of the response variable (e.g. due to waves)

$r_{\text{max}}$ = maximum of $m$ unit Rayleigh distributed variables:
\( P\{r > r_{\text{max}}\} = m \exp \{-0.5 r_{\text{max}}^2\} \leq 1 \) \hfill (14)

\( m \) = number of repetitions during operation time (e.g. number of waves)

The limit value of the strength and the mean and standard deviation of the response are functions of the various random variables discussed before:

\[
\begin{align*}
\tau_{\text{limit}} &= h_1(X_1 \ldots X_n) \\
\tau_{\text{aver}} &= h_2(X_1 \ldots X_n) \\
\sigma_{\tau} &= h_3(X_1 \ldots X_n)
\end{align*}
\hfill (15)
\]

The standard deviation normally follows after integration of the response spectrum over \( \omega \) (see section 2.5.6).

2.4.2. Partial factor method

In the partial factor method all random variable are replaced by design values \( X_d \). The values \( X_d \) follow from:

\[
\begin{align*}
X_d &= \gamma X_k \quad (\text{for load type variables}) \\
X_d &= X_k/\gamma \quad (\text{for resistance type variables})
\end{align*}
\hfill (16, 17)
\]

The \( \gamma \) factors are tuned in such a way that:

\[ X_d = F_x^{-1}(\Phi(-\alpha\beta)) \hfill (18) \]

The value of \( \beta \) follows from section 2.3. The \( \alpha \)-values, according to ISO [20], are:

<table>
<thead>
<tr>
<th>Resistance parameters</th>
<th>Dominant</th>
<th>Not dominant</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.8</td>
<td>0.3</td>
</tr>
<tr>
<td>Load parameters</td>
<td>-0.7</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

The \( \alpha \)-values may also follow from special background calculations.

2.5. Models and transfer functions for the instability limit state function

2.5.1. Hydrodynamic loads

The following Froude - Krylov model gives relatively simple expressions for the hydrostatic pressures and the pressures due to waves on a wall of a floating caisson (see Figure 5):
\[ p_1(z) = \gamma_w(z-Z_e) \]  
(19)

\[ p_3(z) = (\eta_i + \eta_r) \gamma_w e^{kz} \]  
(20)

\[ p_4(z) = (\eta_t + \eta_f) \gamma_w e^{kz} \]  
(21)

\( \eta_i \) = maximum water elevation for the incident wave
\( \eta_r \) = maximum water elevation for the reflected wave = \( c_r \eta_i \)
\( \eta_t \) = maximum water elevation for the transmitted wave = \( c_t \eta_i \)
\( \eta_f \) = maximum water elevation for the diffracted wave = \( c_f \eta_i \)
\( c_r \), \( c_t \), \( c_f \) = reflection coefficient (see Annex A)
\( \gamma_w \) = unit weight of water
\( Z_e \) = vertical displacement of the wall (heave coupled with roll)
\( k = 2\pi/L \)

For some applications the pressure on the toe can be ignored.

The height of reflected and transmitted wave, heave and roll characteristics can be calculated after the various models like:

- laboratory and in situ test [see references 3,5,8,9,13,14,30,31,33,35,]
- potential theory

For numerical values see Appendix A, for numerical models see Annex B.
2.5.2. Heave due to wave loading

The equation of heaving motion can be expressed as (see figure 5):

\[ m \cdot \ddot{Z} = F_{pz} + F_{iz} + F_{wd} - W \]  

\( F_{pz} \) = pressure force  
\( F_{iz} \) = inertial force in the heaving direction  
\( F_{wd} \) = drag due to wave generation  
\( W \) = weight of breakwater  
\( Z \) = upward vertical displacement of the mass centre of the block  
\( m \) = mass of caisson

The various forces are given by:

\[ F_{pz} = B_c \cdot l_c \cdot \frac{p_3(-h_d) + p_4(-h_d)}{2} \]  
\[ F_{iz} = m_h \cdot (\ddot{u}_z - \ddot{Z}) \]  
\[ F_{wd} = N_z (\ddot{u}_z - \ddot{Z}) \]

\( p_3(-h_d) \) = pressure at the base due to wave motion on the seaward side  
\( p_4(-h_d) \) = pressure at the base due to wave motion on the shadow side

Figure 5: Distribution of water pressure on structure
CHAPTER 3  CAISSON RELIABILITY DURING TRANSPORT AND PLACING

$c_l$ = length of caisson  
$c_B$ = width of caisson  
$m_h$ = hydrodynamic added mass in heave motion (see Annex A)  
$u_z$ = fluid vertical displacement  
$N_Z$ = damping factor (see Annex A)

On the basis of differential equation (2.5.2.1) for the heave motion the transfer function $H_{Z\eta}(\omega)$ for the vertical displacement $Z$ of caisson may be derived as [3]:

$$H_{Z\eta}(\omega) = \frac{2}{\omega^2} \frac{\sqrt{e^2 + f^2}}{\left[1 - \frac{\omega_Z^2}{\omega^2}\right] + \left(2 \cdot \zeta_Z \frac{\omega_z}{\omega}\right)^2} \tag{26}$$

where

$$e = \frac{N_Z}{m + m_h} \frac{g}{2 \cdot h_d \cdot \omega} \left[\cosh(ks) - \cosh(kd)\right] \tag{27}$$

$$f = \frac{m_h}{m + m_h} \frac{g}{2 \cdot h_d} \left[\cosh(kd) - \cosh(ks)\right] \tag{28}$$

$$\gamma_w \cdot B_c \cdot l_c \cdot \frac{1}{4 \cdot (m + m_h)} \left[1 + c_r + c_t + c_f\right] \frac{\cosh(ks)}{\cosh(kd)} \tag{29}$$

$$\omega_Z^2 = \gamma_w \cdot B_c \cdot \frac{l_c}{m + m_h} \tag{30}$$

$$2 \cdot \zeta_Z \cdot \omega_Z = \frac{N_Z}{m + m_h} \tag{31}$$

$m$ = mass of caisson  
$h_d$ = draft of caisson  
$d$ = water depth  
$s$ = $d-h_d$  
$c.$ = reflection, transmission and diffraction coefficients (see Annex A)  
$L, T$ = length and period of wave  
$\zeta_Z$ = damping ratio of heave motion  
$\phi_Z$ = phase angle  
$\omega_Z$ = eigen frequency of heave motion
This function may be used as part of the transfer function for the limit state of unstable motion.

For comparison with a numerical analysis: see Annex B.

2.5.3. Roll motion due to wave loading

The equation of roll motion due the wave action about a lateral axis $y$ through the centre of mass can be expressed as (only the case of fixed ballast is considered here):

\[ I_{yy} \cdot \ddot{\theta} = M_{i\theta} + M_{D\theta} + M_{B\theta} + M_{ip} \]  

(32)

- $M_{i\theta}$ = moment caused by inertial wave forces
- $M_{D\theta}$ = damping moment caused by wave generation due to rolling motion
- $M_{B\theta}$ = restoring moment due to displacement of buoyancy centre
- $M_{ip}$ = moment due to wave pressure forces at the caisson periphery

The moments are given by:

\[ M_{i\theta} = I_0 \cdot (\ddot{\psi} - \ddot{\theta}) \]  

(33)

\[ M_{D\theta} = N_0 \cdot (\ddot{\psi} - \ddot{\theta}) \]  

(34)

\[ M_{B\theta} = -W \cdot (GM) \cdot \theta \]  

(35)

\[ M_{ip} = l_c \int_{-h_a}^{\eta_1 + \eta_r} p(x, z)(z - h_g)dz - \int_{-h_a}^{\eta_1 + \eta_r} p(x, z)(z - h_g)dz - \int_{-\gamma}^{\gamma} p(x, z)x dx \]  

(36)

$GM$ = distance between centre of gravities and meta centre
$I_0$ = hydrodynamics (added) moment of inertia
$\psi$ = rotation displacement of fluid
$N_0$ = damping factor of roll
$h_g$ = distance between still water level and centre of rotation ($\approx 0.1 h_d$)
$l_c$ = length of the caisson

\[ p(x, z) = \gamma_w \left[ \eta_1 + \eta_r - \eta_1 - \eta_r \right] \frac{B}{2} x + \frac{\eta_1 + \eta_r + \eta_1 + \eta_r}{2} \exp \frac{-2\pi x}{L} \sin \frac{2\pi x}{L} \]  

(37)

The solution is:
\[ H_{\theta \eta}(\omega) = \frac{1}{\omega^2 \cdot \left( I_{yy} + I_0 \right)} \cdot \frac{M_{ip}(\omega)}{\eta_1(\omega)} \cdot \frac{1}{\sqrt{\left( 1 - \frac{\omega_0^2}{\omega^2} \right)^2 + \left( 2 \cdot \zeta_0 \cdot \frac{\omega_0}{\omega} \right)^2}} \] 

(38)

where \( \omega_0 \) is eigen frequency of roll motion and \( \zeta_0 \) is the damping coefficient (see Annex A):

\[ \omega_0^2 = \frac{W \cdot GM}{I_{yy} + I_0} \] 

(39)

\[ 2 \cdot \zeta_0 \cdot \omega_0 = \frac{N_\theta}{I_{yy} + I_0} \] 

(40)

Numerical experiments have shown that the moment \( M_{ip} \) is a non-linear function of wave height. Up to now various researchers have expanded the moment \( M_{ip} \) in power series neglecting the parts of higher order. In our research let us assume the linear approximation of moment \( M_{ip} \) vs. wave height.

For comparison with the numerical model, see Annex B.

2.5.4. Roll motion due to wind

The quasi static equation for cross loading for the wind action is given by:

\[ W \cdot GM \cdot \theta = M_{ws} = \frac{1}{2} \cdot \rho \cdot \left( C_p \cdot h_e \cdot \frac{h_e}{2} + C_f \cdot B_c \cdot \left( h_e - \frac{h_d}{2} \right) \right) \cdot w_m^2 \cdot l_e \] 

(41)

The dynamic part is described by:

\[ \left( I_{yy} + I_0 \right) \cdot \ddot{\theta} + N_\theta \cdot \dot{\theta} + W \cdot GM \cdot \theta = M_{wd} = \frac{1}{2} \cdot \rho \cdot \left( C_p \cdot h_e \cdot \frac{h_e}{2} + C_f \cdot B_c \cdot \left( h_e - \frac{h_d}{2} \right) \right) \cdot w_g^2 \cdot l_e \] 

(42)

\( M_{ws} \) = quasi static heeling moment due the wind action
\( M_{wd} \) = dynamic heeling moment due the wind action
\( \rho \) = density of air = 1.023 kg/m³
\( z \) = vertical co-ordinate from the centre of gravity
\( C_p \) = pressure coefficient on the walls (see Figure 6)
\( C_f \) = friction coefficient on the caisson top (see Figure 6)
This leads to the following transfer function:

$$H_{\theta g}(\omega) = \frac{1}{\omega^2 \cdot (I_{yy} + I_{\theta})} \left[ C_p \cdot h \cdot \frac{h}{2} + C_f \cdot B \cdot \left(h - \frac{h_d}{2}\right)\right] \cdot I_c$$  \hspace{1cm} (43)

2.5.5. Roll due to towing and maneouvring

The heeling moment due the towing and manoeuvring may be estimated from (see Figure 4):

$$F_t = \frac{1}{2} C_D \cdot \rho_w \cdot B \cdot h_d \cdot v_t^2 \cdot |\sin(\delta)|$$  \hspace{1cm} (44)

$$M_t = F_t \cdot e_t$$  \hspace{1cm} (45)

$F_t$ = average force in line
$e_t$ = arm of the towing force
$\delta$ = angle of towing direction

In the absence of more precise data it is assumed that the towing force is independent of the towing direction. The angle of the towing direction may be beyond 90° due some yaw [36, Appendix C]. The resulting roll of the caisson is given by:

$$\theta = \frac{M_t}{W \cdot GM}$$  \hspace{1cm} (46)
2.5.6. Limit state function

We can now express the limit state function (2.1.1). Using the general formulation (2.4.13) and the results of section (2.5) explicitly in the basic variables:

\[ g(x) = h_c - h_d - 0.5B_c \{ \theta_{tug} + \theta_{ini} \} - r_{max}\sigma_\Delta \{ H_s T_t H_{tug} V_m \ldots \} \]  

The tug angle can be expressed as:

\[ \theta_{tug} = \left( 0.5C_D C h_d v^2 c \sin \delta \right) / W GM \]  

As \( \Delta = Z - B_c \theta / 2 - \eta_i - \eta_r \) we may find \( \sigma_\Delta \) from:

\[ \sigma_\Delta^2 = \int \left[ H_{Z\eta} + 0.5 B_c H_{\theta\eta} + 1 + c_r \right] S_{\eta\eta} \, d\eta \]  

Note that we have neglected phase differences between the various contributions. Wind has been left out and should be added if an important contribution is to be expected (e.g. when transport is done with very small draft).

2.6. Models and transfer functions for the Limit states of cracking and yielding

2.6.1. General

The general calculation procedure can be summarised as follows:

(1) Define the loadings due to weight, ballast, waves, wind and towing
(2) Calculate the response performing a structural analysis
(3) Check the limit state equations for cracking and collapse

Step (2) can be performed using a static and a dynamic analysis. In a static analysis one takes the pressures following from the dynamic load analysis of section 2.5. The structure is then supported in a statically determinate way, which means that the support reactions will be zero.

In a dynamic analysis one has the following choices:
(a) one takes the loads from step (1) and uses very weak spring supports; this is similar to the static procedure, but high frequencies may give rise to dynamic effects in the structure
(b) the first and second step are combined using a fluid structure interaction model.
(c) the first and second step are combined using a simplified fluid structure interaction model in the form of a Winkler model

As far as the structural model is concerned, a linear model will be assumed, both for the cracking as for the collapse limit state. In the latter case a linear model is conservative as no redistribution of stress peaks is taken into account. For more information on options (b) and (c), see Annex B.

The load definitions for the static analysis and for the dynamic analysis, option (a) are presented in the subsequent sections for waves coming form various directions.

2.6.2. Pressure on the floating structure for roll motion

Using the notation from section 2.5 the pressure at both sides of the caisson are for a harmonic wave $\eta_i \sin (\omega t - kx)$:

$$p(z) = \gamma_w z + \gamma_w \left\{ Z + B_c \theta / 2 + (\eta_i + \eta_r) e^{kz} \right\} \sin(\omega t \pm kB_c / 2)$$  \hspace{1cm} (50)

The transfer functions $H_{zn}$ and $H_{\theta \eta}$ may be obtained from section 2.5. Given the pressure $p(z)$ the bending moments in a static response analysis can found from:

$$m = (1/12) p(z) a^2$$  \hspace{1cm} (51)

where $a$ represents the span between the cross walls in the caisson; the moment can be used both for the cracking as the collapse limit state.

2.6.3. Forces in longitudinal direction

The total bending moment due to waves in longitudinal is the sum of a static wave part, a dynamic wave part and a part from towing.
Figure 7: Longitudinal wave loading of caisson if $l_c \approx L$

Static part

If the waves come in the longitudinal direction of the caisson (see Figure 7), the wave elevation causes bending moments, which may be calculated using the beam model:

$$\frac{d^2 M(y)}{dy^2} = q(y) = \pm \gamma_w \cdot B_c \cdot \left( \eta_i \cdot \cos \left( \frac{2 \cdot \pi \cdot y}{L} \right) \right)$$  \hspace{1cm} (52)

The stresses may be calculated using the spectrum approach. The transfer function of the bending moment in an arbitrary cross-section $y$ is:

$$H_M \eta(\omega, y) = \gamma_w \cdot B_c \cdot \left( \frac{L(\omega)}{2 \cdot \pi} \right)^2 \left[ 1 - \cos \left( \frac{2 \cdot \pi \cdot \left( \frac{y + \frac{l_c}{2}}{2} \right)}{L(\omega)} \right) \right] \left[ 1 - \cos \left( \frac{2 \cdot \pi \cdot \frac{l_c}{2}}{L(\omega)} \right) \right] \frac{y + \frac{l_c}{2}}{l_c}$$  \hspace{1cm} (53)

An example of $S_{nn} = H^2 S_{\eta\eta}$ is presented in Figure 8.
Figure 8: Spectrum of moment $S_M$ due the wave action for the cross-section in 1/2 and 1/4 of the caisson length.

The most loaded cross-section is placed near the caisson centre, depending on the proportion of caisson length to wave length. Usually the difference between the most loaded cross-section and middle cross-section is less than a few percents.

The towing part moment coming from the towing forces is given by:

$$M(y) = \int_{0}^{l_c/2} \gamma_w \cdot B_c \cdot \psi \cdot y \cdot dy$$

The notation is like in section 2.4.5. The mean pitch angle $\psi$ may be calculated from:

$$\psi = \frac{M_t}{W \cdot G_m}$$

$$M_t = cF_{t_2}$$

$$F_t = 1/2 \cdot C_D \cdot \rho_w \cdot B_c \cdot h_d \cdot V_t^2$$

**Dynamic part**

In case a dynamic contribution is important, the following transfer function can be used to find the pitch spectrum.
H_{\nu\eta}(\omega) = \frac{1}{\omega^2 \cdot (I_{xx} + I_\psi)} \cdot \int_{-\frac{L}{2}}^{\frac{L}{2}} y \cdot \sin\left(\frac{2 \cdot \pi \cdot y}{L}\right) \cdot dy \right)

\omega_{\psi} = \text{eigenfrequency in pitch}
GM = \text{metacentric height for longitudinal direction}
I_{xx} = \text{mass moment of inertia about the x axis}
\zeta_0 = \text{damping ratio for roll (if not available for pitch)}
l_c = \text{length of the caisson}

\omega_{\psi}^2 = \frac{W \cdot GM}{I_{xx} + I_\psi} \tag{59}

2.6.4. Torsion of the caisson

The wave overcoming in longitudinal or aslant direction coupled with the towing force cause the non-uniform distribution of buoyancy, which is the reason of bending and torsion of the whole structure [10,12,25].

The torsion moment may be important when the waves come aslant. The loading on the structure is shown in Figure 9.
The torsional moment (per unit length) due to the wave action under an angle $\alpha$ is given by:

$$m(y) = \gamma_w \cdot \int_{-B_c/2}^{B_c/2} \eta_i \cdot x \cdot \cos \left( \frac{2 \cdot \pi \cdot (x + y \cdot \tan(\alpha))}{L \cdot \cos(\alpha)} \right) \cdot dx$$  \hspace{1cm} (60)

$$\tan(\alpha) = \frac{B_c}{l_c}$$

The internal moments can be derived from the torsional beam theory, although results are not very accurate. It is better to use FEM. An example may be found in Annex C.
3. RELIABILITY VERIFICATION OF SINKING DOWN PROCEDURE

3.1. Limit States to be considered

Sinking down is often considered as the most critical part of the construction of caisson breakwaters. The following disasters have been noticed:

1. It has occurred twice, that uncontrolled caisson motion has made it impossible to finish the operation accurately. In one case where the eigenfrequency of the caisson during placing was nearly identical to the dominant frequency in the wave spectrum. The severe motions hindered to achieve an appropriate placing precision and probably partly disturbed the rubble mound. In another case (Las Palmas) the storm and the limited stability of the water ballasted caisson caused the caisson to sink down with a final misplacement of about 1 m.

2. Due to the impact of placing and liquefaction of the soil, the Ekofisk Gravity Offshore Platform has slid about 7 m. This kind of failure may also occur in the case of the caisson breakwaters. To avoid the unknown torsion and bending of the structure due to placing on uneven bottom, sand is used which is sensitive to liquefaction.

3. The collapse of the wall of the breakwater in Gdansk. There were two reasons: an error by the contractor when placing the reinforcement and a wrong procedure to produce the ballast. The caisson had been placed with water ballast, after which a high efficient dredger pushed sand into the caisson so that impact caused collapse of the wall.

One has suggested that the placing might be in danger due to the following reason:

- inaccuracy of placing
- washing up of soil during sinking down
- liquefaction of top sand layer
- cracking/failure due to uneven ballast loading
- cracking/failure due to randomly uneven bottom

The problem of placing precision seems to be the most complicated to analyse because the basic motion equations for say, surge and yaw do not always take into account the complicated geometry (two or more tugs, several lines, wave reflection and diffraction on the already placed caissons etc.) and need to be analysed on the level of non-linear dynamics [36].

In this document only the last limit state, that is, cracking and failure due to an uneven bottom, will be considered.
3.2. **Random Variables**

As basic random variables may be considered:

- unevenness of the bottom
- angle of friction $\phi$
- Young’s modules $E$
- Poisson ratio $\nu$

The surface of rubble mound should be modelled as a random field. It is known to have an unevenness between 50 and 200 mm and radius of autocorrelation between 5 and 20 m [39,60]. Let us suppose that the unevenness can be modelled as a homogeneous normal filled, with zero mean, a standard deviation of 100 mm and a correlation function given by:

$$C(r_1,r_2)=\exp(-c((r_{1x}-r_{2x})^2+(r_{1y}-r_{2y})^2))$$  \hspace{1cm} c > 0  \hspace{1cm} (61)$$

$r_1,r_2$ = location vectors of 2 points (1) and (2)
$c$ = correlation distance parameter

The soil properties will be treated as random variables, with the following properties:

<table>
<thead>
<tr>
<th>random variable</th>
<th>$X$</th>
<th>mean</th>
<th>variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>angle of friction</td>
<td>$\phi$</td>
<td>nominal</td>
<td>$\text{cov} = 0.20$</td>
</tr>
<tr>
<td>Young’s modules</td>
<td>$E$</td>
<td>nominal</td>
<td>$\text{cov} = 0.50$</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>$\nu$</td>
<td>nominal</td>
<td>$\text{cov} = 0.10$</td>
</tr>
</tbody>
</table>

The nominal values should follow from field investigations.

3.3. **Reliability Requirements**

Only reliability requirements for cracking and local collapse will be considered. The values can be the same as for the transport stage:

- cracking $\beta > 1.5$ $p < 0.070$
- collapse $\beta > 3.0$ $p < 0.001$

The requirements hold per cross section.
3.4. Verification procedures

The methods are similar to section 2.4.

The full probabilistic analysis should be done by Monte Carlo. For the design value method: see 3.5.

3.5. Stresses due to uneven bottom

The bearing capacity of a rubble mound foundation in the context of a random field simulation have to consider that the caisson is supported on hills and has no contact in the valleys. In this case the base contact zone may be selected as local spots of spherical shape, such that its radius of curvature \( R \) fits as close as possible the actual geometry of the unevenness [55] (see Figure 10).

\[
R = \frac{h}{2} + \frac{D^2}{8 \cdot h}
\]

(62)

The position of the spots and their heights follow directly from the random field simulation.

![Figure 10: Geometry of bottom unevenness](image)

The contact stresses for one spot may be elastic or plastic. The plastic stresses provide an upper limit and are expressed by the bearing capacity formulae:

\[
q_{pl} = \frac{1}{4} \sqrt{\pi \cdot \gamma \cdot d \cdot N } \\
N = 0.9 \cdot \exp(\pi \cdot \tan(\phi) \cdot \tan^2(45 + \phi/2) - 1) \cdot \tan(\phi)
\]

(3.5.2)

The average elastic stresses at the contact area may be calculated from Hertz’s theory of contact as
The contact area diameter of the stress \( q \) may be expressed as a function of the base contact force \( \Delta F \)

\[
d = \begin{cases} 
\frac{6 \cdot R \cdot \Delta F \cdot (1 - v^2)}{E} & \text{elastic stresses} \\
1.4 \sqrt{\frac{\Delta F}{\gamma \cdot N_r}} & \text{plastic stresses}
\end{cases}
\]  

and the local settlement \( \Delta z \) as

\[
\Delta z = \frac{d^2}{4 \cdot R}
\]

Due to the fact that the most dangerous moment of ballast is not known, it is suggested to use an incremental procedure of calculating the bearing capacity.

The following conditions have to be satisfied for the equilibrium for each stage of ballast

\[
W_c + W_b = \sum_{i=1}^{n} \Delta F_i
\]

\[
\sum M_x = \sum_{i=1}^{n} \Delta F_i \cdot y_i = 0
\]

\[
\sum M_y = \sum_{i=1}^{n} \Delta F_i \cdot x_i = 0
\]

\( W_c \) = caisson weight \\
\( W_b \) = ballast weight \\
\( M_x, M_y \) = global moments \\
\( n \) = number of spots, where the contact occurs

The eventual interaction with soil ballast can be estimated by FEM, using the fair interface FE (with friction and compression, but no tension).

References
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[15] "DIANA Finite Element Analysis” (programme and user's manual), TNO Building and Construction Research, Delft
[16] "Analysis of Spread Mooring Systems for Floating Drilling Units" API Recommended Practice 2P, USA 1987
[18] Det Norske Veritas "Rule for classification. Mobile offshore units"
[19] van den Bosch J. J., de Zwaan A. P. "Roll damping by free surface tanks with partially raised bottom" TNO Ship Research Centre, Delft 1974
[38] Nishigori T. "Optimum design and work execution in the construction of a man-made island for the site of Gobo Thermal Power Station", Civil Engineering in Japan, 1984
[40] Dmitrieva I. " DELFRAC 3-D potential theory including wave diffraction and drift forces acting on the structures " Delft University of Technology, 1994
[45] Garrison C.J. "Hydrodynamics of large objects in the sea; Part II: Motion of free-floating bodies", Journal of Hydrantronics Vol. 9, No. 2 1974
[50] Cywinski Z. "Thin-walled structures" (manuscript) Gdansk Technical University
[51] Bielewicz E. "Random fields. Applications in structural mechanics" (manuscript) Gdansk Technical University
[55] Smits F.P. "Geotechnical design of gravity structures" Delft Soil Mechanics Laboratory
[56] Bernitsas M.M., Kekridis N.S. "Simulation and stability of ship towing"
[60] Keaveny J.M., Nadim F., Lacasse S. "Autocorrelation functions for offshore geotechnical data" ICASSAR'89
[61] Cakmak A.S. "Soil dynamics and liquefaction" Elsevier Amsterdam 1987
HYDROMECHANIC CHARACTERISTICS

Added mass and damping in heave motion

The added mass \( m_h \) and damping \( N_z \) in heave motion for the vertical floating caisson may be taken from Gerritsma’s investigation [5], see Figure I.1

![Figure I.1](image)

\[ \frac{m_h}{\rho_w A_l c} \]

\[ \frac{N_z}{\rho_w A_l c \sqrt{\frac{B_c}{2g}}} \]

Figure I.1: Hydrodynamic mass and damping coefficient for a rectangular 2D cross-section, \((B_c/h_d=2)\)

Added mass and damping in roll motion

The added hydrodynamic moment of inertia \( I_0 \) may be estimated as a part of the "dry" moment of inertia \( I \).

\[ I_0 \approx \begin{cases} 0.50 \cdot I & \text{for wave action} \\ 0.15 \cdot I & \text{for wind action} \end{cases} \quad (I.1) \]

The "dry" moments of inertia for caissons may be estimated as:

\[ I_{xx} = 0.1 \cdot \left( h_c^2 + l^2 \right) \cdot m \quad (I.2) \]

\[ I_{yy} = 0.1 \cdot \left( h_c^2 + B_c^2 \right) \cdot m \quad (I.3) \]

The damping ratio may be estimated by the experimental formula by Journee [13]
The last term may usually be neglected.

**Reflection coefficients**

The coefficients $c_r$, $c_t$, $c_d$ describe the reflected, transmitted and dissipated parts of wave energy due to caisson being in water and are defined as:

\[
c_r = \frac{H_r}{H_i}
\]

\[
c_t = \frac{H_i}{H_i}
\]

\[
c_d = \frac{H_d}{H_i}
\]

The following condition from energy balance has to be satisfied:

\[
c_r^2 + c_t^2 + c_d^2 = 1
\]

The value of $c_r$, the *reflection coefficient* was computed in a numerical model of the floating caisson in DIANA for a draft to water depth ratio $h_d / d = 0.3 \div 0.5$ (see Figure I2).

The value of the *transmission coefficient* $c_t$ has been measured many times on floating breakwaters. The value is $c_t$ is in the range of 0.4-0.8 and depends on the geometry and dimensions of the breakwater.

The *diffraction coefficient* $c_d$ may be estimated after [41], which have been measured for breakwater standing on the sea bottom.

In the case of a floating structure the diffracted part of the energy is limited by the caisson draft. It is described by kinetic energy ratio $KER$.

\[
KER(z) = 1 - \frac{\sinh(2 \cdot k \cdot (d - z))}{\sinh(2 \cdot k \cdot d)}
\]

For numerical calculation the average diffraction coefficient $c_d$ along the caisson length may be taken from formula
\[ c_r = \text{KER}(h_d) \left[ 5.17 \cdot \left( \frac{L}{l_c} \right)^{0.5} - 4.6 \cdot \frac{L}{l_c} + 0.37 \right] \]  

(I.10)

**Figure I.2:** Reflection coefficient \( c_r \)
**NOTATIONS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_c$</td>
<td>width of caisson</td>
</tr>
<tr>
<td>$c$</td>
<td>reflection, transmission and diffraction coefficients (see Annex A)</td>
</tr>
<tr>
<td>$C_p$</td>
<td>pressure coefficient on the walls (see figure 6)</td>
</tr>
<tr>
<td>$C_f$</td>
<td>friction coefficient on the caisson top (see figure 6)</td>
</tr>
<tr>
<td>$C_D$</td>
<td>drag coefficient</td>
</tr>
<tr>
<td>$d$</td>
<td>water depth</td>
</tr>
<tr>
<td>$e_t$</td>
<td>arm of the towing force</td>
</tr>
<tr>
<td>$f_{ct}$</td>
<td>the tensile strength of concrete</td>
</tr>
<tr>
<td>$F_t$</td>
<td>average towing force</td>
</tr>
<tr>
<td>$F_{pz}$</td>
<td>pressure force</td>
</tr>
<tr>
<td>$F_{iz}$</td>
<td>inertial force in the heaving direction</td>
</tr>
<tr>
<td>$F_{wd}$</td>
<td>drag force due to wave generation</td>
</tr>
<tr>
<td>$GM$</td>
<td>metacentric height for longitudinal direction</td>
</tr>
<tr>
<td>$h_c$</td>
<td>caisson height</td>
</tr>
<tr>
<td>$h_d$</td>
<td>caisson draught</td>
</tr>
<tr>
<td>$h_e$</td>
<td>emerged height of caisson = $h_c - h_d$</td>
</tr>
<tr>
<td>$h_i$</td>
<td>displacement due to roll</td>
</tr>
<tr>
<td>$h_g$</td>
<td>distance between still water level and centre of rotation ($\approx 0.1 h_d$)</td>
</tr>
<tr>
<td>$H_s$</td>
<td>significant wave height</td>
</tr>
<tr>
<td>$I_{\theta}$</td>
<td>hydrodynamics added moment of inertia</td>
</tr>
<tr>
<td>$I_{xx}$</td>
<td>mass moment of inertia about the x axis</td>
</tr>
<tr>
<td>$k$</td>
<td>wave number</td>
</tr>
<tr>
<td>$l_c$</td>
<td>length of caisson</td>
</tr>
<tr>
<td>$L$</td>
<td>wave length</td>
</tr>
<tr>
<td>$L_w$</td>
<td>length parameter $\approx 220$ m</td>
</tr>
<tr>
<td>$m$</td>
<td>mass of caisson</td>
</tr>
<tr>
<td>$m$</td>
<td>number of repetitions during operation time (e.g. number of waves)</td>
</tr>
<tr>
<td>$m_r$</td>
<td>resistant moment in wall or bottom</td>
</tr>
<tr>
<td>$m_a$</td>
<td>loading moment in wall or bottom</td>
</tr>
<tr>
<td>$m_h$</td>
<td>hydrodynamic added mass in heave motion (see Annex A)</td>
</tr>
<tr>
<td>$M_{i0}$</td>
<td>moment caused by inertial forces</td>
</tr>
<tr>
<td>$M_D0$</td>
<td>damping moment caused by wave generation due to rolling motion</td>
</tr>
<tr>
<td>$M_B0$</td>
<td>restoring moment due to displacement of centre of buoyancy</td>
</tr>
<tr>
<td>$M_{ip}$</td>
<td>moment due to wave pressure forces at the caisson periferi</td>
</tr>
<tr>
<td>$M_{ws}$</td>
<td>quasi static heeling moment due the wind action</td>
</tr>
<tr>
<td>$M_{wd}$</td>
<td>dynamic heeling moment due the wind action</td>
</tr>
<tr>
<td>$n$</td>
<td>number of spots, where the contact occurs</td>
</tr>
<tr>
<td>$N_Z$</td>
<td>damping factor (see Annex A)</td>
</tr>
<tr>
<td>$N_0$</td>
<td>damping factor of roll</td>
</tr>
</tbody>
</table>
\( p_s(\ ) \) = water pressure
\( P_f \) = failure probability
\( r_{\text{max}} \) = maximum of \( m \) standard Rayleigh distributed variables:
\( T_z \) = zero crossing wave period
\( u_z \) = fluid vertical displacement
\( v_{\text{m}} \) = mean wind velocity
\( W \) = weight of breakwater
\( W_b \) = ballast weight
\( X_i \) = random variable
\( z \) = vertical co-ordinate from the centre of gravity
\( Z \) = instantaneous vertical position of the caisson
\( Z_e \) = vertical displacement of exterior wall (heave coupled with roll)
\( \theta \) = angle of inclination
\( \delta \) = angle of towing direction
\( \psi \) = rotation displacement of fluid
\( \zeta_\theta \) = damping ratio for roll (if not available for pitch)
\( \zeta_Z \) = damping ratio of heave motion
\( \phi_Z \) = phase angle
\( \gamma_w \) = unit weight of water
\( \rho_{\text{air}} \) = density of air = 1.023 kg/m\(^3\)
\( \rho_w \) = water density
\( \omega \) = gust frequency
\( \omega_p \) = eigenfrequency in pitch
\( \omega_p \) = peak energy frequency
\( \omega_Z \) = eigenfrequency of heave motion
\( \beta \) = roughness coefficient \( \approx 6.5 \)
\( \beta \) = reliability index
\( \tau \) = response variable (e.g. stress)
\( \tau_{\text{limit}} \) = limit value of the response variable (e.g. strength)
\( \tau_{\text{max}} \) = maximum value of the response variable during the operation period \( t_{\text{op}} \)
\( \tau_{\text{aver}} \) = mean value of the response variable (e.g. stress due to self weight)
\( \sigma_t \) = tension stress, due to the various loadings
\( \sigma_z \) = standard deviation of fluctuating part of the response variable
\( \eta_f \) = maximum water elevation for the diffracted wave = \( c_t \eta_i \)
\( \eta_i \) = maximum water elevation for the incident wave
\( \eta_r \) = maximum water elevation for the reflected wave = \( c_r \eta_i \)
\( \eta_s \) = instantaneous elevation of water surface above still water level on the seaward side of caisson
\( \eta_t \) = maximum water elevation for the transmitted wave = \( c_t \eta_i \)
CHAPTER 4: IN-SERVICE BEHAVIOUR OF CELLULAR REINFORCED CONCRETE CAISSONS UNDER SEVERE WAVE IMPACT

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ABSTRACT

This report describes some of the current analytical techniques which may be used by structural engineers to assess the behaviour of cellular RC caissons under severe wave impact. Analyses of this type may be required in order to check the preliminary design calculations when fixing the size of the structural elements and determining the amount of reinforcement needed to allow the structure to resist the loads as intended. Analyses are also needed when assessing the integrity of existing structures following accidental damage or deterioration. Simplified models (and highly simplified Limit State Functions) may be generated from the more detailed analyses in conjunction with field data on the actual performance of full-scale structures (and, in some cases, experimental model testing).

After identifying some possible failure modes, and examining the role of each of the key structural elements in transmitting the load to the foundation, this Section describes a 3 Degree-of-Freedom Dynamic model which simulates the linear flexure of the front wall of a caisson structure under wave impact. The deflection is linked to the dynamic bending moments acting in the wall. The influence of the stiffness of the foundation on the wall response is illustrated in a series of simulations. Following this, two types of shell analyses are presented and described in some detail. The role of the support conditions on the front wall is explored using these models and the effects of material and geometric non-linearity explored. A simplified visco-plastic crack model is used to represent fracturing in the wall in the time-domain non-linear shell analysis code.

The features of a new 3-dimensional Finite Element continuum analysis code are then described. This part of the report includes discussions on the appropriate choice for the time-stepping scheme, the complete non-linear solution strategy, the solver for very large systems of equations and the State-of-the-Art constitutive of concrete with an overview of the important issues of mesh sensitivity and localisation. This part of the report finishes with a discussion on the introduction of fluid elements to treat the coupled fluid-structure interaction.
and recent advances in the dynamic modelling of the far-field to account for the radiation
damping effects in both the soil and the water. Finally the capabilities of a dedicated FE pre-
processor to rapidly generate 3-dimensional meshes is presented.

1. INTRODUCTION AND IDENTIFICATION OF TYPICAL CAISSON FORMS

Prior to presenting some possible structural failure modes and analysis methods, it is appro-
priate to briefly review the three basic phases of structural design. The first stage consists of
devising an overall structural scheme which meets the intended use, is safe, constructible and
economically viable. The caisson arrangement will be influenced by the placing and trans-
portation method adopted (for example, lowered by cranes from barges, carried by an
overhead crane running on the existing caissons, floated out, formed partly in-situ). This, in
turn, may be controlled by the availability of local materials (for example, aggregate), skills
and a suitable pre-casting site. The second stage comprises performing the initial calculations
to determine the approximate sizes of the structural components and estimate the cost of the
materials, temporary works and develop a more detailed construction method and sequence.
Simplified rules are often used to quantify loadings and idealised models are used to ap-
proximate the manner in which the structure carries the loads. In the final stage, the adequacy
of each structural member is assessed for a full suite of possible load cases. Detailed checks
on internal resistance, structural deformations, reinforcing arrangements and materials
specification form part of the third phase.

In the case of a caisson breakwater, the conceptual design will identify the constrains
operating at the particular site. The overall dimensions and geometric form of the structure
will typically be dictated by geomechanical and hydraulic conditions. For example, the
frictional resistance which can be mobilised at the foundation-base interface (to prevent rigid
body sliding) will control the width of the base slab. The height of the structure will be
governed by the structure's intended purpose, the tidal range and the maximum wave height to
be resisted without over-topping.

1.1. Planar rectangular multi-celled caissons

A typical structural arrangement for a cellular, rectangular, reinforced concrete caisson with a
vertical face is illustrated in Figure 1. The diagram shows an isometric view of one half of a 7
by 4-celled caisson.

This form of caisson typically comprises 8 different types of load-bearing elements (i) the
front wall, (ii) rear wall, (iii) side walls (not shown), (iv) internal walls, (v) base slab, (vi) top
slab, (vii) crown wall and (viii) shear keys (not always present).
Figure 1. Isometric view of one-half of a caisson, Genoa Voltri, Italy

The planar front wall in this class of structure reflects the incident wave. Internal cell sizes are usually of the order of 4 to 5m square, although circular (cylindrical) internal cells have been used successfully on some projects (for example, the extension to the Reina Sofia breakwater at Las Palmas, Spain). Use of the latter can result in a smaller reinforcement requirement as the loads are transferred through the caisson more by compressive arching action, than flexure. The walls are usually slip-formed either continuously, or in distinct lifts.
Figure 2. Schematic view of different caisson types
1.2. Perforated rectangular multi-celled caissons

Perforated caissons are becoming increasingly popular because they can create a more tranquil sea state in front of the structure (due to reduced reflections) and also lead to reduced material costs although the latter may be offset by increased formwork costs.

The relative area of the perforations with respect to the total frontal area typically lies in the range 25 to 40%. Both circular and rectangular apertures have been used. In the case of the breakwater at Dieppe (France), the front and rear faces (as well as the top slab) have circular holes, whereas the internal walls have large rectangular perforations. Internal cells in a perforated caisson often have a thick layer of (non-structural) concrete ballast to add stability to the base of the structure.

![Circular caissons](image)

Figure 3. Circular caissons

1.3. Circular-fronted caissons

Circular fronted caissons do not require such large wall thicknesses as rectangular caissons because the external wave pressure is transmitted to the foundation by in-plane compression (that is, through compressive membrane stresses) rather than flexure. As examples, the Hanstholm (Denmark) and Brighton (UK) breakwaters have similar circular forms, whereas the Duca degli Abruzzi breakwater in Naples (Italy) exhibits a hybrid rectangular-circular footprint. Whilst total impact forces may be reduced on circular caissons, care is needed to avoid wave trapping and local high pressures in the clutches where two neighbouring caissons meet. In the case of the Hanstholm and Brighton caissons, the units were lowered into position by means of a rail mounted gantry crane straddling adjacent caissons.
1.4. Alternative designs

There is a growing tendency to adopt hybrid caisson forms in new breakwater designs to optimise the solution. Thus, perforated circular front walls with an open structure to the front cells can be combined with a planar rear wall which has smaller perforations. One problem with adopting a perforated structure lies with difficulty in obtaining reliable design pressure intensities as a result of the highly turbulent flow within the cells.

2. STRUCTURAL FAILURE MODES AND LOAD ACTING ON THE CAISSON

The geometric form of reinforced concrete caissons includes the simple rectangular boxes as well as complex multi-celled arrangements with circular front and rear walls and perforations. Each structure is designed within the constraints of the specific site, the local conditions, the intended purpose and the funding available. The multitude of structural forms makes generalisation of the structural behaviour difficult, however there are some common, basic failure mechanisms. The role of each of the key elements is described below. The manner in which the load is transferred through an element down to the foundation dictates the potential failure mode. When bending stresses dominate, flexural cracking and concrete crushing is possible. If local shear stresses are too high, a punching mechanism may occur under overload. Large scale twisting of the structure (possibly through differential settlement, or damage during the float-out phase) may give rise to diagonal torsional fractures. Figures 4 and 5 identify some of the mechanisms although clearly not all modes may occur in any one structure.
Figure 4. Some possible failure modes in RC caissons

In principle, once in service, the failure of concrete caisson breakwaters to provide tranquil water within a harbour by breaching the sea wall may be the result of both large-scale rigid body translation of the structure (due to global sliding at the base-foundation interface or rotational collapse of the foundation) and local rupture in the structural elements. The latter requires a sequence of damaging events to lead to a failure state.
The progressive loss in structural integrity may start by chloride ingress in the splash zone of the face of the breakwater. Small cracks may be present in the front wall, near the transverse cross walls, as a result of earlier wave impacts. If unheeded (and if exacerbated by thermal cycling), the chlorides may penetrate to the reinforcing steel, building-up sufficient concentration to provoke the onset of corrosion. Continued corrosion can result in a loss of bond, reduction in steel cross-sectional area, weakening of anchorage and bursting-off of the cover concrete. All these mechanisms can further weaken the reinforced concrete cross-section. If no significant reserve of strength exists at that section, the wall may rupture under repeated storm loading. Without a regular programme of inspection, diagnosis and repair, progressive
deterioration of a wall panel may occur. Should sufficiently large cracks be induced in the front face, then this may lead to a washing-out of ballast in the cells. The ultimate consequence of loosing ballast, will be to reduce the frictional resistance at the foundation-base interface resulting in an increased risk of sliding failure.

In order to prevent such a chain of events, coastal/structural engineers need some guidance on how to assess the likelihood of each mechanism. Note that partial collapse of the front wall or even minor shearing dislocation between caissons could also lead to loss of support and serviceability of the top slab, damaging crane rail-tracks, service ducts and/or vehicle access. The consequent reduction, or loss, of access to the structure may significantly restrict harbour operations without actually resulting in a breech of the sea wall.

Multi-celled reinforced concrete caissons are generally highly redundant structures with several alternative load paths available. Local damage to the sea wall, in the form of the bursting or spalling of concrete will not immediately lead to a critical failure situation. Reasonable engineering judgement must therefore be exercised before structures are condemned just on the basis of unsightly corrosion stains or local loss of cover. In many cases, the structure may go on to provide years of active service before a collapse state is approached.

Once a possible failure mode has been defined, then the loading acting to create the rupture and the resistance of the member need to be quantified such that a realistic assessment of the reserve of strength may be estimated. Provided relatively slender sections are used, then the structural members will be subjected to essentially biaxial states of stress. However, local thickening (chamfers) at junctions between walls and base/top slabs create quite complex restraint conditions for the wall elements, therefore care is needed in arriving at a realistic estimate of the internal stresses acting the structure.

Possible loading during the in-service life include (i) permanent loads resulting from the dead weight of the structure (using submerged densities, where appropriate) and the superstructure as well as the horizontal soil pressure from the fill inside the cells and from the foundation reaction (ii) variable loads arising from changes in the water level, from pulsating and impact loads (including up-lift effects under the base slab) and over-topping wave loads as well as superimposed harbour traffic loads (iii) accidental loads resulting from boat impacts during mooring and falling masses during cargo loading/unloading operations. Clearly, in regions where seismic activity occurs, the earthquake induced ground motions can lead to structural distress.
3. IDENTIFICATION OF STRUCTURAL IDEALISATIONS

Before individual structural members are designed, the load paths must be identified and the basic global structural action understood. It is not possible to treat the structure as an equivalent 2-d plane strain problem because of the arrangement of cross walls which stiffen a rectangular caisson (this also holds for circular caissons, for obvious reasons). The three-dimensionality of the structure therefore needs to be taken into account in order to appreciate the manner in which the forces are transmitted through the walls to the base slab, and down through the foundation.

The front wall will be subjected to horizontal pressures acting outwards (due to the ballast fill in the cells) at low tide\(^1\) and horizontal pressures acting inwards when struck by a storm wave. This loading will cause a rectangular panel to act rather like a one-way horizontally spanning slab supported along its length by the vertical cross-walls (Figure 6).

This one-way action only holds for horizontal strips remote from the top and bottom slabs (that is, at least one span width above the base slab and below the top slab). Close to the base and top slabs, the action is essentially two-way and the deflections will be reduced. It is worth pointing out that the maximum pressure from the internal ballast will occur near the base slab whereas the maximum pressure from the wave loading will typically occur near the top slab. For the purpose of a preliminary sizing of the front wall, the peak wave pressure\(^2\) may be considered to be acting uniformly over a horizontal strip across the caisson face. A unit width beam continuously supported over the internal cross-walls may be analysed to determine the wall thickness and maximum percentage of reinforcement required. Note that the ability of the wall to resist the outward pressure from the ballast alone must be considered as an important load case.

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\(^1\) or when a wave trough occurs in front of the wall.
\(^2\) minus the active ballast pressure, if the fill is in full contact with the wall near the top.
The stiffening effect in the wall due to the fill material behind it, as a wave strikes the front face, is quite difficult to accurately assess; although simple calculations indicate that this effect will be small and so it is generally neglected. It must be remembered that the front wall may also carry a moderately high vertical axial load and bending moments from the top slab. The compression loading will come from the weight of the crown wall and top slab and self weight of the front face, in addition to some proportion of any live load acting on the top slab.

The \textbf{rear wall} will be subjected to a similar loading regime as the front wall except that the wave pressures will be very much reduced. Berthing forces could, however, be significant for a harbour quay.

The \textbf{side walls} must be designed to retain the ballast fill and resist in-plane shear stresses in order to transfer the horizontal loads from the front face to the base slab. The in-plane shear stiffness will generally be so high as to render these stresses very small. Depending on the degree of inter-connectivity between adjacent caissons, the side walls may also be required to resist the local horizontal forces carried by the vertical shear keys and the (relatively minor) wave impacts in the clutches.

The \textbf{internal cross-walls} will carry the vertical loads from the top slab to the foundation and contribute to the transverse stiffness of the caisson box by transferring the horizontal forces (mobilising the transverse, front-to-back, wall's in-plane stiffness) from the external walls to the base slab. These walls should be designed to support the ballast fill pressures assuming no fill in the neighbouring cell\textsuperscript{3}. The presence of the internal transverse and longitudinal walls

\textsuperscript{3} This condition could occur during the placing stage.
add greatly to the torsional rigidity of the caisson, an important consideration during the float-out and towing phases.

The **base slab** will be subjected to vertical pressures acting upwards from the supporting foundation and uplift water pressure during a wave impact. These loads will be in equilibrium with the downward forces arising from the weight of the caisson. The vertically downward loads will be transmitted to the base slab via the walls and ballast. Bending moments resulting from horizontal pressures acting on the walls will also be carried into the base slab. A *Beam-on-an-Elastic Foundation* model could be used to determine the design moments and shear forces, however a simpler one-way spanning (front-to-back) beam model could also be used for the preliminary design. Moments acting at each of the side-to-side internal wall locations could be shared according to the effective lateral stiffness of the front-to-back internal walls. The base slab must also be able to withstand the bending moments and shear forces induced as a result of the structure receiving only partial support from the foundation.

The **top slab** will typically be required to withstand harbour traffic loads (including crane forces) and any loads resulting from vertical wave slamming during over-topping. Depending on the aspect ratio, l/d, of the internal cells, the top slab may be designed either as spanning one-way (l/d <2) or two-way (l/d >2) over the internal walls. The top slab can provide very considerable transverse stability to a cellular caisson by virtue of its high in-plane stiffness. This membrane action contributes to the distribution of the horizontal forces (acting on the front face) out to other internal and external walls. The top slab may be cast directly on the ballast fill, or formed by casting a thinner in-situ reinforced layer over a series of pre-cast slabs or beams. The latter construction technique, although quicker, will leave a void underneath the top slab.

The **crown** (or sea) **wall** and associated super-structure must be designed to resist a severe storm wave crashing onto its vertical face without inducing significant damage. This element is subjected to the largest temperature variations. Depending on the location of the breakwater, the concrete may be exposed to temperatures below freezing, or temperatures up to 40°C. Structurally, the crown wall may be treated as either a simple, monolithic gravity element, or a vertical cantilever depending on its relative slenderness. In either case, the horizontal load may be idealised as being transferred by the shear resistance acting at the horizontal interface between the base of the crown wall and the top slab.

If present, **shear keys** form a mechanical interlock which is designed to share the load between adjacent caissons. These are considered to be highly desirable. In one approach, transfer of the horizontal wave loads is achieved by relying on the concrete's shear resistance (in a vertical plane) over the full height of the key. A second, preferred approach, is to introduce a granular fill into the gap between caissons (over the full height and most of the width) to mobilise the frictional resistance of the confined material. This technique places fewer restrictions on the precision of the geometric alignment needed between neighbouring caisson units.
A reinforced concrete caisson with a carefully designed concrete mix and having sufficient attention paid to reinforcing details at laps and corners will provide many years of excellent service if appropriate supervision was provided during the construction phase. Looking to the future, there is now scope for caissons to significantly increase in size. Lengths of over 100m are perfectly plausible provided the global bending and torsional stiffnesses are sufficient to cope with the loads induced during towing.

This report focuses on the behaviour of the front face of a vertical breakwater as this member is considered the most critical element in a cellular RC caisson by virtue of receiving the largest wave loading. Of course, all components of the structure will need to be carefully designed when detailing a complete RC caisson.

An important philosophy of this report is to emphasise that there exists a hierarchy of structural models which may be used to examine the caisson’s response. These range from the simple beam analogies to fully non-linear, coupled, 3-dimensional, dynamic Finite Element fluid-soil-structure interaction analyses. It is considered here that the most appropriate method of analysing such a structure involves careful use of the Finite Element Method coupled with sectional analysis and sound engineering judgement. Because of the relatively complex load sharing which takes place within a cellular caisson, most design engineers undertake linear finite element analyses to determine maximum bending moment and shear force envelopes. This technique has transformed the way in which structures have been designed during the past 35 years. Today, even the smallest design office can gain access to a general purpose linear analysis FE programme. However, despite enormous increases in the processing power of modern computers, full three-dimensional dynamic analyses for impact problems demand significant computer resources. For this reason, simplified approaches based on the assumed behaviour of individual elements, are still used in the preliminary design stage. Unfortunately, no single simple analytical model is relevant for all caisson structures. The justification for the simplified structural idealisations is examined first. The initial approach is based on a 3 degree-of-freedom, lumped parameter, dynamic model. This class of model can help the engineer assess whether a dynamic analysis is warranted for the front wall.

Considering an equivalent unit width beam (with no compressive reinforcement), spanning one-way continuously over at least 6 equal-span cells, a highly simplified Limit State Equation for flexural failure in an under-reinforced section may be given by

\[ g_1 = g_1(\rho_r, d, f_y, \alpha, f_{ck}, p, L) = \rho_r d^2 f_y (1-(0.4\rho_r f_{ck}/\alpha f_{ck}))-0.08pL^2 \]

where \( \rho_r \) is the area ratio of steel reinforcement with respect to the concrete cross-sectional area \( D(0.015-0.04) \), \( d \) (see Figure 7) is the depth of the section from the compression face to the centre of the tensile steel reinforcement \( D(0.25-1.5m) \), \( f_y \) is the characteristic yield
strength of steel reinforcement LN(460MPa, 10MPa), $\alpha$ is a coefficient which takes account of the long-term affects on the compressive strength and of the unfavourable effects resulting from the way in which the load is applied (adopt $\alpha=0.85$ as a default value), $f_{ck}$ is the characteristic compressive strength of concrete LN(40-60MPa, 4-8MPa). EC2 denotes a concrete with a characteristic cylinder strength of 30MPa and a characteristic cube strength of 37MPa, as grade C30/37 concrete. Other grades include C35/45, C40/50, C45/55 and C50/60. Concrete of grade at least C40/50 should generally be used in a marine environment to limit the chloride diffusion. $p$ is the net uniformly distributed pressure acting on the member (in the case of the front wall, $p$ is the arithmetic sum of the applied wave loading and the internal cell pressure. Finally, $L$ is the effective span distance between the supports. The factor 0.08 is chosen as a representative value for the maximum (mid-span) bending moment occurring in the middle of the outer-most span of a caisson with 6, or more, cells.

![Figure 7. Idealised Stress Block for a Reinforced Concrete Beam under Flexure](image)

The following expression applies to the shear state in a beam at a distance $d$ from the edge of the support wall. The factor 0.6$pL$ corresponds to the maximum shear force experienced in the outer-most span, nearest the internal support.

\[
g_2 = g_2(f_c, d, \rho_l, \sigma_{cp}, p, L) = (0.0525f_{ck}^{2/3}(1.6-d)(1.2+40\rho_l)+0.15\sigma_{cp})d-0.6pL
\]

where $\rho_l$ is the lesser of longitudinal tension reinforcement ratio and 0.02, $D(0.005-0.02)$, $\sigma_{cp}$ is equal to $N/A_g$ where $N$ is the axial force and $A_g$ is the gross area of the cross section. If punching shear is to be checked in a slab or wall, then the term $-0.6pL$ is replaced by the actual level of shear force acting on the loaded area, the term $0.15\sigma_{cp}$ is not included and the term $(1.2+40\rho_l)d$ is now multiplied by $b_w$, the length of the critical shear perimeter.
Cracking in concrete members in a seawater environment will accelerate the rate of chloride penetration and thus speed up the rate at which corrosion may first appear. Therefore, it is necessary to pay particular attention to prevent the development of cracks with widths of 0.3mm of more.

\[
g_3 = g_3(\eta, f_{ij}, \sigma_t) = 90 (\eta f_{ij})^{1/2} - \sigma_t
\]

where \( \eta \) is a coefficient which is equal to 1 for normal (round) bars and 1.6 for deformed bars, \( f_{ij} \) is the characteristic tensile strength of concrete (in MPa) and \( \sigma_t \) is the actual stress in the tensile reinforcement (also in MPa). Note that if \( 90 (\eta f_{ij})^{1/2} \) is greater than \( 0.5f_e \), then \( \sigma_t \) should be compared against \( 0.5f_e \) where \( f_e \) is the stress in the reinforcement corresponding to the end of the elastic phase.

The final simplified LSE describes the rate of Chloride penetration through concrete sections as follows

\[
g_4 = g_4(C_{cr}, C_o, x_c, D_c, t_l) = C_{cr} - C_o(1 - \text{erf}(x_c/(2D_c t_l)^{1/2}))
\]

where \( C_{cr} \) is the critical chloride ion density (in kg.m\(^{-3}\)) when corrosion starts at the surface of the reinforcement, \( C_o \) is the measured chloride ion density at the surface of the concrete, \( x_c \) is the depth of the concrete cover, \( D_c \) is the chloride diffusion coefficient (in m\(^2\).s\(^{-1}\)) and \( t_l \) is the lifetime of the structure (or the time at which an assessment is to be made).

As noted above, a cellular caisson with sufficient flexural and shear reinforcement (with attention paid to detailing for shrinkage, corners, laps and joints) offers a multitude of load paths to transmit the forces. This is particularly so for many of the older existing caissons, as wall sections tend to have been over-sized as a result of over-conservative, simplified analyses. Local spalling and even significant corrosion in certain areas can often have little real effect on the overall stability of the structure. One concept which is not always appreciated is that by increasing the cover to the reinforcing steel in a flexural member, one is not automatically improving the durability of the section as the likelihood of cracking on the tensile face is increased.
4. STRUCTURAL DYNAMICS

The governing equation of motion for a linear dynamic multi-degree-of-freedom system (such a discrete, lumped parameter model or finite element model) is given as:

\[
[M] \ddot{d} + [C] \dot{d} + [K] d = f
\]

Where \([M]\) is the system mass matrix, \([C]\) is the damping matrix and \([K]\) the stiffness matrix. \(f\) is the vector defining the forcing function (with, or without body-loads) and \(d\) is the resulting displacement with the single and double over-dots representing velocity and acceleration respectively. If non-linear phenomena are present then a direct time-integration scheme must be used to solve for the unknowns.

In practice, lumping of the structural masses to create a diagonal mass matrix is often employed. This can lead to attractive, efficient integration schemes base on explicit methods. In the University of Sheffield code \(yaFEc\) the mass lumping is achieved by scaling the diagonal terms in the consistent element mass matrices in proportion to the total element mass. Note that lumping masses may lead to inaccurate results in the case of coarse FE meshes and/or irregular shapes.

The presence of damping in reinforced concrete structures is less important for very short duration loading events, such as impact or blast problems, however if the longer term response is needed then damping should be included. Damping may arise from a number of sources; (i) material damping as a result of internal friction creating hysteresis loops in the material stress-strain curves, (ii) frictional effects on a larger scale either due to contact/loss of contact at the supports or sliding at the soil-structure interfaces and (iii) radiation damping through the soil and water. In many cases these different frictional effects are collected together and treated as single effect through introduction of Rayleigh Damping. In this case the \([C]\) matrix is defined as the summation of a mass term \(\alpha[M]\) and a stiffness term \(\beta[K]\). The constants \(\alpha\) and \(\beta\) may be determined from the expected damping ratios at two different frequencies. Whilst this method is relatively simple to apply, it is considered preferable to identify the true source of the damping and model it properly. Note that the relative merits of different time-stepping (integration) algorithms are discussed elsewhere in this report.

5. SIMPLIFIED 3-DOF DYNAMIC MODEL OF DEFORMATION OF FRONT WALL

In order to examine whether a full dynamic model is justified when analysing the front face of a vertical caisson breakwater subjected to a wave impact, a simplified 3-degree of freedom model may be used (Figure 8). This model builds upon the elastic translational and rotational models developed by Oumeraci and Kortenhaus and Pedersen. Two of the degrees of freedom
correspond to the rigid body horizontal translation and rotation; the third degree of freedom represents bending of the front wall.

The dynamic equation of motion for such a 3-DoF system is given by

\[
\begin{bmatrix}
    m - m_w & 0 & 0 \\
    0 & \Theta_t & 0 \\
    0 & 0 & m_w
\end{bmatrix}
\begin{bmatrix}
    \ddot{x} \\
    \ddot{\phi} \\
    \ddot{x}_w
\end{bmatrix}
+ \begin{bmatrix}
    c_s & -c_s l_{h_4} & 0 \\
    -c_s l_{h_4} & c_\phi + c_s (l_{h_4})^2 & 0 \\
    0 & 0 & m_w
\end{bmatrix}
\begin{bmatrix}
    \dot{x} \\
    \dot{\phi} \\
    \dot{x}_w
\end{bmatrix}
+ \begin{bmatrix}
    k_x + k_w & -k_s l_{h_4} & -k_w \\
    -k_s l_{h_4} + k_w l_{h_2} & k_\phi + k_s (l_{h_4})^2 & -k_w l_{h_2} \\
    -k_w & 0 & k_w
\end{bmatrix}
\begin{bmatrix}
    x \\
    \phi \\
    x_w
\end{bmatrix}
= \begin{bmatrix}
    f_{h_1} + f_{h_2} \\
    f_{h_1} l_{h_2} - f_{h_1} l_{h_4} + f_v l_v \\
    f_{h_2}
\end{bmatrix}
\]

where \( m \) is the total mass of the caisson, \( m_w \) is the mass of a full height rectangular panel on the front face, \( \Theta_t \) is the rotational inertia of the caisson, and \( x \) and \( x_w \) represent the horizontal displacement of the body of the caisson (minus the front wall) and displacement of the caisson including the front wall, respectively (over-dots and double over-dots signify first and second order differentiation with respect to time). \( \phi \) indicates the rotation of the caisson and \( c_s \) the damping of the coupled foundation/fluid. \( L_{h_4} \) is the level arm length between the point of horizontal reaction and the centre of rotation, \( c_\phi \) is the rotational damping. \( k_x \) is the foundation stiffness, whereas \( k_w \) is the stiffness of the front wall and \( k_\phi \) is the rotational stiffness.
Figure 8. Diagram of 3-DoF Dynamic Lumped Parameter Model

Vertical equilibrium and motion are not addressed in this model (the rubble mound reaction equates to the vertical load resulting from the structure's self weight). The horizontal load is split into three parts; upper and lower forces which do not bear onto the front wall ($f_{h1}$ and $f_{h3}$), and a mid force which acts on the wall ($f_{h2}$). $l_{h1}$ is the vertical lever-arm distance from centroid of the caisson to $f_{h1}$, $l_{h2}$ is the vertical lever-arm distance from centroid of the caisson to $f_{h2}$ (shown as zero in Figure 8) and $l_{h3}$ is the vertical lever-arm distance from centroid of the caisson to $f_{h3}$. $f_v$ is the vertical uplift force and finally $l_v$ is the horizontal lever-arm distance from centroid of the caisson to $f_v$.

The model has been coded using a Newmark time-integration scheme using the following MATLAB script.

```matlab
%thickness_of_front_wall=0.7; E=25e9; Erub=200e6; nu=0.5;
G=Erub/(2.0*(1+nu)); %rubble G=67-230MPa

A=444.97; B=18.5; d_w=20; height_of_front_wall=19.55;
L=30.1; Lc=4.18; Lc_eff=0.4654*Lc; % Note: Lc should be reduced to 4m?
rho_cai=2150; rho_con=2450; rho_s=2000; rho_w=1025;

m_cai=rho_cai*A*L; m_hyd=1.40*rho_w*(d_w^2)*L;
R1=sqrt(B*L/pi); m_geo=(0.76*rho_s*(R1^3))/(2-nu);

m_tot=m_cai+m_hyd+m_geo; m_tot=m_tot*(Lc/L);
```

- 18 -
\[
\Theta_{cai} = 2.28329 \times 10^9; \quad \Theta_{hyd} = (0.122 + 0.063) \rho_w (d_w^4) L; \quad \Theta_{geo} = 0.64 \rho_s (R2^5) \quad \Theta_{tot} = \Theta_{cai} + \Theta_{hyd} + \Theta_{geo}; \\
\Theta_{tot} = \Theta_{tot} (Lc/L); \\
m_{wall} = \rho_{con} \cdot Lc_{eff} \cdot \text{thickness of front wall} \cdot \text{height of front wall}; \\
beta_x = 1.1; \quad \beta_{th} = 0.46; \\
k_{x} = 2(1+n)G^* (\sqrt{B_L}) \beta_{x}; \quad k_{x_{\text{temp}}} = k_{x}; \quad k_{x} = k_{x}/(Lc/L); \\
k_{th} = (G/(1-n)) \beta_{th} (B^2) L; \quad k_{th_{\text{temp}}} = k_{th}; \quad k_{th} = k_{th}/(Lc/L); \\
p_{max} = 660000; \quad I = (1/12) \text{height of front wall} \cdot (\text{thickness of front wall}^3); \\
Lxm = 28/Lc/71; \quad W = p_{max} \cdot L \cdot \text{height of front wall}; \quad k_{wall} = 254.33 \times E^1/(Lc^3); \\
lsd = 0.00; \quad a = 12.09 - lsd; \quad nth = 1.219; \quad Bx = (7 - (8 \times n)) m_{tot}/(32(1-nu) \times \rho_s (R1^3)); \\
Dx = 0.288/\sqrt{Bx}; \quad C_x = Dx \times 2 \times \sqrt{k_{x_{\text{temp}}} \times \Theta_{tot}}; \quad C_{x} = C_{x}/(Lc/L); \\
\beta_{x} = 3(1-n) \cdot (\Theta_{cai} + (m_{cai} \times (a + lsd^2)))/(8 \times \rho_s (R2^5)); \quad \\
D_{th} = 0.15/(1+(nth \times Bth)) \times sqrt(nth \times Bth); \\
c_{th} = D_{th} \times 2 \times \sqrt{k_{th_{\text{temp}}} \times \Theta_{tot}}; \quad c_{th} = c_{th}/(Lc/L); \quad c_{wall} = 0.0; \\
\]

\[
\begin{align*}
\text{f}_{\text{max hor}} = \text{pmax}; \quad H = 28.5; \quad \text{wall base height} = 0.95; \\
\text{temp} = (((d_w - (0.2 \times H))/(0.8 \times H) + 1) \times (H - d_w)/2) + (1.45 \times d_w/2); \\
\text{f}_{\text{hor1 percentage}} = ((d_w - (0.2 \times H))/(0.8 \times H) + ... \\
(1.8 \times H - \text{height of front wall} + \text{wall base height}))/2)/temp; \\
\text{f}_{\text{hor2 percentage}} = (0.45 + (0.55 \times \text{wall base height} / d_w))/temp; \\
\text{f}_{\text{hor3 percentage}} = (((d_w - (0.2 \times H))/(0.8 \times H) + 1) \times (H - d_w)/2 + 1.45 \times d_w/2)/temp - \text{F}_{\text{hor1 percentage}} \times \text{F}_{\text{hor3 percentage}}; \\
\text{f}_{\text{max hor}} = ((\text{f}_{\text{max hor}} + (0.45 \times \text{f}_{\text{max hor}})) \times d_w/2 + (\text{f}_{\text{max hor}} + (0.8 \times H - H + d_w)/2) \times (H - d_w))/Lc/L; \\
\text{F}_{\text{max hor}} = \text{F}_{\text{max hor}}/(Lc/L) \times 337000; \quad \text{F}_{\text{max up}} = \text{F}_{\text{max up}}/(Lc/L); \\
\text{lh1} = 12.12; \quad \text{lh2} = 0.19; \quad \text{lh3} = 11.64; \quad \text{lv} = 2.77; \\
M = \begin{bmatrix} m_{tot} - m_{wall} & 0 & 0 \\
0 & Theta_{tot} & 0 \\
0 & 0 & m_{wall} \end{bmatrix}; \\
C = \begin{bmatrix} c_{x} & -c_{x}^{*}a & 0 \\
-(c_{x}^{*}a) & (c_{th} + (c_{x}^{*}a^{*}a)) & 0 \\
0 & 0 & c_{wall} \end{bmatrix}; \\
K = \begin{bmatrix} (k_{x} + k_{wall}) & -(k_{x}^{*}a) & -k_{wall} \\
-(k_{x}^{*}a) + (k_{wall} \times lh2) & (k_{th} + (k_{x}^{*}a^{*}a)) & -(k_{wall} \times lh2) \\
-k_{wall} & 0 & k_{wall} \end{bmatrix}; \\
F_{\text{max}} = \text{zeros}(3,1); \\
F_{\text{max}}(1) = \text{F}_{\text{max hor}} \times (\text{F}_{\text{hor1 percentage}} + \text{F}_{\text{hor3 percentage}}); \\
F_{\text{max}}(2) = (\text{F}_{\text{max up}} \times \text{lv}) + (\text{F}_{\text{hor1 percentage}} \times \text{F}_{\text{max hor}} \times \text{lh1}) - \text{F}_{\text{hor3 percentage}} \times \text{F}_{\text{max hor}} \times \text{lh3}; \\
F_{\text{max}}(3) = \text{F}_{\text{hor2 percentage}} \times \text{F}_{\text{max hor}}; \\
\end{align*}
\]
\[ \text{static_res} = K \max; \stat_{\text{disp w inc r}} = \text{static_res}(3) - \text{static_res}(1) \]
\[ \text{freq_cais} = \left( \sqrt{\frac{(k_x+k_{\text{wall}})}{(m_{\text{tot}} - m_{\text{wall}})}} \right) / (2\pi) \]
\[ \text{freq_wall} = \sqrt{\frac{k_{\text{wall}}}{m_{\text{wall}}}} / (2\pi); \ T_1 = 1 / \text{freq_cais}; \ T_3 = 1 / \text{freq_wall} \]

\[ \delta T = \min([T_1 \ T_3 \ t_d]) / 20; \ \text{steps} = 2.0 \times \max([T_1 \ T_3 \ t_d]) / \delta T; \]
\[ \text{steps} = 5.0 \times t_d / \delta T; \ \text{steps} = \text{ceil}(\text{steps}); \]
\[ \text{displ} = \text{zeros}(3, \text{steps}); \ \text{vel} = \text{zeros}(3, \text{steps}); \ \text{acc} = \text{zeros}(3, \text{steps}); \]
\[ \text{F} = \text{zeros}(3, \text{steps}); \ t = \text{zeros}(1, \text{steps}); \]
\[ \text{displ}(:,1) = [0 \ 0 \ 0]'; \ \text{vel}(:,1) = [0 \ 0 \ 0]'; \ \text{F}( :,1) = [0 \ 0 \ 0]'; \]
\[ \text{acc}( :,1) = \text{inv}(M) \times (\text{F}( :,1) - (C \times \text{vel}( :,1)) - (K \times \text{displ}( :,1))); \ t(1) = 0; \]
\[ \alpha = 1/2; \ \beta = 1/6; \]

\[ \text{for i=1:(steps-1)} \]
\[ \ t(i+1) = t(i) + \delta T; \]
\[ \ \text{if}(t(i+1)\leq t_r) \]
\[ \quad \text{F}_{\text{hor}} = (t(i+1)/t_r) \times \text{F}_{\max \_\text{hor}}; \]
\[ \quad \text{F}_{\text{up}} = (t(i+1)/t_r) \times \text{F}_{\max \_\text{up}}; \]
\[ \ \text{else} \]
\[ \quad (t(i+1)\leq t_d) \]
\[ \quad \text{F}_{\text{hor}} = (\text{t_d}-t(i+1))/(t_d-t_r) \times \text{F}_{\max \_\text{hor}}; \quad \text{F}_{\text{up}} = (\text{t_d}-t(i+1))/(t_d-t_r) \times \text{F}_{\max \_\text{up}}; \]
\[ \ \text{end} \]
\[ \text{else} \]
\[ \quad \text{F}_{\text{hor}} = 0; \quad \text{F}_{\text{up}} = 0; \]
\[ \text{end} \]
\[ \text{F}(1,i+1) = \text{F}_{\text{hor}} \times (\text{F}_{\text{hor1 \_percentage}} + \text{F}_{\text{hor3 \_percentage}}); \]
\[ \text{F}(2,i+1) = \text{F}_{\text{up}} \times \text{lv} + (\text{F}_{\text{hor1 \_percentage}} \times \text{F}_{\text{hor} \times \text{lh1}}) - \]
\[ \quad (\text{F}_{\text{hor3 \_percentage}} \times \text{F}_{\text{hor} \times \text{lh3}}); \quad \text{F}(3,i+1) = \text{F}_{\text{hor2 \_percentage}} \times \text{F}_{\text{hor}}; \]
\[ \text{displ}(:,i+1) = \text{inv}(((1/(\beta \times (\delta T^2))) \times M) + ((\alpha/\beta) \times \text{displ}( :,i)) + \]
\[ \quad ((1/(\beta \times (\delta T^2))) \times \text{vel}( :,i)) + \]
\[ \quad (((1/(2 \times \beta)) - 1) \times \text{acc}( :,i))) + \]
\[ \quad C \times ((\alpha/\beta) \times \text{displ}( :,i)) + \]
\[ \quad (((\alpha/\beta) - 1) \times \text{vel}( :,i)) + (((\alpha/\beta) - 2) \times (\delta T^2) \times \text{acc}( :,i)))); \]
\[ \text{acc}( :,i+1) = \text{displ}( :,i+1) - \text{displ}( :,i) - \delta T \times \text{vel}( :,i)) / (\delta T^2); \]
\[ \text{vel}( :,i+1) = \text{vel}( :,i) + (((1-\alpha) \times \text{acc}( :,i)) + (\alpha \times \text{acc}( :,i+1))) \times \delta T; \]
\[ \text{if}((\text{displ}(3,i+1)-\text{displ}(1,i+1)) > \text{reldispmax}) \]
\[ \text{reldispmax} = \text{displ}(3,i+1)-\text{displ}(1,i+1); \]
\[ \text{end} \]
\[ \text{end} \]
\[ \text{dynamic magnification} = \text{reldispmax} / \text{static disp wall} \]

% 1: translation of caisson (-wall) 2: rotation caisson 3: translation of wall+caisson
\[ \text{plot}(t, \text{displ}(3,:)-\text{displ}(1,:)); \text{ title}'(\text{Displacement of the front wall with time}); \]
\[ \text{xlabel}'(\text{time (s)}); \text{ylabel}'(\text{horizontal displacement (m)}); \]
Figure 9 shows a simulation of the response of Genoa Voltri breakwater during a wave impact where peak pressures of 660kPa are assumed. The natural frequency of the front wall in a typical Genoa Voltri caisson is calculated to be of the order of 140Hz, whereas the frequency of global rotation the structure is approximately 1Hz.

<table>
<thead>
<tr>
<th>$G_{rm}$</th>
<th>200MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_r$</td>
<td>0.01s</td>
</tr>
<tr>
<td>$t_d$</td>
<td>0.0025s</td>
</tr>
<tr>
<td>$d$</td>
<td>0.7m</td>
</tr>
<tr>
<td>$p$</td>
<td>660kPa</td>
</tr>
</tbody>
</table>

Figure 9. Horizontal motion of Caisson under Wave Impact

Figure 9(a) shows the total displacement of the front wall under a triangular pressure pulse of duration, $t_d$, 0.01s and rise time, $t_r$, 0.0025 s. Note that the peak displacement is of the order of 7 mm.

Figure 10. Horizontal motion of Caisson under Wave Impact with longer Rise Time and Impact Duration
Figure 9(b) shows the relative mid-span displacement of the front wall under the same loading. The short duration impact causes a dynamic amplification (in this linear analysis) of approximately 1.63. This reduces to 1.33, if the duration remains constant, but the rise time increases to 0.005s.

Figure 10 shows a second simulation, with a tenfold increased impact duration (from 0.01 to 0.1s), but the same peak pressure. In this case, the global maximum caisson displacements are much larger (approximately 70mm), but the wall deflections are smaller (0.85mm compared to 1.4mm) as the external force no longer excites the wall bending mode so clearly. For this structure, wave impacts with a rise time greater than about 0.075s will lead to dynamic deformations essentially equal to the static deformation (0.85mm). Thus, an equivalent static analysis is reasonable when designing the front wall for moderate to longer duration wave impacts. Note that because of the low natural frequency associated with the rigid body motion of the caisson (compared to that of the wall), changes of even one or two orders of magnitude to the rubble mound stiffness have a negligible influence on the relative wall displacement. Thus, a minimal role is played by the foundation in influencing the wall's maximum bending moments, for the example considered here.

It is important to remember that even though the dynamic bending moments may be higher than the equivalent static moments, it does not imply that a section will fail if it has been design to only just resist the static loads. In order to determine if dynamic rupture will occur, a non-linear analysis is required. The impact load will be on the structure for a very short time. Some of this load will be resisted by the inertial forces and there may be sufficient ductility in the section to partially yield without complete loss of load carrying capability.

6. DYNAMIC FE SHELL ANALYSIS

6.1. Simplified Rectangular Plate Analysis

If the wall panels in a vertical breakwater are planar and rectangular then it is possible to use closed-form solutions for orthotropic Mindlin plates resting on Winkler springs in a dynamic elastic analysis. Use of the convolution integral may be made to obtain the dynamic displacements and moments for arbitrary excitation. The code TRAM (Hinton, and Vuksanovic, 1988) has been used to compare solutions with simpler and more advanced analysis methods. One limitation of this approach (in its current form) is the imposition of only either fully fixed or rotationally free boundary supports; neither of which properly describes the fixity of the front wall across the transverse walls.
6.2. Non-Linear Layered Shell Analysis

Whilst the 3-DoF model captures the basic dynamics of the front face of a caisson, and the Plate Analysis method provides valuable guidance on the response of rectangular panels, the use of a layered shell, explicit FE analysis code gives a more advanced tool for relatively thin walled, curved structures. Within this framework, through thickness cracking may be simulated using an equivalent smeared approach operating at nine Gauss points in each layer of each element. Cracks may open (and close) normal to the shell layers. The rate at which softening (leading to a complete loss of tensile load capacity) occurs is controlled by the Specific Fracture Energy and the inelastic strain rate. Geometric non-linearity has also been included in the following example, to quantify the membrane stiffening effects under increased deformations. Dynamic equilibrium is expressed as

\[ \sum_n [M][\dot{d}] + \sum_n [C][\ddot{d}] + \sum_n [K][d] = \sum_n \{f\} \]

where \([M]\), \([C]\) and \([K]\) are the elemental mass, damping and stiffness matrices respectively, \(\{f\}\) are the external forces and the summation symbol implies addition of each elemental contribution to the global system of equations. The family of enhanced isoparametric shell elements used in the FE code were originally developed by Huang. These elements exhibit superior characteristics (in the sense of a reduced tendency to shear-lock as the shell becomes thinner) over conventional shell elements. The reinforcement is treated as a stiffer layer within the shell. A total of ten layers through the thickness of the shell were adopted (6 layers to represent the concrete and 4 for the steel). A lumped mass matrix scheme is used whereby the element mass is distributed in proportion to the diagonal terms of the consistent mass matrix.

As in the 3-DoF studies, a rise time of 0.0025s and duration of 0.01s was assumed in these analyses. Note from the deformed plot the two-way bending action near the top and bottom of the wall panels. Figure 11 shows the horizontal displacement contours on the front face of a multi-celled caisson subjected to a triangular pressure pulse with a bi-linear vertical distribution. Figure 12 gives the corresponding displacement-time histories. The curve denoted non-linear 0.7m includes both material and geometric non-linearity effects. A second example analysis (using the same pressure pulse) illustrates the effect of using a reduced front wall thickness (linear 0.5m and non-linear 0.5m). Only one half of a single wall panel was considered; the panel being idealised as simply-supported on the bearing edges.

Hydrodynamic added mass and damping effects are neglected here, the latter is considered to have little influence on the peak displacement, which is realised very early in the analysis in this short duration impact.
7. NON-LINEAR DYNAMIC 3-DIMENSIONAL FE CONTINUUM ANALYSIS

Fully 3-dimensional FE non-linear dynamic codes demand very significant computer resources, yet such techniques are needed in many structural analysis problems. For example, in the case of a caisson sitting on a rubble mount, over 20,000 20-noded brick elements may
be needed to represent the structure in sufficient detail. If 1000 time-steps are to be followed and, on average, 10 non-linear iterations are required to reach dynamic equilibrium, then an analysis may take over 100 hours running on the latest generation Unix workstation. Even the condensed results from the analysis may consume well over 1GB of disc storage.

Figure 13. FE Idealisation Fluid-Soil-Structure System

It is strongly recommended that a linear analysis be undertaken prior to performing any non-linear analyses. Linear analyses will provide significant insight into the way the structure is transmitting the loads and the results should be used to verify if the mesh density, alignment and boundary conditions are appropriate.
At the University of Sheffield a new finite element code and caisson pre-processor have been specifically written for the PROVERBS research study. The new code, *yaFEc*, offers a number of unique features to provide a robust advanced simulation tool. The features include the use of a fast pre-conditioned gradient iterative solver within a Hilber-Hughes-Taylor time-stepping algorithm and the use of fully consistent tangent matrices and a Closest-Point return scheme (in a Newton-Raphson non-linear solution approach) for the hardening/softening plasticity model. The microplane constitutive formulation is also included as an optional material model.
Figure 15  Maximum and minimum FE bending moment distributions across the front wall

Figure 14(a) shows the (exaggerated) deformed shape, with horizontal displacement contours superimposed (bending moments, or shear forces could be plotted in a similar manner) for the Genoa Voltri breakwater. In this example, the ballast fill inside the cells has been modelled by 3-d continuum elements. Figure 14(b) shows the comparable results for the situation where the ballast has not been included in the analysis. Both plots show the deformed structure at the same stage of loading under identical pressure pulses ($p_1=660\text{kPa}$, $t_r=0.0025\text{s}$ and $t_d=0.01\text{s}$). Note the much higher bending deformation in the front wall in the second case. Such analyses illustrate the progressive transmission of the pressure pulse through the structure into the foundation. Figure 17 gives the corresponding horizontal displacement-time curves for the (ballast filled) front wall.

The bending moments may be extracted from the Gauss point stress profile through the front wall. The stresses are integrated with respect to the distance through the wall to obtain the resultant forces from which the bending moment may be directly extracted. The maximum bending moments predicted by the FE analyses are approximately one-third those
Figure 16 shows the progressive transmission of the stress wave through a caisson structure without any fill material in the cells and without a top slab. Such a situation may arise during the construction phase immediately after placing. The analysis predicts significant bending in the front and cross walls. Note that the transverse (front-to-back) walls play the dominant role in transferring the load down to the base slab. The contours in Figure 16 plot the maximum resultant displacements at 4 different timesteps.
It should be remembered that all the above 3d FE examples neglect any restraint which may be offered by the adjacent cells and neighbouring caissons as the transverse cross-walls are considered as free to translate in the plane of the slice (but not laterally). Smaller displacements will result if the stiffening effect from the thick side walls and adjacent caissons, plus the spatially localised pressure distribution (rather than long crested assumption) were taken into account.

![Figure 17 Transient wall displacement from 3-d FE analysis](image)

Care should be exercised before reading to much into the direct comparisons between the four structural models (simplified flexural LSE, 3-DoF dynamic model, non-linear FE shell analysis and 3d continuum dynamic FE analysis) because of the slightly different boundary conditions and material constants used in the runs. The bending moment predicted by the flexural LSE, over-estimates the static value given by the 3-DoF model by just 5%. This is because the bending moment factor for a multi-celled rectangular caisson has been rounded-up. The peak dynamic displacement in the wall predicted using the 3-DoF model is approximately 1.5 times the value given by the shell analysis. This is due to the full centre-to-centre span (of 4.18m) being used in the former analysis, whereas the clear span (or 4m) was used in the latter. It is also a result of a low elastic modulus (25GPa) being used in the 3-DoF but not FE shell analysis (where 30GPa was used). The shell analysis predicts a maximum wall displacement of twice that observed in the full 3-dimensional FE analysis. This last disagreement is largely due to the shell analysis assuming simply supported boundary conditions, whereas the continuum analysis considers the restraint generated by the cross walls. When one looks at the total horizontal displacements (3-DoF and 3-dimensional FE continuum analyses), the FE simulation predicts a peak translation of about 14mm, whereas the 3-DoF model gives just 7mm. This is due to the fact that the FE analysis adopted a lower berm stiffness than that used in the 3-DoF model, and the FE analysis reported here neglected the inertial mass contribution from the fluid (this was included, in a simplified manner, in the 3-DoF model). Note that in all three dynamic analyses, the times at which the wall reaches its
maximum relative displacement are similar (approximately 0.005s, for an impact with a rise time of 0.0025s and duration 0.01s).

One may conclude that the three types of dynamic analyses show broad qualitative agreement, yet for this type of structure only the full 3d FE analysis can properly take account of the restraint conditions existing at the junctions of the front wall and the transverse walls. Today, sensitivity analyses using sophisticated 3-d linear (and to a lesser extent, non-linear) FE codes such as ABAQUS, ANSYS, DIANA, LUSAS and DYNA may be undertaken in any design office. Whilst an equivalent static analysis is appropriate for all but the shortest duration impacts (in the case of the rectangular, cellular breakwater described above), structural engineers now have the means to investigate the effect of including, or neglecting, phenomena such as material non-linearity in the soil, loss of contact between the base and the foundation during rocking and reduced steel reinforcement area and softened concrete (to simulate the corrosion). Although the underlying physics controlling the structural response is well understood, comparisons with results from real structures are still needed to provide greater assurance that all the dominant mechanisms have been addressed. Large scale laboratory investigations have shown that there remains more work to be done in the area of fluid-structure interaction. All analyses discussed here have assumed a pressure-time history for the wave impact which is independent of the motion of the structure. After describing the solution strategy and some advanced constitutive models for concrete (the results of the nonlinear analyses are to appear in the PhD theses of Mesmar and Wu) there follows a discussion on how hydrodynamic and radiation damping effects may be included in the analysis.

7.1. Solution strategy

7.1.1. Direct Methods

Whilst iterative methods have been popular over the past 15 years when solving very large systems of equations, recent multi-front solution methods (Davis and Duff, 1997) are re-establishing the direct method as a serious contender when handling sparse, non-symmetric Finite Element equations. The robustness and efficiency of these methods are currently under investigation at the University of Sheffield.

7.1.2. Indirect Method

The FE code yaFEc currently employs a bi-conjugate gradient element-by-element iterative solver with diagonal pre-conditioning. This algorithm can operate on non-symmetric equations and appears to perform well even for highly non-linear problems. Current research is investigating the benefits of extending the method to encompass a stabilised, squared bi-conjugate gradient approach (Barrett et al. 1990).
7.2. Constitutive Models for Concrete

7.2.1. Elasto-Plastic Models

An advanced hardening-softening plasticity model for concrete, based on the Specific Fracture Energy approach adopted at the University of Colorado, has been recently developed at the University of Sheffield (Tahar and Crouch, 1997 and Tahar and Crouch, 1998). This formulation overcomes some of the difficulties present in the original Colorado models; namely the presence of singularities in the yield functions at the point where they cross the hydrostatic axis and the lack of a fully consistent tangent material matrix coupled to a Closest Point projection algorithm. The latest version of this constitutive model is currently undergoing material point simulation trials and will be incorporated into the general purpose FE code yaFEc during 1999. Investigations into the robustness of the stress return algorithm have revealed a very stable strategy now exists even for trial states which exist well outside the yield surface.

7.2.2. Other Modelling Frameworks

The microplane model for concrete is considered to currently represent one of the most accurate multiaxial constitutive formulations. Originally developed at Northwestern University, USA, this model has recently been the subject of a detailed examination at Sheffield University. Unlike the original (incrementally cast) microplane model (Bazant and Oh, 1985 and Bazant and Prat, 1988a 1988b), the explicit version (Carol et al., 1992) provides a direct relationship between the total stress and total strain measures. The microplane strains themselves are the resolved components of the macroscopic strain tensor. The microplane stresses are related to the microplane strains through a series of empirical expressions which allow the deviatoric and shear microstresses to undergo softening both in compression and tension. The macroscopic volumetric response also exhibits softening in tension, but nonlinear hardening in compression. In its explicit version, the model has 14 material constants. Qiu (1999) has developed a systematic calibration procedure to determine the material constants given conventional or unconventional multiaxial test data.

The basic scheme for computation of the macroscopic stresses (given the macroscopic strain state) can be summarised as follows. The microplane strains are evaluated from the macroscopic strains using the decomposition onto the plane. Then the microplane stresses are computed and finally, the new macroscopic stress tensor is obtained by summating the microplane stresses using a numerical approximation. In general, the more integration points used, the more accurate the numerical integration will be; although this is at the expense of additional computational effort. Motivated by the desire to maximise the accuracy, yet minimise the storage requirements, a new hierarchical, adaptive microplane scheme has been developed. The basic idea is that one starts the calculation adopting a low-order integration rule within the mildly nonlinear (pre-peak) range. Upon reaching the material state satisfying
some localisation/instability criterion, one could continue the calculation after switching to a higher order integration rule. Several switches could be made as different thresholds are met. This approach assumes that the material constants have been optimised for the highest order integration rule to be used in the analysis.

The main issue here is how to transfer, or map, the old state variables, based on the old integration rule, to the new state variables, based on the new integration rule. The two sets of state variables must clearly be consistent with one another to allow the analysis to proceed in a reliable manner. The solution technique proposed here is based on a multi-stage scheme (see Qiu and Crouch, 1997 for further details).

7.2.3. Localisation and Mesh Objectivity in a Softening Medium

It is now widely accepted that standard local constitutive models are inappropriate for materials which exhibit strong strain softening. When the material tangent stiffness matrix ceases to be positive definite, the governing partial differential equations may loose ellipticity, which renders the boundary problem ill-posed. From a numerical point of view, this situation manifests itself by spurious mesh sensitivity of finite element computations; strain localises into a narrow band whose width depends on the element size and tends to zero as the mesh is refined. The corresponding load-displacement diagram always gives a snapback for a sufficiently fine mesh, and the total energy dissipated by fracture converges to zero.

The simplest, but crudest, remedy (which is often used in engineering applications) is to adjust the post-peak slope of the stress-strain diagram as a function of the element size. When this is done properly, the energy dissipated in a band of cracking elements does not depend on the width of the band. More refined techniques which attempt to ensure objectivity are those methods known as localisation limiters. Such methods include the use of higher order gradient models (Aifantis, 1984; Schreyer and Chen, 1986; Vardoulakis and Aifantis, 1991; de Borst and Muhlhaus, 1992 and Pamin, 1994), Cosserat continuum models which include couple terms (Muhlhaus and Vardoulakis, 1987; de Borst, 1991 and Steinman and Willam, 1992), or viscoplastic regularisation (Neddleman, 1987). An alternative localisation limiter is provided by the concept of non-local averaging which may, in principal, be applied to any type of constitutive model (Bazant, 1984; Pijaudier-Cabot and Bazant, 1987; Bazant and Lin, 1988 and Bazant and Ozbolt, 1990). In the work undertaken at Sheffield University, the viscoplastic regularisation technique, based on a Duvaut-Lions model, has been adopted. Whilst this method is stable and appears most promising, it appears only in specialised research codes. Further details may be found in the Doctoral Thesis of Mesmar (1999). Note that although all these methods introduce a significant computational overhead, they typically lead to faster convergence within the non-linear iteration loop.
7.3. Modelling Technique for the Reinforcement

Reinforcement in concrete may either be modelled in a distributed sense by adding directional stiffness to the concrete element or by linking discrete beam or bar elements to the concrete element nodes. In the former approach, some codes (for example ANSYS) distribute the stiffness over the entire element, whereas others (DIANA) allow bars or membranes to be defined within the element. In either method, perfect bond between the steel and concrete is always implied.

The second method of modelling reinforcement, whereby bars are modelled by truss elements linked to the nodes of the parent concrete elements, is now as the discrete approach. This method is generally more time-consuming to set-up but it does enable bond-slip and dowel action effects to be incorporated into the analysis. Simple elasto-perfectly plastic (or strain hardening plasticity) models for the reinforcement may be used in both methods. A recent plane stress approach which treats reinforced concrete as a homogenous material has been shown to offer strong simulation capabilities for shear wall and deep beam analyses.

7.4. Fluid-Structure Interaction

Whilst the current state-of-the-art in CFD-FE modelling has not yet reached a level of maturity to include of a 3-dimensional fluid domain which is able to realistically capture the complex hydro-dynamics of wave breaking and slamming, it is relatively straightforward to introduce an inviscid compressible fluid to simulate the pressure transients in a coupled fluid-structure interaction analysis.
7.4.1. Governing Equations (Acoustics)

Full dynamic equilibrium for the coupled fluid-structure motion is given by the following expression

\[
\begin{bmatrix}
M & 0 \\
\rho Q^T & \tilde{M}
\end{bmatrix}
\begin{bmatrix}
\ddot{d}_s \\
\ddot{p}_f
\end{bmatrix}
+ 
\begin{bmatrix}
C & 0 \\
0 & \tilde{C}
\end{bmatrix}
\begin{bmatrix}
\dot{d}_s \\
\dot{p}_f
\end{bmatrix}
+ 
\begin{bmatrix}
K -Q \\
0 & \tilde{K}
\end{bmatrix}
\begin{bmatrix}
\ddot{d}_s \\
p_f
\end{bmatrix}
= 
\begin{bmatrix}
F_s \\
0
\end{bmatrix}
\]

All terms relating to the structure (identified by the subscript s) are identical to those given earlier for the shell analysis. The additional terms involving the sub-matrices [C] refer to the fluid-structure coupling, or fluid domain alone [\tilde{M}], [\tilde{C}] and [\tilde{K}]. Further details may be found in Zienkiewicz and Taylor.
This coupled system leads to a non-symmetric form which may be solved for any set of input forces, \{F_s\}, by the generalised SST step by step algorithm. In the SST algorithm, the displacement and pressure vectors are approximated by quadratic functions of time

\[
\{u_{n+1}\} = \{u_n\} + \{\dot{u}_n\}t + (t')^2 \{\alpha_n^u\} / 2
\]

\[
\{p_{n+1}\} = \{p_n\} + \{\dot{p}_n\}t + (t')^2 \{\alpha_n^p\} / 2
\]

Inserting these expressions into the governing equation of equilibrium yields a symmetric system expressed in terms of the intermediate variables \{\alpha\} which may be solved for using standard algorithms. Care should be taken to avoid model predictions where low (or even negative) fluid pressures appear.

Note that the maximum spacing of nodes in order for the FE mesh to replicate the waveform depends on the frequency of the stress wave. Ensuring at least 5 nodes are available over the wavelength will generally capture the wave transmission. For example, if the dominant frequency is of the order of 25Hz, then the nodal spacing should be no more than 15m in water. For higher frequencies, the nodes should be more closely spaced (Tyrrell, 1998).

Looking to the future, it is anticipated that further research and development in numerical modelling studies will lead to greater realism in coupled CFD-FE simulations.

7.5. Modelling the Dynamic Far-Field

Stiff structures sitting on relatively soft foundations exhibit a dynamic response which may differ significantly from those supported on effectively rigid foundations. In the first situation, part of the vibrational energy may be dissipated into the supporting medium by radiation of stress waves and by hysteretic (material damping) in the medium itself. These effects can lead to a strongly damped response which should be taken into account if an accurate simulation of the real behaviour is to be made.

For a typical caisson structure, the role played by the foundation in influencing the bending response of the front wall is negligible. However, realistic models of the soil are needed if seismic analyses are to be performed. Recent innovative work by Wolf and Song (1996) has lead to a new method of treating the far-field in time-domain dynamic soil-structure-interaction studies. The following partitioned equation of motion includes additional terms which account for the behaviour of the soil beyond a boundary (identified with the subscripts b). This scheme requires the convolution of \[M^o\] with previous velocities, however the procedure can readily be incorporated into a standard time-stepping scheme (such as Newmark, or HHT). Neglecting any explicit damping matrix (radiation damping is automatically satisfied and material damping can appear through a non-linear structural stiffness matrix) we have
The method leads to very significant computational savings in large scale analyses, avoiding the use of huge extended meshes or inaccurate transmitting boundaries. Work has recently been completed at the University of Sheffield to incorporate this scheme within yaFEC. Note that the method may be applied to model the far field in both the solid and the fluid phases.

### 7.6. FE Pre-Processor

A dedicated FORTRAN pre-processor has been written at the University of Sheffield specifically for rectangular multi-celled caisson structures. The user enters the following information which defines a particular structure:

(i) total length of caisson, (ii) total height of caisson (including base slab but excluding sea wall super-structure), (iii) total width of caisson (excluding heel and toe regions of base slab), (iv) height of sea wall, (v) height of berm foundation, (vi) front slope of berm, (vii) rear slope of berm, (viii) width of upper level of berm near toe, (ix) width of upper level of berm near heel, (x) number of elements (in vertical direction) in berm, (xi) relative increase in depth of elements in berm, (xii) number of elements in berm projecting in front of toe, (xiii) number of elements in berm projecting behind heel, (xiv) number of caisson cells across width, (xv) number of caisson cells over length, (xvi) number of elements in each caisson cell, over width, (xvii) number of elements in each caisson cell, over length, (xviii) number of elements over height (not including base slab or top slab), (xix) width of chamfer, (xx) number of elements in toe region of base slab, (xxi) number of elements in heel region of base slab, (xxii) number of elements over height of sea wall, (xxiii) thickness of front wall, (xxiv) thickness of rear wall, (xxv) thickness of base slab, (xxvi) thickness of top slab, (xxvii) thickness of side walls, (xxviii) thickness of internal cross walls and (xxix) chamfer size.
The programme generates the full nodal co-ordinate and element topology lists and includes the ability to apply standard boundary restraint conditions as well as equivalent nodal forces corresponding to pressure loading on the front face and underside of the base slab. Note that the berm and foundation may be modelled by this programme and the top slab may be removed from the caisson. Fill material in the cells may be included or removed. Use of this code speeds up analysis time significantly as well ordered regular meshes with over 20,000 elements may be generated in less than one second on a standard workstation. This enables comparisons between alternative designs to be made relatively quickly although moderately large analyses (say with over 10,000 20-noded brick elements and 1000 time-steps) still consume significant CPU time on even the fastest workstations.

7.7. Post-Processor (DANPLOT)

Post-processing of the Finite Element results is achieved by using the public-domain software DANPLOT which has been developed in the Civil Engineering Department at the University.
of Manchester. This flexible programme accepts nodal co-ordinates and element topology data, together with nodal and/or integration point data (such as resulting displacements and/or internal stresses and strains) and creates deformed mesh plots, contour plots and vector plots which may be exported as Postscript files. A range of element types is accepted in DANPLOT, including all those used by {yaFEc}. The mesh may be rotated, enlarged and viewed from any position. Addition of text and other features, such as backgrounds and related figures, is best performed by importing the resulting Postscript file into a vector based drawing package, such as Adobe Illustrator or Corel Draw. Animated graphics (such as deformed plots or eigen-modes) may be produced in DANPLOT by defining and replaying a sequence of images. Other propriety post-processing tools (such as Hypermesh and FEMGEN) may of course be used, provided transfer programmes are written to allow rapid interfacing between the FE code and the graphics routines.

REFERENCES


Tyrrell, R J, (1998), How to get started in Acoustic Analysis, NAFEMS.

CHAPTER 5: SOME OBSERVATIONS ON THE DURABILITY AND REPAIR OF CONCRETE STRUCTURES IN A MARINE ENVIRONMENT

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ABSTRACT

This report gives an overview of some of the durability issues which face the structural engineer when designing low maintenance reinforced concrete elements which are to operate in a very aggressive marine environment for up to 100 years. The key degradation mechanisms are discussed and a variety of protection/repair techniques presented. These aspects can control the design of marine structures. Some of the issues raised will help an engineer to arrive at an appropriate management strategy for maintaining the integrity of their structure throughout its lifetime.

1. INTRODUCTION

When considering durability issues it is important to note that it is not just the properties of the materials alone which define performance. The long-term performance of a particular section depends intimately on the location of the point under consideration and the significance of the member with respect to the overall structural behaviour (that is, the consequences of that member failing to satisfy a particular design limit state).

2. PROGRESSIVE DETERIORATION

It is vital to appreciate that reinforced concrete structures deteriorate progressively. In many cases this deterioration is negligible for a period of up to 50 years or longer. Damage levels (due to corrosion of reinforcement, for example) may be defined in terms of several phases associated with increasing exposure period (i) arrival at an initiation threshold (ii) preliminary corrosion (iii) appearance of first crack (iv) loss of steel section and (v) loss of structural
integrity. Premature deterioration caused by reinforcement corrosion has been attributed to inadequate specification, poor design detailing and poor construction practice such as insufficient compaction, inappropriate curing or inadequate cover. Existing design codes (such as BS8110) typically embrace a deemed to satisfy approach. Adopting a probabilistic approach provides a rational approach both for the design of new structures and the development of a viable maintenance strategy to repair existing structures. These notes review the basic deterioration mechanisms and describe in some detail alternative repair techniques which may be used for RC caisson structures.

3. COMMON DETERIORATION MECHANISMS

3.1. Chloride contamination

Chloride contamination can occur as a result of chloride ingress from the severe coastal environment where direct contact with salt water occurs, or of being cast into the concrete (e.g. as a result of calcium accelerators or contaminated aggregates or mix water).

In some respects, chloride ingress from the environment is similar to carbonation in as much as there is a period before the steel becomes activated. The principle difference is that the carbonation front is well defined, with the change from pH 13 to pH 9 occurring over a very few millimetres, while the chloride threshold level is less defined and the change in chloride concentration is more gradual. Furthermore, by the time a bar becomes active due to the ingress of chlorides, there will already be chlorides at concentrations below the activation level in advance of the corroding steel.

If corrosion is detected at a very early stage and immediate action is taken, the approach to repair may be relatively straightforward. One method is to reduce the level of chloride in the cover zone using the electrochemical chloride extraction technique. However, this is generally a costly and Alternatively, chloride contaminated concrete can be physically removed and replaced (where the steel is exposed, it must be thoroughly cleaned of rust and chlorides before reinstating the section with cementitious mortar to re-passivate the steel). In either case, additional protection using a surface treatment would prevent further ingress of chlorides.

If the decision to repair is delayed and chlorides penetrate beyond the reinforcement at a sufficiently high level, then the extent of cutting out and bar preparation will increase substantially.

Failure to remove all areas of chloride contaminated concrete, for example by only repairing cracked or spalled areas, can lead to corrosion of the steel in adjacent areas due to the incipient anode effect. Various patch repair systems have been claimed to reduce this effect. For instance, electrically isolating primers for the steel (e.g. epoxy resin) or the use of highly
resistive cementitious repair mortars may improve the situation slightly, but they are unlikely to completely eliminate incipient anodes since cathodes will exist in other non-repaired areas. Further, in the case of electrically isolating primers, if chlorides, even at very low levels, can penetrate beneath the coating at the ends of the repair, corrosion may occur very rapidly in the absence of the protection usually afforded by the alkaline environment in concrete. The use of sacrificial steel primers (e.g. zinc rich epoxy coatings) maintains a high negative potential in the repaired area due to their preferential corrosion, thus reducing the risk of incipient anodes and possibly also eliminating the risk of crevice corrosion. However, there is only limited experience with their use. In short, conventional repair of heavily chloride contaminated concrete is unlikely to be successful unless all chloride contaminated concrete is removed from around the bar.

It is clearly impossible to remove all contaminated concrete when the chloride has been cast-in. If the risk of further corrosion in the future is acceptable, breaking out followed by patch repair can be attempted. The principal difference when compared with normal patch repair is the need to prevent back diffusion of chlorides from the old concrete into the repaired areas. Thus particular attention must be paid to the barrier properties of the bond coat, patch material and reinforcement primer. Because all contaminated concrete cannot be removed, all repairs will be susceptible to incipient anode development and crevice corrosion, as indicated above. In many cases the most satisfactory long-term approach to arresting chloride induced corrosion is by application of a cathodic protection system (although this decision depends on the location of the element to repair). This removes the ability of chloride contaminated concrete to corrode under whatever environmental factors prevail. While cathodic protection is expensive to install, if properly designed it has low running costs and may significantly reduce the need for patch repairs in the short and long term.

**3.2. Concrete Carbonation**

Repair to carbonated concrete is relatively straightforward. Because corrosion will only occur when the bar lies in carbonated concrete, the remedial process involves identifying areas of the structure where the cover is lower than the depth of carbonation. This approach will identify both damaged areas and areas of concrete which are active but not yet cracked or spalling, thereby eliminating the risk of further spalls occurring immediately after the repairs are completed. Lasting repair will be effected by restoring the alkaline environment around the bar, using a cementitious patch repair mortar. As an alternative, re-alkalisation can be used to re-passivate the steel without the need to cut out and repair active, but as yet undamaged areas. With either method, application of an anti-carbonation surface coating is recommended to prevent further carbonation in un-repaired areas.

The above approach will give a long-term solution requiring only the maintenance of the anti-carbonation coating. Depending on the use of the structure, cheaper options may be preferred. These include:
(i) repair only visibly damaged areas, relying on the anti-carbonation coating to reduce the rate of corrosion in active areas, (ii) repair only visibly damaged areas and accept a continuing high maintenance expenditure or (iii) use holding repair to reinstate spalls, followed by full repairs (or demolition) at a later date.

4. REPAIR AND PROTECTIVE METHODS AS PART OF A MAINTENANCE STRATEGY

This part of the report provides some background information on the appropriateness of different concrete repair and protective techniques, systems and materials for reinforced concrete caisson structures operating in severe coastal environments. These notes apply to both the super-structure and the below-water structure although the different local conditions will generally result in different repair treatments. Therefore, before appropriate materials and techniques can be selected for the repair of a deteriorating structure, a number of structure specific factors will need to be considered to ensure that the repair successfully achieves its objectives. Before repair is contemplated, the cause(s) and extent of the deterioration must be determined, such that the cause(s) may be removed (if possible). This necessitates a thorough investigation of the structure. It is beyond the scope of this report to describe in detail the investigation procedure. However, it should be carried out by experienced personnel without bias to a particular repair procedure or material. It is of course important to identify the intended purpose and service life of the repair, that is, whether the repair is cosmetic or structural and whether extended life is necessary due to operational difficulties which may prevent regular access to the areas being repaired. The in-service environment of the repair material should be quantified and any restrictions on surface preparation techniques identified (for example, if it may be applied under water or not).

Factors which directly damage the concrete are water-based chlorides, carbonation, frost action, alkali aggregate reaction, sulphate attack, chemical/acid attack, mechanical (e.g. abrasion or impact) or fire damage. Structural overload can also damage reinforced concrete members. By far the most common cause of damage in a caisson breakwater is reinforcement corrosion, caused by chloride ions from the sea water penetrating into the substructure. It is stressed that all repair situations are unique and in all cases it is essential to seek expert advice before deciding on a repair method.

The first step in any repair strategy is a thorough assessment of the damaged structure or component, including evaluation of the following: (i) the cause of damage or deterioration, (ii) the extent of damage or deterioration and (iii) the effect of that damage has on the structural behaviour of the component or structure. The initial assessment is the basis on which all subsequent decisions on repair techniques and materials to be adopted are made. Consequently, the importance of undertaking the assessment in a well planned scientifically based manner and using suitably experienced personnel cannot be overemphasised.
Having determined the cause of the damage or deterioration on the basis of the condition survey or damage audit, the choice of repair strategy may be made. Maintenance policies applied to existing structures generally involve the decision to implement one of the following strategies:

(i) do nothing (except take measures to protect public safety) and accept reduction in structure life,
(ii) carry out holding repairs to slow down corrosion rate, accepting the need for further repairs at intervals in order to reach desired life of structure,
(iii) carry out a once and for all refurbishment to enable the structure to reach its desired life,
(iv) demolish and rebuild all or part of the structure.

Depending on the causes, extent and effect of the deterioration, and on the chosen repair strategy, a number of basic repair or protection techniques can be applied: (i) local patching/concrete replacement and crack filling, (ii) surface treatments, (iii) other techniques including cathodic protection, re-alkalisation and chloride extraction, local strengthening of members, and migratory corrosion inhibitors.

For each repair situation, there are usually several technically acceptable techniques, each with its own particular combination of costs for labour, materials, access, requirement for continued use of the structure during repair, and future maintenance. As a basic philosophy behind selecting the most appropriate repair options, each solution should be costed for the required future design life of the structure, so that both initial and long term maintenance costs are included. There are unfortunately no unique solutions to the lowest cost, and each particular project must be treated on its merits.

4.1. Patch repairs

A patch repair generally consists of a coating for the exposed reinforcement, a bonding coat applied to the exposed concrete substrate, the patching material, and sometimes a surface treatment applied to the entire member to prevent further deterioration. As well as whether it will perform its basic task of restoring the concrete profile and protecting the reinforcing steel, factors to be considered when selecting a patch repair system are its ease of application, compatibility with the structure and substrate, and its durability.

Assuming the appropriate product has been selected for the given conditions, it is very important that the manufacturers’ application instructions are then followed carefully. Workmanship is a critical factor, and it is often difficult to ascertain whether the material or applicator is at fault if the repair fails to perform satisfactorily. A manufacturer’s guarantee or warranty will therefore usually not be very helpful if a repair fails.
The basic choice of patching material is between a cementitious system (cement only or polymer modified) and a resin system. The choice of material should be based on what it is being asked to do and what the environment in which it is being asked to function is. In all cases the first step in selecting an appropriate material is to identify the cause of the deterioration and remedy it if possible. The client then needs to make a decision on what type of repair he requires, e.g. structural or cosmetic. Only then can appropriate materials be selected.

It is usually desirable that the mechanical properties of the repair material should resemble as closely as possible those of the structure being repaired. Thus as a general rule, cementitious systems should be selected. They can also provide fire resistance, whereas resins soften at relatively low temperatures and may also be combustible (although this is unlikely to be an issue for a marine structure). Cementitious systems are cheaper than those based on resins, though labour usually accounts for a large proportion of repair costs. Their performance can be further enhanced by the use of mineral/pozzolanic admixtures. Polymer modified cements can also offer greatly improved performance.

Although their mechanical compatibility with concrete cannot match that of cementitious mortars, there are some applications where resins or resin mortars are more suitable. In cases where cover thickness is restricted, resin mortars may provide less permeable cover than a cementitious mortar, though the permeability of the latter can be reduced by incorporating polymer or mineral/pozzolanic admixtures. Resins can also provide high resistance to chemical and physical attack. With suitable formulation they can also offer rapid development of high strength and the ability to cure or harden under environmental conditions outside the range of cementitious mortars. (Some compounds are not suitable for use in confined spaces, though, and good ventilation is always desirable). Sometimes feather edges cannot be avoided and, although polymer admixtures may make it possible to use cement mortar patches, resin mortars are often more suitable.

Suppliers of patch repair materials generally recommend their use as part of a patch repair system. The system may consist of a number of components including a bonding bridge to promote good adhesion between the repair mortar and the concrete substrate, an anti-corrosion primer for the steel reinforcement and a coating applied over the whole surface of the structure after repair to reduce carbonation or to improve chemical resistance. A fairing coat or levelling mortar is often required for the areas not needing repair before the coating can be applied. The following sections indicate the function of each part of the repair system.

4.1.1. Anti-corrosion reinforcement primer

In cases where steel reinforcement has corroded to some extent, it essential to remove loose or poorly adhering corrosion products and chloride contaminated corrosion products. Where the corrosion products are dense, firmly adherent and free from chlorides, they may be treated by a rust converter. The subject of chemical rust converters is very complex though, and there is
need for more research. Where significant loss of section has occurred, new steel will need to be added. Where such additional steel is required, bond and anchorage problems will arise, and a structural engineer should be consulted to ensure that the design requirements are satisfied. Freshly cleaned steel will begin to rust within a few hours if it is not protected. There should therefore be a minimum of delay, especially if the steel is to receive a treatment which will not provide a passivating alkaline environment.

4.1.2. Cementitious primers:

Hand-applied cementitious repair mortars are generally much drier than normal concrete at the time of application and it is therefore common practice to apply a cementitious slurry coating to the steel reinforcement to ensure intimate contact of the alkaline protective medium. Some early specifications called for a simple OPC and water slurry to be used. However, such slurries were found to have very poor application properties, they thickened rapidly (and so were likely to be repeatedly diluted) and soon dried to a dusty powder of unbonded partially hydrated cement. These difficulties were overcome by the incorporation of polymers, typically SBR or acrylic dispersions, which ensured a coherent well-bonded film of cementitious slurry.

4.1.3. Epoxy resin primers

Some systems use epoxy resin coatings, usually containing an anti-corrosive additive such as zinc chromate, iron oxide or even ground cement clinker. However, although the coatings provide excellent barrier protection of the steel, if corrosion is initiated beyond the coated area, rusting may progress along the unprotected steel underneath the epoxy coating (Mays 1992). Also, barrier coatings are only as good as the quality of workmanship in application. They may also require the addition of fine aggregate on the surface to provide bond with the repair mortar - an extra application and one easily omitted. Dripping of the coating onto the prepared substrate can ultimately reduce bond if it dries.

4.1.4. Zinc-rich epoxy primers

The rusting of steel is an electrolytic process; hence the electrolytic or galvanic approach to its prevention which is used with zinc-rich epoxy primers. Mays (1992) notes that a plain cement slurry provides a more highly alkaline environment than the mature concrete beyond the repair, making the repaired zone cathodic and so giving good protection within the repair at the expense of the adjacent length of bar, which may become a sacrificial anode. However, the zinc-rich coating produces a completely reversed electrolytic balance, keeping the repair zone anodic, so that the untreated areas beyond the repair are relatively cathodic with respect to the repair. The repair mortar keeps the zinc coating in a stable condition so that the repaired and adjacent areas are maintained in a healthy equilibrium. Coatings complying with BS 4652: 1971 Metallic zinc-rich priming paint give a dried film containing more than 95% of metallic zinc, so that there is insufficient resin binder to effect any sort of barrier to
the steel and electrical isolation does not occur - contact between the metallic particles ensures a conductive path between the repair mortar and the steel. It is essential that they be thoroughly stirred before application to ensure zinc dispersion through the binder.

Mays (1992) acknowledges that it has been suggested that zinc-rich coatings should not be used in conjunction with cementitious materials because of the susceptibility of zinc to alkaline attack. There is still a lack of information in this area. Zinc-rich coatings have generally been acknowledged as being beneficial, but their long term performance is unknown.

4.1.5. Bonding bridges

Most proprietary patch repair systems include some form of bonding bridge or primer to promote adhesion of the repair material to the concrete substrate. These coats were originally water-cement slurry. Generally they are now given real adhesive properties by modification with a natural rubber latex or synthetic polymer dispersion such as PVA (though as indicated in Section 4.1.2., homo-polymer PVA should not be used where it will experience wet service conditions), SBR or an acrylic latex. The viscosity of such mixtures can often be adjusted to give the rheological properties required depending on the location of the repair (i.e. horizontal, vertical or overhead). The repair mortar must be applied before the slurry has dried out. Since re-application of a polymer modified cement bond coat is not recommended, it follows that great care must be taken to apply this type of bond coat just in advance of the mortar or concrete application, but as the rate of drying of any water based material can be very variable, there is always a risk that premature drying may occur. If it does, the only recourse is to remove the bonding coat back to clean concrete and start again. Some systems are available with a bonding coat of aqueous polymer dispersion alone, which are not immediately water resistant on drying. Some can be shown to be reactivated up to 24 hours after drying by the application of a wet cementitious mortar and still develop a good bond. This may be acceptable on a horizontal repair, but in vertical and overhead situations the bond coat is required to afford some grab to the repair mortar to help hold it in position until it has hardened. This can be achieved by the application of a second coat of those systems that have been shown to have this reactivation capability. Alternatively, re-emulsifiable PVA modified cement may be suitable in areas where the concrete will remain dry in the future.

Prior to application of any aqueous bond coat, whether slurry or dispersion, it is important that the concrete substrate be thoroughly saturated (but with no standing water). Where this is not possible, these bonding coats cannot be used. The alternative is to use an epoxy resin bond coat. The fresh cementitious mortar or concrete should be applied while the resin is still in a tacky condition. Resin bond coats can also be formulated for situations where the substrate concrete is likely to remain permanently saturated, when a polymer-dispersion based coat would be unlikely to develop a good bond.
4.1.6. Mortars and concretes

The majority of small concrete repairs are carried out by the hand application of cementitious mortars or, where larger areas are involved, concretes (incorporating significant quantities of coarse aggregate). (There may be practical difficulties with this method for overhead application). These mortars and concretes are usually modified by the addition of a polymer emulsion or dispersion. It would be unwise to specify a particular polymer (e.g. PVA, SBR, acrylic, styrene-acrylic) at the exclusion of others since there are no distinctive differences in performance between the groups, whereas within each group there can be wide variations in performance depending on the proportions and types of monomers employed in their manufacture, the amount and type of dispersion agent, surfactant, stabilizer, anti-oxidant, etc. The only sensible choices which can be made are those based upon performance of the total formulation, not its composition.

Hand placement may not be appropriate in some situations, particularly where large areas of concrete need to be replaced, or where congestion of reinforcement might prevent effective repair. Alternative methods of application are by spraying or by using shuttering and a flowable mortar. As a general rule, as repairs increase in area and depth, different methods become the most practical solution. For small areas (up to 1 m$^2$) and moderate depths (say up to 50 mm), hand applied general purpose mortars (giving strengths typically up to 40 N/mm$^2$) are appropriate. For small areas (up to 1 m$^2$) and deeper than 50 mm, hand applied high build mortars may be appropriate, though strength is relatively low (say 20 N/mm$^2$). If high strength is required, spray or poured mortars are used, giving strengths of, say 60 N/mm$^2$. For large areas of any depth above 20 mm and any strength, sprayed or poured repairs should be used. The latter is more costly because of the amount of temporary formwork required. It is noted that in such cases, the repair system may be different. For example with super-fluid concretes, there may be less justification for anti-corrosive treatment of the steel reinforcement, since this type of concrete fully wets and envelops the steel. There is also no need for a bond coat because the new material is fully supported in place by the shutter for several days, during which time a strong cementitious bond is able to develop. Sprayed concrete is also less likely to require anti-corrosive treatment of the steel reinforcement and the use of a bonding bridge. The manufacturers instructions should specify the system requirements.

Although the development of cementitious materials has reached the stage that they are generally the first choice for a repair, resin mortars are still appropriate under certain circumstances, as discussed in Section 4.1. A resin repair mortar, carefully formulated and designed for maximum thermal compatibility with the substrate can provide the best protection, providing a layer which is highly impermeable to the ingress of external aggressive agents. As with other materials, to achieve good durable repairs, careful selection of the resin composition and grading of the fillers appropriate to the application and service conditions is essential. Resin repair mortars are generally hand applied.
4.1.7. Coatings

After a local patch repair, it may be necessary or desirable to protect the structure from carbonation or chemical or physical attack by using a coating (though coatings are seldom used to protect against physical attack in a repair situation). Structural concrete will have blow-holes which are almost imperceptible until the surface is coated with a thin pigmented film. Whilst heavy-build coatings, often fibre filled, can bridge smaller blow-holes, the greater their capacity in this respect, the greater will be their surface texture. With these coatings, the larger blow-holes will require filling first and with thin-film coatings, all blow-holes must be filled. This is usually done by the application of a cementitious fairing, smoothing or levelling coat.

These should be revised and modified dependant on the hazards of a particular contract. As indicated in Section 4.1.6, hand placement may not be appropriate in some situations, and spraying or the use of shuttering and a flowable mortar might be employed. In such cases, the repair system may be different, and the application method will reflect this. Further details of the procedures for these techniques are given in the texts, reports and standards specified at the beginning of this section. In the case of sprayed concrete the reader is also referred to The Concrete Society (1979 and 1980) and ACI Committee 506 (1990).

Figure 1

4.2. Crack filling

As with all repairs, to select the most appropriate crack injection system, it is first necessary to consider the requirements of the repair: is structural reinstatement necessary, is the crack
live, or is it necessary to achieve a seal? Where possible, the causes of cracking should be remedied before repair is undertaken.

In the repair of live cracks it is necessary to make provision for movement to continue after repair. This usually involves treating the crack as a movement joint rather than simply filling it. Where no further movement is expected, cracks can be filled using either a cement grout or by injection of a polymer. Polymers have a number of advantages, depending on the particular formulation, and are most commonly used. These advantages include controllable setting times and low viscosity (allowing resin to penetrate into much finer cracks). Once cured, the tensile strength and bond strength will also exceed the strength of the surrounding concrete, giving a structural repair.

In the repair of live cracks, it is usually necessary to make provision for movement to continue after repair. In the case of live cracks requiring structural repair, although high strength flexible epoxy resin systems are available, their extensibility is very limited (about 30%) and in moderately fine cracks they will behave in the same way as completely rigid resins. Even if there is no structural requirement, the use of an elastomeric system to simply fill a live crack is unlikely to succeed due to the large strains required. In such cases, treating the crack effectively as a movement joint should be considered. Methods for doing this are discussed in most standard texts on concrete repair (e.g. Allen et al 1993), and several proprietary systems are available for the purpose. Dead cracks, however, can usually be repaired to return the structure to a condition approaching its un-cracked state. Non-structural filling of cracks to prevent moisture, atmospheric gases and other potentially deleterious substances from finding a relatively easy pathway to the reinforcement is sometimes required. In this case, cracks wider than about 1 mm in the upper surfaces of slabs, etc., can often be sealed by brushing in dry cement followed, if necessary, by light spraying with water. For cracks wider than about 2 mm, a cement-polymer dispersion may be worked into the crack. However, in both cases, the depth of penetration will be variable, and limited if the crack narrows. Alternatively, a low viscosity liquid polymer can be used in this case. Reservoirs are formed on the surface around the cracks using modelling clay, and the resin is poured in and allowed to penetrate under gravity. Some of these materials will penetrate cracks down to 0.1 mm width. This system is unreliable, though, and is very rarely used. It is usually necessary to apply a positive or negative pressure to assist with flow into the crack. This is usually applied by an injection system, or, where a large number of cracks occur over an area, by use of vacuum injection.

4.2.1. Choice of materials

The material used for crack repair must be such as to penetrate easily into the crack and provide a durable bond to the crack surfaces. The interface of the material and the crack surfaces should be such as not to allow infiltration of water and to resist the physical and chemical attacks likely to be encountered.
4.2.2. Cement grouts

Although the injection of cement grout into a crack is often the first option an engineer will consider, it is unlikely to provide a full structural repair of narrow cracks. Investigations have shown that cement grout cannot generally penetrate, or is ineffective, where the crack width is less than 0.75 mm (Mays 1992), though some fine and microfine ground cements may now allow penetration of cracks with widths down to 0.2 mm.

Cement grouts can be used for the repair of wide cracks but, to avoid the penalties of high shrinkage, proprietary grouts based upon shrinkage compensated cement systems should be considered. Whilst many cementitious grouts contain expansive agents, a clear distinction should be made between those which expand in the fluid state and those which produce expansion after setting. The former develop a low pressure by the generation of small gas bubbles which aid penetration. However, it is only the latter, which expand in the hardened state by a limited and carefully controlled ettringite formation, that can truly be called shrinkage compensated. They can achieve a fully monolithic union with the fractured host concrete.

4.2.3. Polymers

When it is necessary to ensure, as far as possible, that the sealant penetrates to the full depth of the crack, injection of polymer grout under pressure is the most commonly used method. Epoxy resins, polyurethane resins, acrylic resins and unsaturated polyester resins are all used for crack injection.

The formulations of commercially available injection resins vary widely in their properties, and care must be taken in making a proper selection. Important properties of any injection resin are its resistance to moisture penetration and to alkaline attack from the cement. Where tensile strength is a requirement, the tensile strength of the resin should approach that of the concrete as closely as possible. Therefore, a stiff and highly adhesive resin is desirable. These properties are available in epoxy or unsaturated polyester resins. Some epoxies may suffer from brittleness, shrinkage, and also thermal mismatch. Epoxy resins bond well to both wet and dry concrete, but polyester resins may not bond to wet concrete. Acrylic resins can also give structural repairs, but do not bond to wet concrete. Polyurethane resins will not give a structural repair, but will bond well in the wet and are recommended, along with some acrylics, where moisture resistance is the main requirement.

When treating vertical and inclined surfaces or soffits, the injection methods may take advantage of thixotropic grout in order to prevent it flowing out again, though generally it is better to use a low viscosity resin and seal the crack to prevent leakage. Thixotropic formulations are most useful where cracks extend through the thickness of a member, but access can only be gained from one side. The material flows when pressure is applied but remains in position when pressure is removed.
It is generally not practical to inject resins into cracks much less than 0.1 mm wide at the face, though it is possible with special resin formulations. Provided the crack is of sufficient width to make resin entry feasible, the low viscosity resin systems are able to penetrate deep into the finer depths of the crack. However, this is only possible with an appropriately long reaction time. Fast reactive systems will only close the crack at its surface.

At the other extreme, unfilled resins are not usually recommended for injection into cracks greater than 10 mm in width because of the risk that they develop too much heat during cure and subsequently contract.

When concrete surfaces contain large numbers of cracks, such that it would not be economical or practical to attempt to seal them individually, vacuum injection may be appropriate. Vacuum can also be used to assist with conventional resin injection under certain circumstances. As with all types of injection process, vacuum injection must be carried out by experienced operators who can select appropriate materials and degrees of vacuum for any particular circumstances.

4.3. Surface Treatments

A wide range of surface treatments are applied to concrete to provide enhanced durability. Surface treatments may be used to prevent indirect deterioration due to chemical/physical processes (e.g. loss of integrity following carbonation or the ingress of chlorides and subsequent corrosion of the reinforcement). Alternatively, coatings may be used to protect the concrete itself from direct chemical attack (e.g. by acids, sulphates, etc) or direct physical attack (e.g. freeze-thaw damage, impact or abrasive wear). Coatings are also applied where enhanced durability is not necessarily the prime requirement (e.g. waterproofing, aesthetics). As well as being applied to sound concrete, surface treatments can be used to cover cracks or for binding a loose and friable concrete surface.

When selecting a surface treatment, as well as its ability to perform its primary function, other factors such as ease of application, compatibility with the substrate, crack bridging ability, durability and coatability should be considered.

It is very important with all surface treatments that the manufacturers’ application instructions, including ambient conditions and surface preparation, are followed carefully. The particular requirements will vary according to the individual product and this should be a consideration in the choice of a product. This is becoming increasingly pertinent comment as manufacturers improve the specifications of their products to gain competitive advantage, often at the expense of increasing sophistication and so increasing limitations on application conditions.
It is important to be aware that all surface treatments have a finite life and the initial decision to treat a concrete surface must therefore be made with the knowledge that further periodic maintenance or re-application will be required. This commitment must be a factor in the economic choice of whether or not to apply a treatment and of the particular material chosen.

4.4. Other restorative techniques

In addition to the so-called conventional repair and protection techniques of patch repairs and crack filling and surface treatments, there are a number of other possibilities. These include electrochemical techniques such as cathodic protection, chloride extraction and re-alkalisation, as well as migratory corrosion inhibitors. Another technique is the retrofitting of external reinforcement for local strengthening/stiffening of members.

The application of these techniques to reinforced concrete is relatively new, and the associated technology is developing more rapidly than that for conventional repair techniques. However, they are quickly gaining acceptance and, except for the migratory corrosion inhibitors, are well recognised as useful options for repair and maintenance of reinforced concrete. In the case of the migratory corrosion inhibitors, further development and experience is still required.

4.4.1. Cathodic Protection

Cathodic protection (CP) of steel in sea water has been practised since 1824 (Davey, 1824). During the past 60 years, CP has been used extensively and successfully to protect steel from corrosion in water and soils (e.g. pre-stressed concrete water pipelines and buried reinforced concrete water tanks). These applications allowed the use of conventional buried pipeline cathodic protection design principles and anode systems.

The earliest developments of CP of above ground reinforced concrete were in the protection of bridge decks in the late 1950s. In the period 1973-1980, 35 similar systems were installed on bridges. Recent developments in anode systems have extended the practical application to other structures and buildings.

Electrochemical corrosion
To understand CP, it is first necessary to understand a little about the process of corrosion in wet (aqueous) conditions. Corrosion, other than that arising from gaseous diffusion, is a process which requires an electrical potential difference to exist between different sites on the surface of the material and that these sites are connected by an electrolyte. If a piece of steel is immersed in water (Figure 2), it will corrode because of the in-homogeneity of the surface microstructure in contact with the water. The corrosion process involves dissolution of the steel at a positively charged part of the surface (the anode), whereby ferrous ions move in solution to a negatively charged part of the surface (the cathode). This ionic movement
creates an electron movement and hence a current flow to balance the charge. The same principle applies to steel in moist, porous concrete.

Figure 2. How corrosion occurs (after Mays, 1992).

It is possible to protect steel reinforcement in an ionically conducting environment by connecting it to a current source which will supply electrons to the metal so that it becomes the negative side of the circuit, as shown in Figure 3. The current can be gradually increased until all the small local cells are progressively reversed to the ultimate state of inhibition. This process is known as cathodic protection.

The external current for CP can be provided in two ways: either by connecting the reinforcement to a metal which is higher in the electrochemical series of metals than steel, e.g. magnesium, aluminium or zinc; or by applying a direct current to the reinforcement. The choice of method depends on the environmental conditions of the structure and the voltage necessary to overcome its resistivity.

CP can be applied to steel reinforcement in two types of concrete structure - those exposed to the atmosphere and those submerged or buried - and different systems will usually be appropriate. In the case of a submerged or buried structure, if the reinforcement is corroding, the concrete will be wet and probably contaminated e.g. with seawater. The resistivity is likely to be very much lower than with atmospherically exposed structures and the limited voltage outputs from dissimilar metal anodes can be economically exploited. It is only
necessary to arrange a connection between the chosen metal and the steelwork for current to flow. The anode will then slowly dissolve over a period of months or years. This is termed *sacrificial protection*.

![Diagram of cathodic protection](image)

Figure 3. CP Technique (after Mays, 1992): (a) A freely corroding reinforcement bar, (b) A cathodically forced surface.

The resistivity of atmospherically exposed structures, even if they are relatively heavily contaminated with salts, is such that a much higher voltage is required to spread the protection current around the whole reinforcement cage. The material through which the protective current is passed into the structure, and which becomes the anode, is made of an inert material which does not corrode with time. This current is supplied from a power unit. This is called impressed current cathodic protection.

It is further noted that when the structure is fully immersed in an electrolyte, it can be provided with anodes remote from the structure. If the structure is neither immersed nor buried, but is in the air, the anode has to be applied to the structure itself in order that the current can be passed through the electrolyte to the cathode. Further, in this case, because the concrete is a relatively poor conductor of electricity and because the anode is placed close to the steel cathode, the anode must be distributed over the surface of the concrete to ensure uniform protection. These notes deal only with impressed current cathodic protection.

As noted above, steel in concrete is generally protected from corrosion by a passivating layer which forms on the surface of the steel. There are two major situations, however, in which the corrosion of reinforcement can take place. These are where concrete has carbonated, and where concrete has been contaminated by chloride.
Repair of carbonation induced corrosion can be accomplished by conventional repair techniques by breaking out and patching where steel is corroding and applying an anti-carbonation coating if necessary. With chloride contaminated concrete, however, conventional patch repairs require all the contaminated concrete to be cut away in order to avoid the development of incipient anodes adjacent to repaired areas. When CP is applied, only the damaged concrete needs to be replaced. Thus CP is ideally suited for reinforced concrete structures suffering from chloride induced corrosion.

Cathodic protection is applicable to any structure or part of any structure where the reinforcement is in continuous contact with the concrete electrolyte and where the concrete itself is continuous, i.e. the concrete itself is not delaminated and the reinforcement is not in unfilled ducting (such as post-tensioned pre-stressing steel) (Concrete Society, 1989). However, there are certain secondary effects which might occur and which might produce damage to either the elements of the CP system or to parts of the structure. These are discussed below.

Short circuits and continuity - direct electrical contact between anode materials and reinforcing steel creates electrical short circuits which will adversely affect the performance of the protection, particularly in the vicinity of the shorted area. Such shorts can be caused by tie wires or rogue steel. Also, where there is a lack of continuity of the steel, some areas will not receive adequate protection and may experience an increase in corrosion activity due to stray currents. Both these situations can usually be easily remedied - by removal or electrical insulation of tie wires or rogue steel in the former situation, and by bonding or connection by cable in the latter.

Hydrogen - care must be taken that overprotection does not occur, resulting in hydrogen gasification at the concrete-steel interface and thereby weakening the mechanical bond (Mays, 1992). In addition, hydrogen could conceivably enter the steel structure under stress with resultant embrittlement and cracking to steel grades which are sensitive to the magnitude of hydrogen absorbed (in particular, pre-stressing steel). For this reason, the negative steel/concrete potential has to be carefully limited (Concrete Society, 1989).

High resistance due to patch repairs, surface treatments and coated steel - materials like epoxy mortars may need to be removed if CP is to be applied due to the risk that parts of the structure may be unprotected as the CP current cannot reach all areas. This is also a problem with surface coatings and with epoxy coated rebars.

Galvanised steel - although galvanising will conduct, the criteria for CP of steel in concrete do not apply to zinc. Since both the zinc and iron will be corroding, they will create very complex corrosion cells.
Alkali silica reactivity (ASR) - as hydroxyl ions are generated on cathodes, and ASR is caused by high alkalinity, it may be exacerbated by CP. This action might have a deleterious effect on the bond with the reinforcement, because it occurs principally at the steel/concrete interface. However, there are no known examples of this being the case.

Anode - the primary anodic reaction would be expected to produce products having a low pH and some chlorine could be produced. Since the anode is usually placed directly onto an alkaline material, it must be expected that some reaction may occur. Inevitably, this reaction will be near the anode/structure interface and could be expected to promote detachment of the anode or rupturing at the interface between any anode embedment material and the structure, or the anode.

There are many types of anode, and the selection of the correct one is important in determining the overall cost and durability of the system. Surface anodes used in CP systems can be grouped into seven main generic types. A full description of these, along with their operating characteristics, is given in Concrete Society (1989). A briefer description is given below.

**Conductive coatings** These are proprietary coatings containing a conductive filler such as powdered graphite. They are applied at a thickness of the order of 400 μm and utilize current feed wires in the form of titanium mesh or carbon fibre mesh strips placed at roughly 3 m centres. The coatings are dark in colour because of the graphite content, but they can be overcoated either for aesthetic reasons or to reduce temperature gain. As with most coatings designed for concrete, the life expectancy for conductive coatings is 10-15 years, though problems at the concrete/coating interface could cause bond failure and reduced life. Local damage caused by impact or abrasion can be easily repaired and is not likely to cause significant problems as the current provided by adjacent areas of coating should continue to provide protection.

**Titanium mesh** The anode is formed from titanium mesh with a strand thickness in the range 0.5 - 2 mm and apertures with aspect ratio of 1:2 with shorter dimensions in the range 30 - 100 mm. The strands are given an application of a proprietary coating which has an active electro-catalitic role in the CP circuit. Mesh anodes are overlaid with a layer of cementitious mortar which encapsulates the strand and passes current to the concrete. Mesh anodes have been designed for a life expectancy of greater than 25 years on the basis of the anticipated consumption rate of the active coating. However, production of acid at the anode by the CP process may curtail the life of the system. As with any system involving an overlay, bond with the concrete is of critical importance. The mesh anode can be repaired if damaged, though not easily.

**Conductive Polymer cable-Ferex™** a proprietary system made by Raychem, consisted of copper wire surrounded by a conductive polymer coating. The strand is typically 8 mm in diameter. It is laid in wave patterns on the concrete surface and held in position with clips. It
is then overlayed with concrete. The manufacturer claimed an anticipated operational life of 30 years at the recommended current densities. However, the anode was found to deteriorate. This system is no longer marketed, though a large number were installed world-wide.

**Sprayed zinc** A layer of metallic zinc is applied to the concrete surface at a thickness of about 200 μm. Application is by arc spray or oxygen/acetylene or oxygen/propane gun. Electrical connection is by titanium or stainless steel plates fixed to the concrete with an insulating layer of resin before spraying. The zinc coating may be overcoated with a conventional paint treatment for aesthetic and protective purposes. Zinc is consumed in the cathodic process and a 200 μm thickness will give a life expectancy of the order of 10 years, depending on current densities. Reduction in durability may occur in applications to wet structures where self-corrosion of the zinc may occur, though the situation can be improved by overcoating. Any localized areas of damage due to corrosion, abrasion or some other physical cause can be repaired by local patching.

**Conductive overlays** These are used mainly on bridge or car-park decks and are usually formed using primary anodes of high-silicon cast-iron overlaid with a conductive asphalt. The cast iron anodes may be installed in recesses in the concrete so that future resurfacing can be carried out without disrupting the system. The conductive asphalt is normally laid at around 40 mm thickness but may be covered by a conventional asphalt as a wearing surface. As with a normal surfacing, the conductive asphalt will have to be replaced within 10-15 years. The conductive asphalt is not a proprietary product, and would normally be manufactured to order by an asphalt supplier. As well as conductive asphalt, some cementitious conductive overlays are available.

**Conductive resins** A Federal Highways Authority research programme in the United States developed anodes made with conductive resins for use on bridge decks. The resins have been used in two ways. In the first method, slots were cut to a depth of 25-30 mm in the deck and a primary anode of titanium or niobium wire was placed into the slot before filling with the conductive resin. In the second method, the resin anode system was placed in linear mounds on the deck before covering with a cementitious overlay. In both cases the anodes formed a grid pattern with the titanium or niobium wires in one direction at approximately 3 metre centres and the strips of resin containing carbon fibres at approximately 300 mm centres.

**Button, ribbon and rod anodes** These are fairly recent developments, where the anode size is minimised by the use of strips or ribbons of coated titanium, or small discreet anodes of various sizes and shapes. Careful system design is crucial in preventing too high a current density at the anode surface. Anodes are buried in the original concrete and linked together.

**Sacrificial anodes** - As well as using zinc as an impressed current anode, it can be used as a sacrificial or galvanic anode. In this case there is no power supply. The zinc corrodes sacrificially, and the current generated protects the steel. Early tests in the USA used zinc sheets and ribbons embedded in concrete. There were problems with expansive corrosion of
the zinc, and poor protection in dry weather. Latest results show these systems performing better than expected, and new trials are underway in Florida in marine conditions using flame sprayed zinc as the anode (COMETT, 1995).

The advantages and disadvantages for the different anodes depend upon the nature of the installation. Conductive asphalt overlays provide, for example, a hard wearing surface, suitable for use on bridge decks or car park decks, but they are impractical for vertical or soffit surfaces. On the other hand, conductive paint systems and sprayed metal systems are relatively easy to apply to vertical and soffit surfaces but are not practical to be applied to wearing surfaces.

A sacrificial CP system is self-controlling and is independent of external power requirements. However, the impressed current system has a power supply. The power source in most CP systems is mains AC. It is necessary to provide a transformer/rectifier (T/R) to step down the voltage and to produce the DC supply that is required. It is often necessary to divide the structure into different control zones to accommodate varying reinforcement densities or other variables and so it is usual to use T/R’s which have multiple outputs which can be individually adjusted. The power required for each CP control zone is typically in the range 40-150 watts. The output is generally a few amps at a low voltage (range 5-48 volts). The output voltage has to be sufficient to overcome the resistive drops in the cabling, reinforcement and concrete as well as the electrochemical potential across the anode/concrete and steel/concrete interfaces, whilst still providing the current density required for protection. Other considerations when selecting a T/R are outlined below.

Overload protection is required both on AC and DC sides of transformer-rectifier units. T/R units must be mounted in a weather- and vandal-proof enclosure. They are often required to operate in extreme environmental conditions and care must be taken either to cool the unit or specify adequate temperature limits. T/R units are available which provide essentially only half wave rectification with limited filtering. This type of rectifier outputs a very noisy signal. Whilst it is not known whether the transients are detrimental, it is normally considered prudent to incorporate additional filtering (see Concrete Society, 1989 for details). Provision should be made to measure current and voltage from each output. A portable digital voltmeter/ammeter may be used in conjunction with permanently installed measurement sockets and perhaps shunts or permanently installed indicators or both. A means of breaking the DC output current must be provided to allow measurement of instantaneous-off potentials.

The number of connections to anodes and to reinforcement must have sufficient redundancy to allow for operation with one or more breaks. The anode connections should comply with the anode manufacturer’s recommendations and should be made accessible for inspection and repair. The cathode connection (to the reinforcement) will require the exposure and cleaning of the reinforcement. Connections may be made mechanically (by drilling and tapping), or by thermal gas or pin brazing. The copper/steel junction is a potential source of galvanic
corrosion and must be adequately protected by suitable coatings; particular care should be exercised with mechanical rather than metallurgical connections.

Cables must be selected for their mechanical strength as well as their electrical parameters. Because of the possibility of abrasive damage during any repair or over-lay application, double insulated cables are recommended. Similar remarks apply to any cable attachments to reference electrodes or test coupons and where extremely severe conditions are likely to apply, additional protection in the form of non-metallic ducts or conduits should be considered.

A sacrificial CP system is self controlling and is independent of external power requirements. However, the impressed current system has a power supply, and this must be controlled so that the amount of current flowing through the system maximizes life, avoids damage, and keeps costs down.

The latter requirement of minimising cost is generally trivial, as most CP systems require less than 200 Watts. However, chemical reactions at the anode produce acidity which attacks concrete and if excessive amounts of acid are produced, the anode will lose its bond with the concrete (and hence its electrical continuity). In addition, if the potential of the steel becomes more negative than -1170 mV versus a copper/copper sulphate electrode (CSE), hydrogen is evolved at the steel/concrete interface (Concrete Society, 1989). This can embrittle the steel (especially pre-stressing steel), and might also cause de-bonding of the reinforcing steel from the concrete. Thus operation of the impressed current protection system has to be monitored and controlled during its lifetime. Provision for this monitoring must be made at the time of installation. The minimum test equipment required for any CP application to reinforcing steel consists of reference electrodes and a high input impedance digital voltmeter. More sophisticated computer controlled systems are also available.

Reference electrodes are used to measure the potential of the reinforcing steel. They can be either permanently embedded in the concrete or can be portable/surface mounted for application to the external concrete face. Surface mounted reference electrodes are normally either copper/copper sulphate (CSE) or silver/silver chloride. Embedded reference electrodes are normally of the silver/silver chloride/potassium chloride double junction type, though other types have also been used (see Concrete Society, 1989). There is increasing commercial development of electrodes for CP as the market expands.

Reference electrode location should be at representative points on the structure. These are normally sites selected as being typical for the exposure, construction, orientation and other factors leading to the present condition of the structure. They may be located by simple repetition or random selection, or at corroding sites selected by a potential survey. The method of selecting representative locations is influenced by the particular structure, its accessibility and size. Normally at least three and as many as ten locations per system zone would be selected for permanent measurements (for inaccessible structures, more permanent
locations would be used). In these areas suitable reference electrodes would be embedded close to the steel to allow the potential of the reinforcing steel to be determined.

In smaller/easily accessible structures, in addition to the permanently mounted reference electrodes, potentials can be obtained using hand held reference electrodes over a representative grid.

In all cases, permanent reference electrodes should not be located at or near an area of recent repair as this will not be representative of the structure. If location in a repair is unavoidable, this fact should be noted so that allowance can be made when results are being assessed.

Digital voltmeters are required to measure the potential of the steel with respect to a reference electrode. Because of the inherent resistivity of the system, digital voltmeters with an input impedance of at least 20 Mohms, and preferably higher, should be used. They should have a range of ±5 V and a resolution of at least ±1 mV and an accuracy of ±1 digit. Multi-channel automated data logger systems are available which are well suited to monitoring large numbers of reference electrodes.

The criterion by which effectiveness of CP protection is judged has been subject to discussion and is not yet finally settled. Several possibilities have been suggested including:

(i) polarization of all reinforcement to a negative potential in excess of 770 mV with respect to a CSE,
(ii) potential decay of at least 100 mV,
(iii) potential shift of 300 mV and
(iv) E-log I determination.

Maintenance of the anode system, cabling and T/R may also be required. If deterioration of the anode system occurs, repair will have to be carried out when the efficiency of the system is likely to be reduced. This may not be difficult with paint systems, but others, such as mesh systems, can cause problems.

Cabling should have a service life of greater than 10 years, or longer if run in a conduit, though vandalism can be a problem. If damage is found on embedded cables, then the anode system may need to be removed to allow repairs. This highlights the need to ensure high integrity connections are made in areas with difficult access after completion.

The T/R unit and junction boxes will have easy access and so will be relatively easy to repair. The only problem which may arise is not knowing that the system has malfunctioned. If a modem system is not being used, it is wise to check that all system lights and power switches are in their designated mode at reasonably frequent intervals (say every one to two months).
4.4.2. Re-alkalisation and Chloride Extraction

Re-alkalisation and chloride extraction (CE) are techniques for rehabilitation of reinforced concrete structures suffering from chloride contamination or carbonation. They are non-destructive techniques, not requiring the removal of contaminated concrete, though loose and de-laminated areas must be repaired. Both are temporary treatments aimed at reinstalling corrosion protection without breaking away structurally sound old concrete and without requiring continuous current application, as is required with CP. Re-alkalisation and CE should not be applied to structures suffering corrosion due to very low cover or very low concrete quality. In those cases, the required control of current density and current distribution cannot be achieved. Moreover, early washing out of alkalinity or penetration of chloride will be probable. Frost or salt crystallisation damage may be caused by the large amounts of salts present (Hondel and Polder, 1992).

Re-alkalisation restores the concrete's alkalinity and is appropriate for carbonated concrete. CE draws chloride from the concrete and is applicable to chloride contaminated concrete (i.e. marine, de-icing salt, calcium chloride accelerator, contaminated aggregate, contaminated mix water). Both techniques can be applied simultaneously if both destructive processes are present.

The re-alkalisation process makes use of two physico-chemical processes, electro-osmosis (the movement of water molecules through a material under the influence of an electric field) and electrolysis (the decomposition of water generating hydroxyl ions under the influence of an electrical current), which take place simultaneously but at different rates.

In the re-alkalisation process a temporary mesh electrode contained within a disposable reservoir of alkaline solution of sodium carbonate is attached to the surface of the concrete (Figure 4). An electrical connection is made between the surface mesh and the reinforcement and a current applied, such that the reinforcement becomes cathodic and the mesh anodic. The migration of water by electro-osmosis toward the reinforcement results in a suction pressure being established in the network of capillary pores and the alkaline solution is drawn into the concrete. Electrolysis at the cathode results in the generation of hydroxyl ions necessary for re-passivation of the steel surface. When the cover zone is saturated (confirmed by testing) the current is disconnected and the external electrode and reservoir are removed. The concrete surface is then coated with a conventional anti-carbonation coating to prevent re-carbonation of the now alkaline pore solutions.

CE makes use of the same electro-chemical principals as re-alkalisation, except that the primary aim is to remove negatively charged chloride ions from within the concrete, by ion migration, to the surface anode (Figure 4).
Both re-alkalisation and CE are non-destructive in so far as removal of concrete is confined to areas which are loose or de-laminated. The concrete surface should be cleaned by grit or water blasting to remove dirt and any coatings; hydrophobic layers may hinder current flow and make treatment impossible. Any metallic fittings which may come into contact with the concrete should be insulated.

Reinforcing steel continuity must be checked to assure metallic conduction throughout the area to be treated since discontinuous steel may undergo severe stray current corrosion.

Short circuits between the anode mesh and reinforcement should be avoided. This may entail patch repairs to spalled concrete with exposed reinforcement, and sealing cracks which extend to the reinforcement. Cement based repair materials are preferred in order to provide sufficient electrolytic conduction. Prior to application, it is also necessary to determine the reference chloride profiles and/or depths of carbonation. This can be achieved by dust sampling, coring and appropriate testing. Electrode potential surveys should also be performed to determine corrosion risk before and after application.

Following preparation and electrical connection with the reinforcement, wooden battens are attached to the concrete surface using plastic plugs. The surface is then sprayed with a layer of cellulose fibre containing an electrolyte solution to a layer thickness of 20-30 mm (though many different materials have been used as electrolyte carriers, from paper to sponges). A steel mesh or a suitable inert mesh (e.g. titanium) is fastened to the wooden battens to act as the anode. Steel mesh has the advantage of being cheap and easily installed. It also eliminates the generation of chlorine gas formed during electrolysis, which can be a problem with a titanium mesh anode. The disadvantage is that the steel mesh may undergo severe corrosion due to the anodic process, causing high resistance layers or staining of the concrete. In some cases, the corrosion can also lead to the anode needing renewal during extended treatments. Titanium mesh has the advantage of being practically inert, thus not requiring
replacement or causing rust stains. The chief disadvantages are high cost, and the formation of chlorine gas, which can represent a health hazard in confined spaces, and can cause corrosion to fittings and electrical equipment, unless suitable precautions are taken.

The whole arrangement is finally over-sprayed with the cellulose/electrolyte to an overall thickness of 50-100 mm. Conventional fibre insulation blowing equipment is used to apply the fibre via a special mixing nozzle. A high pressure water pump provides electrolyte to a set of spray jets built into the nozzle and surrounding the dry fibre outlet. Mixing of fibres and electrolyte thus occurs between the nozzle and the surface, and on the surface itself. The alkaline reservoir can also be provided (especially for soffits) by means of a felt cloth, with a thickness of about 3 mm, instead of the fibre. There is no need for wooden battens, but the system will require continuous wetting. A ponded system, typically using plastic coffer tanks (or a simple damming system for horizontal decks) may also be used.

The electrolyte used in re-alkalisation is 1 molar sodium carbonate solution, unless deleterious reactions with concrete components are expected. Saturated calcium hydroxide solution is used as the electrolyte for CE and for re-alkalisation where alkali aggregate reaction is suspected. In order to prevent liberation of chlorine gas, the addition of manganese (II) ions has been used (Hondel and Polder, 1992). Bennett et al (1993) identify a number of other successful methods for avoiding chlorine gas evolution.

Re-alkalisation and desalination were developed and patented by Norwegian Concrete Technologies (NCT) in the 1980s. Fosroc Expandite Limited has recently acquired NCT and now markets both processes under the Norcure trademark. Because the techniques are patented, they can only be used under licence. Fosroc currently licence a number of contractors to perform re-alkalisation and desalination within the UK. Specifications for the properties of the materials used in the processes can be obtained from Fosroc, but the materials themselves are generally available from a wide range of sources i.e. specific proprietary products need not be used.

DC power is supplied by a transformer-rectifier. The design current density is typically 1 to 3 A/m² steel surface area. An upper limit has to be specified in order to avoid deterioration of the concrete. It has been suggested that a reasonable upper limit on current density seems to be 5 A/m², regarding both steel and concrete area (Hondel and Polder, 1992), though for practical purposes a value of 2 A/m² is more commonly used. Current distribution may be very uneven in cases of multi-layer reinforcement; its effect should be considered and laboratory testing of representative specimens under realistic conditions is recommended. Experiments suggest that most of the current passes from the anode to the first layer of reinforcement, with only negligible amounts of current going to the reinforcement lying deeper into the concrete, or further from the anode in a lateral direction.

Once the system is running, the current should be measured each day to control the current density; this indirectly checks the humidity of the fibre. The fibre should be sprayed with
electrolyte whenever it gets too dry. The duration of the treatment is an important parameter. More correctly, the total charge passed is the decisive quantity. As the current density is typically 1 to 3 A/m², re-alkalisation generally requires 1 to 3 weeks. Chloride removal, though, usually requires 1 to 4 months. However, the time period required to reduce the chloride concentration at the reinforcement to acceptable levels is dependent on a number of factors (Shields et al., 1993, Miller, 1994) including:

(i) type of contamination (it is easier to remove chlorides which have penetrated into the concrete from the environment after construction than cast-in chlorides),
(ii) quantity of chloride to be removed (high chloride concentrations need more time than low),
(iii) water/cement ratio of the concrete (CE is easier with high water/cement ratios),
(iv) temperature (high temperatures are better than low),
(v) cement content (CE is easier with low cement contents),
(vi) quantity of reinforcement (concrete with no steel obviously is not suitable for this process),
(vii) depth of reinforcement (cover thicknesses greater than the chloride depth is better than the opposite).

For re-alkalisation the depth of penetration should be checked after 3 days (cores and phenolphthalein). For CE, two to three weeks should pass before checking the chloride content. If results indicate the process is complete, treatment can be stopped.

Dismantling the system only entails disconnecting leads, wetting the fibre to remove it and careful removal of the electrode net. The concrete surface should be cleaned thoroughly by water jetting or high pressure hosing. Any cavities should be filled with approved material. An anti-carbonation or chloride barrier protective coating system can then be applied to the concrete surface.

A large number of possible side effects of the re-alkalisation and CE techniques have been investigated, especially in relation to the relatively high current densities required for CE. Investigations have been carried out into changes in concrete chemistry, moisture, porosity, frost resistance, permeability, chloride ion diffusivity, and concrete-to-steel bond strength. However, Miller (1994) states that no evidence of significant detrimental effects has been found.

As part of the SHRP program, the effect of the CE process on structural concrete and reinforcing steel was studied extensively in Bennett et al. (1993). They investigated hydrogen embrittlement, bond strength, anode types, factors affecting current distribution, electrolyte types (including the influence on AAR), and long-term effectiveness of the process (including the use of coatings and corrosion inhibitors). They report no damaging effects except at very high current densities i.e. 20 A/m² and over. Current densities of the order of 5 A/m² did not appear to have serious side effects and, further, have been applied on a large scale without any
evidence of damage. It can be assumed that 5 A/m² concrete and/or steel surface area can be taken as a safe upper limit in general (Hondel and Polder, 1992).

However, studies with concretes where AAR is a potential problem have given conflicting results. In the absence of further information, AAR should be considered in the case of treatment using sodium carbonate as an electrolyte. Research is going on into the use of, for example, lithium salts to limit the production of expansive gels (Hondel and Polder, 1992). Also, some efflorescence has been observed in some studies (Hondel and Polder, 1992). This may have a deleterious effect on subsequent coatings.

The durability of the corrosion protection achieved by re-alkalisation and CE has not yet been proven by long term, well documented testing in the field. Theoretically and according to short term testing, well executed re-alkalisation provides sufficient alkalinity to reinstate corrosion protection, unless significant chloride is present. Leaching out of the alkaline material should be avoided. A simple coating may be applied as additional protection (Hondel and Polder, 1992).

It has been shown, both in the laboratory and in the field, that significant amounts of chloride can be extracted from concrete. Where corrosion has not yet started to cause damage, this is valuable as a method to reinstall the structure’s service life expectancy. In cases where corrosion has started to cause damage, the question is whether corrosion will be stopped or slowed down sufficiently. Generally, stopping the corrosion will be difficult to achieve where high amounts of chloride have been added to the concrete mix. This is mainly because chloride remaining between the rebars or liberated from bonding to cement components, will diffuse to the steel and may start corrosion again after treatment is stopped. A significant reduction of the corrosion rate may very well be possible, though. In cases of corrosion damage due to chloride penetration, several (pilot) projects have shown that corrosion activity was significantly reduced. The prevention of new chloride ingress and consequent corrosion initiation may be decisive for the durability of the structure (Hondel and Polder, 1992).

4.4.3. Migratory Corrosion Inhibitors

Corrosion inhibitors are widely used in the protection of metals. There has long been interest in their potential use for protection of concrete reinforcement. They have generally been in the form of admixtures. ACI Committee 212 (1989) indicates that numerous chemicals have been evaluated for this purpose, including chromates, phosphates, hypophosphorites, alkalis, nitrites and fluorides. Only one chemical, calcium nitrite, has been used commercially on any scale as an admixture inhibitor for reinforcing steel. However, these admixtures are not generally suitable for application to existing structures except by drilling or braking out to the reinforcement followed by filling or patching with mortars containing the inhibitor. Some trials of impregnating concrete with calcium nitrite after drying it by heating it to above the boiling point of water have shown promising results. Electro-injection techniques have also been tried using phosphonium based corrosion inhibitors, but with mixed success (Bennett,
1993). A more recent development, however, has been in the availability of *migratory corrosion inhibitor*. These are inhibitor systems that can be applied to the surface of reinforced concrete or injected into the body of the concrete and then migrate to and protect the steel by virtue of their very low vapour pressure.

It is apparent from work carried out as part of SHRP, that migratory corrosion inhibitor systems are available that can potentially significantly reduce the rate of chloride induced corrosion in structures. Maeder (1994) also found some success with individual products. With increasing availability of such materials it is hoped that further independent research and trials will be initiated to prove their performance and effectiveness in comparison with other remedial techniques for corrosion control. There is particular interest in their use with pre-stressed and AAR-susceptible structures where the use of CP may be undesirable (Lambert, 1994).

5. USE OF FINITE ELEMENT ANALYSIS IN DURABILITY ASSESSMENT STUDIES

Over the past three decades, finite element analysis has not just been used by the research community but has become the accepted design tool in structural mechanics. Yet, despite the ability of this method to handle many diverse physical phenomena (in addition to mechanical loading, as described in the previous section) the application of FE to durability problems in complex reinforced concrete structures has only just begun. Given the success of the technique in other branches of engineering and physics, it appears inevitable that the next decade will see, for example, further use of FEA to simulate the time-dependent processes of chloride ingress, carbonation and corrosion of reinforcement. Once the governing mathematical models coupling thermo-mechanical and electro-chemical systems have been properly identified, a common solution approach is possible since all these problems can be expressed by similar integral equations and solved by finite elements. Such analyses will enable predictions of rates of corrosion to be linked to loss of mechanical performance. Other studies, for example, linking moisture removal to restrained shrinkage in repair systems will allow predictions of the likelihood of further cracking to be performed. The progressive growth of fractures in the concrete cover zone (due to cyclic mechanical load or freeze-thaw effects, for example) influences the rate of chloride transport, leading to a faster loss of bond at the steel-concrete interface, more rapid corrosion of the reinforcement and hence a degradation of the sectional moment or force resistance. Since structural integrity is the main issue, then reliable FE modelling of these durability related phenomena is meaningful only if the load carrying capacity of reinforced concrete structures (ignoring the diffusion based effects) can be satisfactorily simulated.
REFERENCES/BIBLIOGRAPHY


Taywood Engineering Research Labs, (1988), Offshore Technology Report OTH 87 247, *Effectiveness of Concrete to Protect Steel Reinforcement from Corrosion in Marine Structures*, HMSO.

