GUIDANCE FOR
CONCRETE STRUCTURES

BACKGROUND REPORT

Author
Offshore Certification Bureau
61 Southwark Street
London
SE1 1SA

Reviewed and prepared for publication by

Sir William Halcrow and Partners Ltd
Vineyard House
44 Brook Green
Hammersmith
London
W6 7BY

Date of Issue: June 1997
This report is made available by the Health and Safety Executive as part of a series of reports of work which has been supported by funds provided by the Executive. Neither the Executive, nor the contractors concerned assume any liability for the reports nor do they necessarily reflect the views or policy of the Executive.

Reports in the OTO series can be obtained from HSE Information Services, Information Centre, Broad Lane, Sheffield S3 7HQ
Tel: 0541 545500.
FOREWORD

This report is the Background Document to Section A23 of HSE’s Guidance Notes \(^{\text{(1)}}\), which was originally referred to as OTH (89) 304. For a number of reasons its publication has been delayed. Since its preparation there have been significant changes to UK legislation for offshore installations. In particular as a result of the issue of the Design and Construction Regulations \(^{\text{(2)}}\) in June 1996 certification of offshore installations will cease by the end of June 1998, and be replaced with a verification scheme described in reference (2).

As a result of the changes to legislation the Guidance Notes \(^{\text{(1)}}\) are now under review. It is recognised, however, that they contain useful material for the design and construction of concrete offshore installations, in particular section 23.

It should be noted that references to ‘certification’, ‘certificate of fitness’ and ‘Certifying Authority’ in this document only apply to the transition period up to June 1996.

May 1997


# CONTENTS

## SUMMARY

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 General</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Method and Programme</td>
<td>2</td>
</tr>
<tr>
<td>1.3 Database</td>
<td>3</td>
</tr>
</tbody>
</table>

## 2. EXISTING GUIDANCE NOTES (3RD EDITION)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 Existing Guidance Notes and other Codes</td>
<td>5</td>
</tr>
<tr>
<td>2.2 Design Parameters for Present Structures</td>
<td>28</td>
</tr>
<tr>
<td>2.2.1 Structure</td>
<td>31</td>
</tr>
<tr>
<td>2.2.2 Design</td>
<td>31</td>
</tr>
<tr>
<td>2.2.3 Materials</td>
<td>32</td>
</tr>
<tr>
<td>2.2.4 Service Performance</td>
<td>36</td>
</tr>
<tr>
<td>2.2.5 Maintenance</td>
<td>37</td>
</tr>
<tr>
<td>2.3 Revisions Required to Guidance Notes</td>
<td>37</td>
</tr>
</tbody>
</table>

## 3. TECHNICAL CONSIDERATIONS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1 Design</td>
<td>39</td>
</tr>
<tr>
<td>3.1.1 Limit State and Load Factor Design</td>
<td>39</td>
</tr>
<tr>
<td>3.1.2 Cracking/Cover/Durability</td>
<td>41</td>
</tr>
<tr>
<td>3.1.3 Impact</td>
<td>53</td>
</tr>
<tr>
<td>3.1.4 Implosion</td>
<td>66</td>
</tr>
<tr>
<td>3.1.5 Temperature Effects</td>
<td>70</td>
</tr>
<tr>
<td>3.1.6 Fatigue</td>
<td>75</td>
</tr>
<tr>
<td>3.1.7 Shear</td>
<td>86</td>
</tr>
<tr>
<td>3.2 Materials</td>
<td>87</td>
</tr>
<tr>
<td>3.3 Construction</td>
<td>96</td>
</tr>
<tr>
<td>3.3.1 Construction and Planning</td>
<td>96</td>
</tr>
<tr>
<td>3.3.2 Inspection and Testing during Construction</td>
<td>101</td>
</tr>
</tbody>
</table>

## 4. BACKGROUND TO DRAFT GUIDANCE NOTES

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 Outline Concept and Form</td>
<td>104</td>
</tr>
<tr>
<td>4.2 Commentary on Clauses</td>
<td>104</td>
</tr>
<tr>
<td>4.2.1 Section 12: Corrosion Protection</td>
<td>104</td>
</tr>
<tr>
<td>4.2.2 Section 23: Concrete</td>
<td>105</td>
</tr>
<tr>
<td>4.2.3 Section 30: Floating Installations</td>
<td>141</td>
</tr>
</tbody>
</table>

## APPENDICES

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appendix 1</td>
<td>Database for Project</td>
</tr>
</tbody>
</table>
LIST OF TABLES

Table 1 : Keyword list for database management ......................................................... 4
Table 2 : Comparison of rules/codes : design - cracking ................................................ 8
Table 3 : Comparison of rules/codes : design - cover ..................................................... 10
Table 4 : Comparison of rules/codes : design - load factors .......................................... 12
Table 5 : Comparison of rules/codes : analysis - fatigue .............................................. 14
Table 6 : Comparison of rules/codes : analysis - impact .............................................. 16
Table 7 : Comparison of rules/codes : analysis - implosion .......................................... 18
Table 8 : Comparison of rules/codes : analysis - temperature effects ......................... 20
Table 9 : Comparison of rules/codes : analysis - shear ............................................... 22
Table 10 : Comparison of rules/codes : specifications - construction ......................... 24
Table 11 : Comparison of rules/codes : specifications - materials ............................... 26
Table 12 : Concrete platforms in European waters ....................................................... 30
Table 13 : Comparison of material data of existing structures .................................... 33
Table 14 : Key references for cover, cracking and durability ........................................ 42
Table 15 : Key references for impact due to dropped objects .................................... 54
Table 16 : Key references for ship impact .................................................................... 61
Table 17 : Summary of ship collision characteristics based on energy methods (after Caldwell et al 1981) .................................................. 63
Table 18 : Summary of ship collision characteristics based on impulse momentum method (after Davies et al 1981) .................................................. 64
Table 19 : Collision forces from 2500 tonne supply vessel based on energy methods (after Furnes et al 1979) .................................................. 65
Table 20 : Key references for implosion ...................................................................... 67
Table 21 : Comparison of desk and computer calculation methods for implosion (after Lloyds 1987) .................................................. 68
Table 22 : Key references for temperature effects ....................................................... 72
Table 23 : Comparison of experimental results for temperature effects (after Clarke et al 1987) .................................................. 73
Table 24 : Key references for fatigue ........................................................................... 76
Table 25 : Key references for materials ..................................................................... 88
Table 26 : Effective diffusivity of chloride ions at 25º C in various cement pastes of W/C 0.5 (after Holden et al 1983) .................................................. 93
Table 27 : Key references for construction topics ....................................................... 97
Table 28 : Key references for inspection and testing during construction .................... 103
Table 29 : Comparison of BS6235 and Guidance load requirements ....................... 107
Table 30 : Comparison of ultimate limit state loading factors .................................... 108
Table 31 : Comparison of serviceability limit state loading factors ............................ 109
Table 32 : Comparison of cover requirements .......................................................... 119
Table 33 : Specified cover for existing installations .................................................... 121
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 1</td>
<td>Location of concrete platforms in the North Sea</td>
<td>29</td>
</tr>
<tr>
<td>Figure 2</td>
<td>Idealised corrosion reaction (after Beeby 1978)</td>
<td>46</td>
</tr>
<tr>
<td>Figure 3</td>
<td>Electrochemical system of corrosion cell</td>
<td>49</td>
</tr>
<tr>
<td>Figure 4</td>
<td>Assessment of possible damage by a Christianson 280mm OD drill collar (after Wimpey 1987)</td>
<td>56</td>
</tr>
<tr>
<td>Figure 5</td>
<td>Assessment of possible damage by a 915mm OD 22m WT drain caisson (after Wimpey 1987)</td>
<td>57</td>
</tr>
<tr>
<td>Figure 6</td>
<td>Velocity of slender objects falling from 35m (after Wimpey 1987)</td>
<td>59</td>
</tr>
<tr>
<td>Figure 7</td>
<td>Velocities of falling bulky objects (after Wimpey 1987)</td>
<td>60</td>
</tr>
<tr>
<td>Figure 8</td>
<td>Fatigue analysis procedure (after Price et al 1987)</td>
<td>78</td>
</tr>
<tr>
<td>Figure 9</td>
<td>S-N curves for reinforcement (after Paterson et al 1987)</td>
<td>81</td>
</tr>
<tr>
<td>Figure 10</td>
<td>S-N curves for concrete (after Price et al 1987)</td>
<td>83</td>
</tr>
<tr>
<td>Figure 11</td>
<td>Relative compressive strength of fog-cured and ocean-cured concrete (after Haynes et al)</td>
<td>95</td>
</tr>
<tr>
<td>Figure 12</td>
<td>Typical concrete platforms installed in the North Sea</td>
<td>100</td>
</tr>
<tr>
<td>Figure 13</td>
<td>Design S-N curve for reinforcement</td>
<td>111</td>
</tr>
<tr>
<td>Figure 14</td>
<td>Typical design S-N curves for submerged concrete (for $f_{cu} = 50$N/sq.mm)</td>
<td>113</td>
</tr>
<tr>
<td>Figure 15</td>
<td>Definition of exposure zones for typical structures</td>
<td>115</td>
</tr>
<tr>
<td>Figure 16</td>
<td>Times for corrosion activation (after Browne 1986)</td>
<td>120</td>
</tr>
<tr>
<td>Figure 17</td>
<td>Forms of impact damage (after Brown and Perry 1987)</td>
<td>123</td>
</tr>
<tr>
<td>Figure 18</td>
<td>Slenderness number $B$ for a complete Cylinder (after Chrapowicki et al 1987)</td>
<td>127</td>
</tr>
<tr>
<td>Figure 19</td>
<td>Partial cylinder slenderness number design envelopes (after Chrapowicki et al 1987)</td>
<td>129</td>
</tr>
<tr>
<td>Figure 20</td>
<td>Implosion design curves for imperfections $&lt;R/200$ (from Chrapowicki et al 1987)</td>
<td>130</td>
</tr>
</tbody>
</table>
SUMMARY

The Guidance Notes are published by the Department of Energy. They indicate the procedures to be followed and technical standards to be achieved whereby offshore installations, in the UK sector of the North Sea, are certified as fit for their purpose. Because of evolving technology, guidance is continuously being revised and updated; in 1984 the Offshore Certification Bureau were commissioned to prepare a review of guidance for concrete structures. This report presents the results of that review as:

- Section 1 - Report on the activities undertaken in revising the Guidance Notes.
- Section 2 - Review of existing codes, guidance and design practice.
- Section 3 - Papers on the major technical topics which form the background to the revised Notes.
- Section 4 - Clause-by-clause commentary on the Guidance Notes to provide a users handbook with amplification of the Clauses.

Available research data, particularly from the Concrete-in-the-Oceans programme, was reviewed to provide a basis for guidance. In addition design and performance data was collected on existing structures in the UK sector of the North Sea.

Design technical reviews conclude that substantial guidance revision is required particularly to incorporate fatigue and impact damage. Load factor design is incorporated based upon a semi-probabilistic design process (level 1) with factors determined by experience, although alternative factors could be derived based on reliability indices. Empirical requirements for impact damage from dropped objects or ship collision are based upon extreme events. Fatigue guidance, in line with recent research, requires all structures subjected to cyclic loading to be assessed. Cover and cracking requirements are relaxed, although checks for dynamic cracks are instigated.

Materials and construction aspects have been reviewed and up-dated.
1. INTRODUCTION

1.1 GENERAL

The Department of Energy's "Guidance on the Design and Construction of Offshore Installations" was first published in 1974. A series of amendments has subsequently been issued but those sections concerned with concrete have remained largely unchanged.

The Concrete-in-the-Oceans programme, to which the Department has been a major contributor, was established to investigate some of the problems relating to the design and construction of concrete offshore structures. The research projects included in this and other similar concurrent programmes have increased knowledge in the field of concrete offshore structures considerably beyond that which existed at the time the Guidance Notes were originally published. Phase I of the Concrete-in-the-Oceans programme was commenced in 1976 with Phase II in 1980; the overall programme was completed in 1986.

As a consequence, the Department of Energy considered that a review of those sections of the Guidance Notes relating to concrete offshore structures should be undertaken. The Offshore Certification Bureau, in January 1984, was commissioned to undertake such a revision. This report presents the results of that review in the form of:

- report on the activities undertaken in revising the Guidance Notes.
- a review of the various major topics which form the technical background to the Notes.
- a clause-by-clause commentary on the Guidance Notes providing, in essence, a users handbook with amplification of the Clauses.

To oversee and advise on the preparation of the draft Guidance Notes a Steering Group was formed, by the Department of Energy, from representatives of the industry and profession. The following formed the Steering Group:

Dr. J.V. Sharp (Chairman) MaTSU
Mr. R. Boom Lloyd's Register of Shipping
Mr. V.J. Bromley Chevron Petroleum Ltd
Mr. A.C. Burdall Offshore Certification Bureau
Dr. J. Clarke Cement and Concrete Association
Mr. P.G. Coates Det Norske Veritas
Mr. M. Collard Sir Robert Mc Alpine & Sons Ltd
Mr. L.G. Ellis Department of Energy
Mr. P. Fidjestol Det Norske Veritas
Mr. N. Foster Department of Energy
Mr. P.S. Godfrey Offshore Certification Bureau
Mr. N. Gunn Shell UK Exploration & Production Ltd
Mr. M.B. Leeming Ove Arup and Partners
Dr. J.F.A. Moore Building Research Establishment
Mr. N.J.M. Wilkins Harwell Corrosion Services.
Towards the end of the project period, Mr. N. Foster and Mr. N. Gunn retired from the group and were replaced by Mr. V.S. Davey and Mr. W. Visser respectively.

The valuable contribution made by members of the Steering Group, who gave their time freely, is gratefully acknowledged. In particular, the assistance of the Project Officer on behalf of the Department of Energy, Dr. J.V. Sharp, is appreciated.

The Steering Group met for a total of six full meetings between March 1984 and June 1986. In addition there were various sub-group meetings and informal discussions as well as correspondence on the issues raised.

1.2 METHOD AND PROGRAMME

Revisions were required only to those aspects of the Guidance Notes relating to concrete structures and were to be prepared to take account of the latest research, most particularly the Concrete-in-the-Oceans programme. Prior to this assignment, the Department had produced revisions to the Guidance Notes in respect of prestressing components and cathodic protection of concrete structures. The relevant background sections on prestressing components are not included in the present document. The sections proposed for revision were:

4.2.3 CONCRETE DESIGN

6.4 STRUCTURAL CONCRETE

7.3 CONSTRUCTION STANDARDS

The project, to the stage of publication of Draft Guidance Notes and the Background Report for comment, was planned in six phases. These may be summarized as:

- **Phase 1** - This phase commenced with an initial appraisal of the current Guidance Notes as well as other codes and regulations which resulted in a series of comments on the existing notes and on the revisions required. The major activity was, inevitably, the establishment of a data base for the project. A limited appraisal of this data was carried out with a view to identifying the major areas of revision required and the most significant research findings of the past few years. From these broad studies, was derived an outline of the proposed Draft Guidance Notes and the Background Report.

- **Phase 2** - A detailed study and appraisal of the latest research data, was carried out concentrating on the data available from the Concrete-in-the-Oceans programme and on those areas of research that indicated the need for significant revision to the Guidance Notes.

- **Phase 3** - Having developed and finalised the basic structure of both the Guidance Notes and the Background Report, the first draft of the documents was prepared. In addition, discussions were held with owners of existing platforms in the UK Sector of the North Sea to obtain feedback on the in-service performance of concrete structures.
• Phases 4 and 5 - During these phases the draft documents were amended in accordance with the Steering Group comments and refined to produce a final draft submitted to the final Steering Group meeting. As final data became available from the Concrete-in-the-Oceans programme, during these phases, it was assimilated into the draft documents. However, due to delays in the Concrete-in-the-Oceans programme and additional studies undertaken as part of this assignment, these phases became significantly longer than originally planned. These additional studies included a detailed calibration of the draft fatigue clause.

• Phase 6 - Following the final Steering Group meeting, required amendments to the documents were made and the final versions printed, bound and issued as "Drafts for Comment".

The revision project was commenced in January 1984 and completed, with the issue of this report, in July 1986.

1.3 DATABASE

A primary task, required for an assignment of this type, was the collection, collation and assimilation of the necessary literature to form a database. To this end, an extensive literature search was made, the resultant references indexed and copies of the references (or, in some cases, abstracts) assembled for use by the project team. For ease of use, a computer based information listing was used.

The full listing of all the references is included as an Appendix to this report.

References were collected from two main sources; firstly from the Offshore Certification Bureau libraries, utilizing established references used for other projects and, secondly, a search, back to 1979, of two technical literature indexes (Compendex and EI Meetings) by a computer based information retrieval service, ESA. The search was limited to 1979 as it was considered that a five year period would provide sufficient data beyond that identified elsewhere. The search yielded approximately 460 references which were sifted by the project team to produce the listed references.

In addition, data and reports resulting from both Phases I and II of the Concrete-in-the-Oceans programme were included, and provided, perhaps, the most important element of the database. These papers were available to the team via the position of Sir William Halerow and Partners as contributors to the programme and as Steering Group members.

References were initially entered in a computer index, arranged alphabetically by author. Entries were in the standard format of author, title, publisher (or publication) and date. The index was mounted on the Bureau's computer using the INFO information management system which provided extensive manipulation facilities.

Each reference was listed alphabetically by author on the master list and also had at least one keyword referring to its subject area. Thus references could be extracted by subject either on main subject (e.g. Design) or a sub-division of a main subject (e.g. Deflection). The list of keywords is given in the following Table 1.
Table 1  
Keyword list for database management

<table>
<thead>
<tr>
<th>Keyword</th>
<th>Subject</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td></td>
</tr>
<tr>
<td>D1</td>
<td>Durability (cover, cracking and corrosion)</td>
</tr>
<tr>
<td>D2</td>
<td>Shells (caissons, implosion, buckling)</td>
</tr>
<tr>
<td>D3</td>
<td>Shear (impact: dropped objects and ship impact)</td>
</tr>
<tr>
<td>D4</td>
<td>Temperature effects</td>
</tr>
<tr>
<td>D5</td>
<td>Fatigue</td>
</tr>
<tr>
<td>D6</td>
<td>Deflection</td>
</tr>
<tr>
<td>D7</td>
<td>General</td>
</tr>
<tr>
<td>Materials</td>
<td></td>
</tr>
<tr>
<td>M1</td>
<td>Cement</td>
</tr>
<tr>
<td>M2</td>
<td>Aggregates</td>
</tr>
<tr>
<td>M3</td>
<td>Water</td>
</tr>
<tr>
<td>M4</td>
<td>Admixtures</td>
</tr>
<tr>
<td>M5</td>
<td>Grout</td>
</tr>
<tr>
<td>M6</td>
<td>Reinforcement</td>
</tr>
<tr>
<td>M7</td>
<td>General</td>
</tr>
<tr>
<td>Construction</td>
<td></td>
</tr>
<tr>
<td>C1</td>
<td>General</td>
</tr>
<tr>
<td>C2</td>
<td>Inspection and Maintenance</td>
</tr>
<tr>
<td>Standards</td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>Standards, Guides and Codes of Practice</td>
</tr>
</tbody>
</table>

Some references on the master list have, after the paper title and before the keyword, an additional designation A. This signified that only an abstract of the paper was held in the database and not the full published paper. Also, some entries were not physically held in the database and were a reference only; these items were given a designation R.
2. EXISTING GUIDANCE NOTES (3RD EDITION)

2.1 EXISTING GUIDANCE NOTES AND OTHER CODES


In the initial stages of the assignment, existing Guidance Notes, including all the revisions up to and including June 1983, were considered. It is, perhaps, inappropriate to discuss these comments in detail as they no longer have relevance to a current document. However, one important change was considered appropriate which does have an impact on the understanding of the revision, this is:

- Recommendations in the Guidance Notes are in the form of variations from CP 110 and/or FIP Recommendations rather than positively stated requirements. It was felt that they would be clearer if, for each topic, specific recommendations were given even where this takes the form of directing the reader to a clause of an existing code.

The revisions to the Guidance Notes were considered in the light of existing codes and their possible future revision. At the time of undertaking this study, the situation appeared to be:

- **BS 8110**: This Standard was issued towards the end of the assignment.

At the commencement of the assignment CP 110 was in force and little progress had been made towards finalising BS 8110 since the closing date for public comment on Parts 1 and 2 in 1982. It seemed that Part 1 was complete except for matters relating to durability, which had caused some concern, and there was a possibility of it being finalised for submission to the main BSI Committee for approval. A draft of the revised code was available to the project team. BS 8110 was revised as follows:

- Part 1 February 1993
- Part 2 May 1989
- Part 3 May 1989

- **FIP (4th Edition)**: This was in the final draft for public comment stage and had recently been submitted to the FIP Council for approval in the early stages of the Guidance Notes revision project. Towards the end of the assignment the published document became available.

- **BS 6349: Part 1: 1984**: This is the Code of Practice for Maritime Structures, Part 1 being the General Criteria. It was technically complete and being proof read in anticipation of publication during the Guidance Note revision.
• **BS 6235: 1982**: This was published in 1982 and an amendment was published for comment in December 1983. The entire code was also being revised; a paper had been prepared putting forward revisions required to Section 7: Concrete Structures, for consideration by the Drafting Committee. The Standard was withdrawn in 1987 and not replaced. For completeness however, references to BS 6235 have been retained.

• **ACI 357R-84**: This was the draft copy, not for publication, of the 1978 guide for the Design and Construction of Fixed Offshore Concrete Structures.

• **Eurocode No. 2**: This was the draft released by the Eurocode Commission, for public comment.

In addition to the above codes, due cognizance was paid to the rules of the various Certifying Authorities. These were:

• **Det Norske Veritas**: The most recent rules were published in 1989 but the comments herein are based on the rules published in 1977.

• **Offshore Certification Bureau**: The rules were published in 1981 and plans were being made for appropriate revision and updating.

• **Lloyds**: No formal rules were published but various draft documents were available and continuously under review. During the project, more formal documents were in course of preparation.

A comparison of these existing Guidance Notes, Codes and Standards was carried out and is given in the following Tables 2 to 11.

The present situation with regard to these documents is understood to be:

• **BS 6349**: This was published in seven parts between 1984 and 1994.

• **Eurocode No. 2**: This has now been issued by CEN as DD ENV 1992, Parts 1,3,4,5 and 6.

• **Det Norske Veritas**: "Rules for Design, Construction and Inspection of Offshore Structures" is no longer available and has been replaced by "Rules for Fixed Offshore Installations". These rules were last updated in 1994. "Guidelines for the Design, Construction and Classification of Floating Concrete Structures", is also no longer available.

• **Offshore Certification Bureau**: The rules published in 1981 have been superseded by computer software entitled OILRIG (Offshore Industries Legislation Regulations Information and Guidance) which is available on annual subscription and regularly updated.

• **Lloyds**: "Rules and regulations for the Classification of Fixed Offshore Installations, Part 5, Concrete Structures" has been issued as four documents last updated in 1989.
<table>
<thead>
<tr>
<th>Table 2</th>
<th>Comparison of rules/codes: design (sheet 1 of 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CRACKING</strong></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>D.Én (3rd Edition)</strong></th>
<th>Crack widths should be calculated either as recommended in CP110 (Appendix A3) or by limiting tensile stresses in reinforcement.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>OCB</strong></td>
<td>As D.Én</td>
</tr>
<tr>
<td><strong>LLOYDS</strong></td>
<td>Reinforced concrete: In submerged and atmospheric zones crack widths should not exceed 0.3mm for normal environmental conditions and reinforcing stress should not exceed 0.8fy for extreme environmental conditions. In splash zone crack widths should not exceed 0.1mm for normal environmental conditions and 0.3mm for extreme environmental conditions.</td>
</tr>
<tr>
<td><strong>DnV (1977)</strong></td>
<td>Control of cracking may be based either on calculation of stresses in the reinforcement or on calculation of crack widths.</td>
</tr>
<tr>
<td><strong>EUROCODE NO. 2</strong></td>
<td>Reinforced concrete: Crack widths should not exceed a recommended value of 0.2mm for severe ambient conditions of exposure. Crack widths should be calculated as a characteristic crack width in accordance with CEB-FIP formula.</td>
</tr>
<tr>
<td><strong>BS6349</strong></td>
<td>As CP110 (1972) but not as BS 8110</td>
</tr>
</tbody>
</table>
### CRACKING

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP110</td>
<td>Reinforced concrete: surface crack widths should not exceed 0.3mm. In aggressive environments, width of cracks should not exceed 0.004 times nominal cover to main reinforcement.</td>
</tr>
<tr>
<td>BS8110</td>
<td>Cracking in reinforced concrete will normally be controlled by compliance with detailed rules on spacing of reinforcement. Where specific attention is required to limit the design crack width to particular values a formula (CP110 formula) is proposed.</td>
</tr>
<tr>
<td>BS6235</td>
<td>Reinforced concrete: Assessed surface width of cracks should not exceed 0.004 times nominal cover to main reinforcement and should never exceed 0.3mm. In normal operating conditions in areas designed for storage this value should be 0.1mm: In extreme environmental conditions this value should be 0.3mm.</td>
</tr>
<tr>
<td>FIP (3RD)</td>
<td>As BS6235 (1982)</td>
</tr>
<tr>
<td>FIP (4TH)</td>
<td>Reinforced concrete: Crack widths should not exceed 0.3mm. Crack widths should be calculated using reliable crack width formulae otherwise excessive cracking should be avoided by designing for low steel stresses.</td>
</tr>
<tr>
<td>ACI (1978)</td>
<td>Control of cracking should be based on limiting reinforcing stresses. Reference should be made to Table 4.1 which is intended to serve as a guide to limiting such stresses.</td>
</tr>
<tr>
<td>Cover</td>
<td>Details</td>
</tr>
<tr>
<td>-------</td>
<td>---------</td>
</tr>
<tr>
<td>OCB</td>
<td>As BS6235 (1982)</td>
</tr>
<tr>
<td>LLOYDS</td>
<td>As FIP (3rd Edition)</td>
</tr>
<tr>
<td>DnV (1977)</td>
<td>Concrete cover not to be less than 40mm in atmospheric zone, not subject to severe splashing, 50mm for all other parts of the structure and not less than 1.5 times the nominal maximum size of the aggregate.</td>
</tr>
<tr>
<td>EUROCODE NO. 2</td>
<td>Minimum concrete cover for reinforcing steel in severe ambient conditions and highly liable to corrosion is not to be less than 40mm (assuming concrete grade at least C40).</td>
</tr>
<tr>
<td>BS6349</td>
<td>Cover to maritime structures should not be less than 50mm</td>
</tr>
<tr>
<td>Code</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>CP110</td>
<td>Nominal cover to reinforcement (exposed to sea water: very severe conditions of exposure) is to be 60mm for grade 40 concrete and 50mm for grade 50 concrete for all other exposure zones.</td>
</tr>
<tr>
<td>BS8110</td>
<td>Nominal cover to reinforcement (exposed to sea water: extreme conditions of exposure) is 60mm for grade 45 concrete and 50mm for grade 50 concrete for all other exposure zones.</td>
</tr>
<tr>
<td>BS6235</td>
<td>Recommended nominal cover to reinforcement is 60mm in submerged zone and 75mm in splash and atmospheric zones, and should be not less than the greatest of 1.5 times nominal maximum size of aggregate, 1.5 times maximum diameter of reinforcement and for bundled bars 1.5 times equivalent bar diameter.</td>
</tr>
<tr>
<td>FIP (3RD)</td>
<td>For wall thickness greater than 0.5m nominal cover to be 60mm in submerged zone and 75mm in splash and atmospheric zones. For thinner walled structures nominal cover should not be less than 1.5 times nominal maximum size of aggregate or 1.5 times maximum diameter of reinforcement, whichever is the greater.</td>
</tr>
<tr>
<td>FIP (4TH)</td>
<td>For wall thickness greater than 0.5m nominal cover to be 50mm in submerged zone and 65mm in splash and atmospheric zones. For thinner walled structures as FIP (3rd).</td>
</tr>
<tr>
<td>LOAD FACTORS</td>
<td></td>
</tr>
<tr>
<td>--------------</td>
<td></td>
</tr>
<tr>
<td><strong>D.En (3rd Edition)</strong></td>
<td>Load conditions as FIP (3rd Ed.). Load Factors may be decided by agreement between owner and certifying authority, provided these are compatible with the values set out in CP110 (1972) and FIP (3rd Ed.). Load Factor for environmental loads to be 1.4 for all cases until further notice.</td>
</tr>
<tr>
<td><strong>OCB</strong></td>
<td>As D.En</td>
</tr>
<tr>
<td><strong>LLOYDS</strong></td>
<td>Ultimate Limit State: Load Factors quoted for two basic conditions: (1) Normal environmental and (2) Extreme environmental. Serviceability Limit State: Load Factors quoted and discussed in Section R3.3.19 to R3.3.22.</td>
</tr>
<tr>
<td><strong>DnV (1977)</strong></td>
<td>Ultimate Limit State: Load Factors quoted for two basic conditions: (1) Ordinary and (2) Extreme. Following text discusses values to be taken for temporary and unmanned conditions plus ULS for fatigue and progressive collapse. Serviceability Limit State: Load Factors quoted in Section 4.4.4.6.</td>
</tr>
<tr>
<td><strong>EUROCODE No. 2</strong></td>
<td>Suggested values are given for the various load types for two conditions: (1) unfavourable effects and (2) favourable effects.</td>
</tr>
<tr>
<td><strong>BS6349</strong></td>
<td>No Load Factors quoted</td>
</tr>
</tbody>
</table>
### Table 4
Comparison of rules/codes: design (sheet 2 of 2)

<table>
<thead>
<tr>
<th>LOAD FACTORS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CP110</strong></td>
</tr>
<tr>
<td>Ultimate Limit State: Load Factors quoted for the three load combinations: (1) dead + imposed, (2) dead + wind, and (3) dead + imposed + wind. Serviceability Limit State: Load Factors quoted for same three combinations as above.</td>
</tr>
<tr>
<td><strong>BS8110</strong></td>
</tr>
<tr>
<td>Ultimate Limit State: Load Factors quoted for three load combinations as CP110 (1972) but two load types Adverse and Beneficial. Serviceability Limit State: Loads and Load Factors as discussed in Section 9 Part 2.</td>
</tr>
<tr>
<td><strong>BS6235</strong></td>
</tr>
<tr>
<td>Ultimate Limit State: Load Factors quoted for three basic conditions: (1) Normal operating, (2) Extreme environmental and (3) Temporary load. Serviceability Limit State: Load Factors quoted and discussed in Section 7.1.4.2.1.</td>
</tr>
<tr>
<td><strong>FIP (3RD)</strong></td>
</tr>
<tr>
<td>Ultimate Limit State: Load Factors quoted for two basic conditions: (1) Normal environmental and (2) Extreme environmental. Factors for temporary load conditions discussed in R3.3.3.1. Serviceability Limit State: Load Factors quoted and discussed in Section R3.3.4.1</td>
</tr>
<tr>
<td><strong>FIP (4TH)</strong></td>
</tr>
<tr>
<td>Ultimate Limit State: Load Factors quoted for two basic conditions: (1) Normal environmental and (2) Extreme environmental for ULS of rupture, progressive collapse and fatigue. Serviceability Limit State: Load Factors quoted for SLS of Durability/deflection/etc and Permanent damage.</td>
</tr>
<tr>
<td><strong>ACI (1978)</strong></td>
</tr>
<tr>
<td>Ultimate Limit State: Load Factors quoted for two basic conditions: (1) Normal environmental and (2) Extreme environmental. Serviceability Limit State: Load Factors quoted and discussed in Section 4.4.2.</td>
</tr>
<tr>
<td><strong>ACI (1984)</strong></td>
</tr>
<tr>
<td>As ACI (1978)</td>
</tr>
<tr>
<td>Code</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>D.En</td>
</tr>
<tr>
<td>OCB</td>
</tr>
<tr>
<td>LLOYDS</td>
</tr>
<tr>
<td>DnV</td>
</tr>
<tr>
<td>EUROCODE No. 2</td>
</tr>
<tr>
<td>BS6349</td>
</tr>
<tr>
<td>Code</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>CP110</td>
</tr>
<tr>
<td>BS8110</td>
</tr>
<tr>
<td>BS6235</td>
</tr>
<tr>
<td>FIP (3RD)</td>
</tr>
<tr>
<td>FIP (4TH)</td>
</tr>
<tr>
<td>ACI (1978)</td>
</tr>
<tr>
<td>Code</td>
</tr>
<tr>
<td>-------------</td>
</tr>
<tr>
<td>D.En (3rd Edition)</td>
</tr>
<tr>
<td>OCB</td>
</tr>
<tr>
<td>LLOYDS</td>
</tr>
<tr>
<td>DnV (1977)</td>
</tr>
<tr>
<td>EUROCODE No. 2</td>
</tr>
<tr>
<td>BS6349</td>
</tr>
<tr>
<td>Code</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>CP110</td>
</tr>
<tr>
<td>BS8110</td>
</tr>
<tr>
<td>BS6235</td>
</tr>
<tr>
<td>FIP (3RD)</td>
</tr>
<tr>
<td>FIP (4TH)</td>
</tr>
<tr>
<td>ACI (1978)</td>
</tr>
</tbody>
</table>
Table 7  
Comparison of rules/codes: analysis (sheet 1 of 2)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>IMPLOSION</td>
<td></td>
</tr>
</tbody>
</table>

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D.En (3rd Edition)</td>
<td>Not covered</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td>OCB</td>
<td>Not covered</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td>LLOYDS</td>
<td>The following Load Factors should be used when designing against implosion failure: 1.2 for short-term load when a failure will not immediately endanger human lives and 1.5 for long-term load when a failure will immediately endanger human lives.</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td>DnV (1977)</td>
<td>Not specifically covered</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td>EUROCODE No. 2</td>
<td>Not covered</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------</td>
</tr>
<tr>
<td>BS6349</td>
<td>Not covered</td>
</tr>
</tbody>
</table>

18
<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>IMPLOSION</strong></td>
<td></td>
</tr>
<tr>
<td>CP110</td>
<td>Not covered</td>
</tr>
<tr>
<td>BS8110</td>
<td>Not covered</td>
</tr>
<tr>
<td>BS6235</td>
<td>Minimal comment</td>
</tr>
<tr>
<td>FIP (3RD)</td>
<td>Not covered</td>
</tr>
<tr>
<td>FIP (4TH)</td>
<td>Minimal comment</td>
</tr>
<tr>
<td>ACI (1978)</td>
<td>Design should be checked to prevent catastrophic collapse during periods of large hydrostatic pressure exposure. Potential failure modes to be considered should be material failure and structural instability.</td>
</tr>
<tr>
<td>Code</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>D.En (3rd Edition)</td>
<td>General comment only</td>
</tr>
<tr>
<td>OCB</td>
<td>As D.En</td>
</tr>
<tr>
<td>LLOYDS</td>
<td>Forces caused by short term thermal loading are usually very high and should be given special consideration. Forces caused by long-term thermal loading are highly influenced by changes in the initial stiffness due to cracking and creep. Unless other values are shown to be more appropriate a reduction of stiffness of 35% may be used.</td>
</tr>
<tr>
<td>DnV (1977)</td>
<td>Accurate calculation of thermal stresses can only be obtained from a non-linear analysis. Effects due to imposed deformations may be calculated using a linear elastic model and a constant modulus of elasticity throughout the structure. Possible reductions due to cracking may be estimated separately.</td>
</tr>
<tr>
<td>EUROCODE No. 2</td>
<td>Not covered</td>
</tr>
<tr>
<td>BS6349</td>
<td>General comment only</td>
</tr>
<tr>
<td><strong>TEMPERATURE EFFECTS</strong></td>
<td></td>
</tr>
<tr>
<td>-------------------------</td>
<td></td>
</tr>
<tr>
<td><strong>CP110</strong></td>
<td></td>
</tr>
<tr>
<td>If a structure is subjected to temperature effects greater than those known from experience to be inconsequential, the resulting internal forces and their effect on the structure as a whole should be considered when assessing crack widths or other forms of local damage.</td>
<td></td>
</tr>
<tr>
<td><strong>BS8110</strong></td>
<td></td>
</tr>
<tr>
<td>Not specifically mentioned.</td>
<td></td>
</tr>
<tr>
<td><strong>BS6235</strong></td>
<td></td>
</tr>
<tr>
<td>The temperature ranges of gas or fluid and sea and air should be considered and the structure designed to resist the maximum induced thermal stresses based upon $E$ values given in Appendix D of CP110 (1972). The rate of change of temperature should be considered.</td>
<td></td>
</tr>
<tr>
<td><strong>FIP (3RD)</strong></td>
<td></td>
</tr>
<tr>
<td>The temperature ranges of oil, sea and air should be considered, and the structure designed to resist the maximum induced thermal stresses. The rate of change of temperature should be considered.</td>
<td></td>
</tr>
<tr>
<td><strong>FIP (4TH)</strong></td>
<td></td>
</tr>
<tr>
<td>As FIP (3rd Edition)</td>
<td></td>
</tr>
<tr>
<td><strong>ACI (1978)</strong></td>
<td></td>
</tr>
<tr>
<td>To assess thermally induced creep the reduced modulus of elasticity method should be used if all structural components are subjected to the same temperature change. For non uniform temperature distributions a more refined methodology is essential to identify unfavourable force distribution.</td>
<td></td>
</tr>
<tr>
<td><strong>ACI (1984)</strong></td>
<td></td>
</tr>
<tr>
<td>As ACI (1978)</td>
<td></td>
</tr>
<tr>
<td>Code</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>D.En (3rd Edition)</td>
<td>Not specifically covered</td>
</tr>
<tr>
<td>OCB</td>
<td>Not covered</td>
</tr>
<tr>
<td>LLOYDS</td>
<td>Shear and torsion resistance: Appendix 1 details design procedures which are based on CP110 (1972), in particular for prestress due to hydrostatic forces. Membrane shear resistance: Appendix 2 details design procedures.</td>
</tr>
<tr>
<td>DnV</td>
<td>Recommendations based on the principle of addition of (1977) component resistances to determine the total shear resistance. Alternatively the load carrying capacity may be determined from an analytical model using a failure criterion for the biaxial state of stress in the compression zone.</td>
</tr>
<tr>
<td>EUROCODE No. 2</td>
<td>A minimum shear reinforcement shall be provided in all cases, independently of the results of any calculations.</td>
</tr>
<tr>
<td>BS6349</td>
<td>Not specifically covered</td>
</tr>
<tr>
<td>CP110</td>
<td>Shear reinforcement should be provided if the calculated shear stress exceeds the recommended values as given in Table 5 but in no case should the shear stress exceed the values given in Table 6.</td>
</tr>
<tr>
<td>BS8110</td>
<td>The calculated design shear stress should in no case exceed 0.8 kN/m² or 5 N/mm² whichever is the lesser whatever reinforcement is provided. (This limit includes a material factor of 1.25).</td>
</tr>
<tr>
<td>BS6235</td>
<td>Not specifically covered</td>
</tr>
<tr>
<td>FIP (3RD)</td>
<td>Not specifically covered</td>
</tr>
<tr>
<td>FIP (4TH)</td>
<td>Not specifically covered</td>
</tr>
<tr>
<td>ACI (1978)</td>
<td>The total shear force that can be resisted at a section may be taken as the sum of the component forces contributed from the concrete, reinforcing steel and prestressing steel. Design and detailing of sections of shear should follow recommendations of ACI 318-77.</td>
</tr>
<tr>
<td>Code</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>D.En</td>
<td>Concrete construction should be carried out in accordance with recommendations of CP110 (1972) supplemented by FIP (3rd Edition). Concrete should be a designed mix and minimum grade to be grade 40 for exposure condition: exposed to salt water or salt water spray.</td>
</tr>
<tr>
<td>OCB</td>
<td>As D.En</td>
</tr>
<tr>
<td>LLOYDS</td>
<td>As FIP (3rd Edition)</td>
</tr>
<tr>
<td>DnV</td>
<td>Construction requirements detailed on subjects specifically relevant to concrete offshore structures including tolerances, joints, reinforcement, the mixing and placing of concrete, concreting in adverse conditions, concreting under water, the curing of concrete, and inspection.</td>
</tr>
<tr>
<td>EUROCODE No. 2</td>
<td>General requirements detailed on concrete, reinforcement and prestressing tendons. Does not provide specific requirements applicable to offshore concrete structures.</td>
</tr>
<tr>
<td>BS6349</td>
<td>Not covered</td>
</tr>
<tr>
<td><strong>Table 10</strong></td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td></td>
</tr>
<tr>
<td>Comparison of rules/codes: specifications (sheet 2 of 2)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
</tr>
</thead>
</table>

<p>| CP110 | General requirements detailed with regard to concrete, reinforcement and prestressing tendons. Does not provide specific requirements applicable to offshore concrete structures. Does cover in detail most concreting subjects. |
| BS8110 | As CP110 (1972) but includes an additional subject of concreting in hot weather. |
| BS6235 | Concrete construction and workmanship should comply with the requirements of CP110 (1972) except for amended requirements such as joints, curing, concreting in cold weather and reinforcement which are relevant to offshore structures. |
| FIP (3RD) | Construction methods and workmanship should follow accepted practices as described in relevant Standards Codes and specialist literature. Only additional recommendations specifically relevant to concrete sea structures are detailed including joints, curing and tolerances. |
| FIP (4TH) | As FIP (3rd Edition) |
| ACI (1978) | Construction methods and workmanship should follow accepted practices as described in ACI 318-77, API RP2A and the specialist literature. Only additional recommendations specifically relevant to concrete sea structures are detailed. |
| ACI (1984) | As ACI (1978) but includes additional subjects on connection of adjoining structures, prevention of damage due to freezing and relocation. |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>D.En</strong></td>
<td>Concrete, reinforcement and prestressing tendons should comply</td>
</tr>
<tr>
<td>(3rd Edition)</td>
<td>with CP110 (1972) supplemented by FIP (3rd Edition), except</td>
</tr>
<tr>
<td></td>
<td>where supplemented or amended for the sections detailed on</td>
</tr>
<tr>
<td></td>
<td>cement type, aggregates, water, admixtures, reinforcement and</td>
</tr>
<tr>
<td></td>
<td>prestressing tendons.</td>
</tr>
<tr>
<td><strong>OCB</strong></td>
<td>As D.En</td>
</tr>
<tr>
<td><strong>LLOYDS</strong></td>
<td>As FIP (3rd Edition)</td>
</tr>
<tr>
<td><strong>DnV</strong></td>
<td>Specifications or certificates for materials to be used in</td>
</tr>
<tr>
<td>(1977)</td>
<td>construction are to be submitted to DnV for approval. It is to</td>
</tr>
<tr>
<td></td>
<td>be certified that the properties of materials under</td>
</tr>
<tr>
<td></td>
<td>consideration are adequate for the intended purpose.</td>
</tr>
<tr>
<td><strong>EUROCODE</strong></td>
<td>Concrete, reinforcement and prestressing tendon requirements</td>
</tr>
<tr>
<td>No. 2</td>
<td>detailed but in general terms and not with regard to offshore</td>
</tr>
<tr>
<td></td>
<td>concrete structures.</td>
</tr>
<tr>
<td><strong>BS6349</strong></td>
<td>Materials should conform to the requirements of CP110 (1972)</td>
</tr>
<tr>
<td></td>
<td>except where supplemented or amended with regard to maritime</td>
</tr>
<tr>
<td></td>
<td>structures but not specifically offshore concrete structures.</td>
</tr>
<tr>
<td>MATERIALS</td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td></td>
</tr>
</tbody>
</table>

| CP110 | Engineer's responsibility to specify the type of concrete required to ensure both the strength and durability of the finished structure. Details are given on mixes, cement type, aggregates, water, admixtures, grades, reinforcement and prestressing tendons. |
| BS8110 | Materials used should satisfy the design requirements for the safety, structural performance, durability and appearance of the finished structure taking full account of the environment to which it will be subjected. Details are given on choice of concrete, aggregates, mixes, reinforcement and prestressing tendons. |
| BS6235 | Materials should comply with CP110 (1972) except where supplemented or amended as detailed for cement, aggregates, admixtures, reinforcement and prestressing tendons. |
| FIP (3RD) | Materials should conform to the requirements to produce a concrete with low permeability. Details requirements on cement, aggregates, water, admixtures, reinforcement and prestressing tendons. |
| FIP (4TH) | As FIP (3rd Edition) |
| ACI (1978) | All materials to be used in the construction of offshore concrete structures should have documentation demonstrating previous satisfactory performance under similar site conditions or have sufficient backup test data. |
2.2 DESIGN PARAMETERS FOR PRESENT STRUCTURES

Offshore structures have been in existence for a relatively short time and so service life cannot be totally indicative of long term performance. Nevertheless, it appeared prudent to make as much use of experience as possible and, to assist in this process, discussions were held with operators responsible for concrete platforms in the UK sector of the North Sea.

A total of twenty-one concrete platforms had been constructed or were planned in the North Sea up to 1986. Ten of these were located in the UK sector. The location of these concrete platforms is indicated in Figure 1 and a detailed listing given in Table 12.

It was felt appropriate to obtain the desired feedback on existing structures by discussions with owners rather than Certifying Authorities or contractors because of confidentiality difficulties. To act as an 'aide memoire' to the discussions a questionnaire was proposed. This, however, was not a series of detailed questions but more general headings to provide a basis for appropriate discussion. The extent to which any one topic was pursued depended upon the particular structure under discussion and its history. The main thrust of the discussions was to identify any failure or problems with the platform, so that appropriate design guidance for new structures could be formulated.

The feedback from industry has led directly to various additional clauses in the Guidance Notes. These are:

- The use of drawdown has led to operating and maintenance problems with some platforms. An additional drawdown clause has been introduced.

- Extra comments have been included, in both the Notes and Report, in respect of construction joints where experience has indicated that problems can occur.

- Many concrete platforms include storage, and the need for process engineering, as well as structural engineering, design of the storage vessels has been identified. There is some record of problems with gas and light ends entrainment, followed by subsequent separation during handling, caused by storage under pressure.

- Generally the amount of marine growth on the platforms considered is small but problems have occurred where removal with water jets has been necessary. Comments have been included in the inspection section of the background report on the desirability, whenever possible, of not cleaning marine growth.

The detailed information received on the existing structures is discussed in the following sub-sections for each of the main discussion topics.
Figure 1
Location of concrete platforms in the North Sea
Table 12
Concrete platforms in European waters

<table>
<thead>
<tr>
<th>Sector</th>
<th>Field</th>
<th>Owner</th>
<th>Number of Concrete Platforms</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>UK</td>
<td>Beryl</td>
<td>Mobil</td>
<td>1</td>
<td>Beryl A</td>
</tr>
<tr>
<td></td>
<td>Brent</td>
<td>Shell</td>
<td>3</td>
<td>Brent B, C &amp; D</td>
</tr>
<tr>
<td></td>
<td>Cormorant</td>
<td>Shell</td>
<td>1</td>
<td>Cormorant South</td>
</tr>
<tr>
<td></td>
<td>Dunlin</td>
<td>Shell</td>
<td>1</td>
<td>Dunlin A</td>
</tr>
<tr>
<td></td>
<td>Frigg</td>
<td>Elf/Total</td>
<td>3</td>
<td>Frigg CDP1, TP1 &amp; MCP01</td>
</tr>
<tr>
<td></td>
<td>Ninian</td>
<td>Chevron</td>
<td>1</td>
<td>Ninian Central</td>
</tr>
<tr>
<td></td>
<td>Ravenspurn</td>
<td>Hamilton Bros</td>
<td>1</td>
<td>Ravenspurn North</td>
</tr>
</tbody>
</table>

Total in UK Waters 11

| Norwegian | Ekofisk | Phillips | 1 | Ekofisk Centre |
| Frigg      | Elf/Total | 1 | Frigg TCP2      |
| Statfjord  | Mobil/Conoco | 3 | Statfjord A, B & C |
| Gullfaks   | Statoil   | 2 | Gullfaks A & B  |

Total in Norwegian Waters 7

| German | Schwedeneck | Texaco | 2 |

Total Platforms Installed 20
2.2.1 Structure

Of the eleven concrete structures in the UK sector of the North Sea, two are perforated wave wall designs and the remainder are tower structures on cellular bases. The first platform to be installed was Beryl A, a Condeep structure, in 1975 with design rules being evolved as part of the design. Hence, the service experience in the UK sector is limited at the time of writing to about ten years. Earlier concrete structures were installed in the Norwegian sector but these were not included in the survey of existing structures.

Water depths vary from 94 to 150 metres with the structures located in the northern North Sea as indicated on Figure 1. Experience on several structures has indicated the desirability of greater freeboard due to wave runup. This affects the ability to inspect underdeck. Modifications or additions to the deck including accessways etc. cause additional wave loading and must themselves be designed for wave crest/spray/splash.

All foundations are gravity bases with, in most cases, skirts. No active foundation has proved necessary and settlements have generally been small and less than predicted. Several of the structures had scour mats installed but no significant movement has been noted except in one case where minimal scour has been identified under pipelines connecting to the platform.

The basic structural concept of all but one structure allowed for oil storage, although in another case the storage has not been developed or utilised. Drawdown has been used extensively to limit temperature stresses by causing additional pre-stress. Loss of drawdown has caused structural damage which could prevent oil storage.

The use of drawdown has given rise to several problems. On four platforms there has been temporary loss of drawdown; two instances were due to dropped objects, one due to ship impact and one due to mechanical damage of the drawdown system. In some cases, maintenance of the drawdown systems is difficult or impossible without reduction in drawdown due to pipe work being permanently submerged.

2.2.2 Design

A variety of design codes were used for the structural design. These include ACI 318-71, DnV rules both pre-publication and 1974, FIP Recommendations (1974) and CP110 (1972).

For all but one structure, calculation of crack widths appears to have been carried out in accordance with CP110 with calculated widths typically limited to 0.2 or 0.3mm or a maximum of 0.004 times the minimum concrete cover. The amount of cover specified to reinforcement has varied from 50mm to 100mm with generally a cover of 70mm being applied to all parts of the structure; one structure was constructed with a specified cover of 50mm + 10mm to all zones. In all cases additional cover was provided to pre-stressing tendons over that provided to reinforcement but this would tend to occur in any case as the tendons are located inside the reinforcement mats.

Implosion was considered as a failure mechanism for all the structures and, generally, the worst load case was during deck mating or installation. Since these were highly controlled temporary conditions, some operators, in conjunction with their Certifying
Authorities believed that lower load factors than for the permanent case were appropriate whereas others have regarded these stages of a project as being financially the most exposed for which higher load factors were appropriate.

Accidental impacts due either to dropped objects or ship impact were not considered in many of the original designs although all structures have been subjected to various forms of retrospective analysis. In some cases this analysis followed some form of accidental impact whilst in others it followed a general awareness that such accidents can and do occur; these checks have indicated that the structures are able to withstand significant impacts without permanent damage.

Fatigue analysis, as part of the design process was carried out for some of the platforms with design being to either DnV rules or FIP Recommendations. In at least one structure a tensile stress limitation of 140 N/sq.mm was applied for fatigue reasons.

None of the operators has observed any failures or deterioration attributable to fatigue. Fatigue tests, under laboratory conditions, have been carried out for rectangular cross-section reinforced concrete beams and failure in such cases has been by fracture of the reinforcement or breakdown of the concrete. In the case of more complex structural elements, such as walls and shells used in offshore platforms, the form of any fatigue failure is matter of conjecture. Areas particularly at risk have been suggested as being at the junction between columns and the base, openings or sudden changes of section and shear walls subjected to reversing loads.

Those structures designed for oil storage considered oil temperatures of 35°C to 39°C after allowing for cooling. In one case, temperatures of up to 55°C are allowed for hot flushing for limited periods and retrospective analysis for another structure indicates that original design for 35°C may be conservative with oil temperatures from the well of 75°C to 50°C being anticipated but actual temperatures being less than 60°C.

2.2.3 Materials

The approaches made to operators included requests for copies of the materials sections of their specifications. These were not available but nevertheless substantial general data was available and this indicated that the properties of the concrete used and their constituent materials were not matters of undue concern.

Table 13 gives a comparison of the concrete mixes used including data on latent hydraulic binders and admixtures. Concrete grades were typically C40 (characteristic strength of 40N/sq.mm) using ordinary Portland cement and, for most structures, some pfa replacement was used, although this was in some instances restricted to the splash zone. A variety of admixtures were used, particularly plasticizers and retarders for slipforming.
<table>
<thead>
<tr>
<th></th>
<th>Platform A</th>
<th>Platform B</th>
<th>Platform C</th>
<th>Platform D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete grade</td>
<td>Minimum grade to be C50.</td>
<td>Minimum grade to be C50.</td>
<td>-</td>
<td>OPC used throughout with PFA added.</td>
</tr>
<tr>
<td>Concrete mixes and</td>
<td>OPC used throughout with ground blastfurnace</td>
<td>OPC used throughout in two types of mix (1) with Hillhouse aggregate</td>
<td>OPC used throughout with ground blastfurnace slag.</td>
<td></td>
</tr>
<tr>
<td>type</td>
<td>slag.</td>
<td>(50PH) and (2) with Furnace aggregate (50PP). PFA added to mixes in concrete used in splash zone.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aggregate</td>
<td>-</td>
<td>Hillhouse and Furnace basalts.</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Admixtures</td>
<td>Retarders used to prevent sticking to shutters.</td>
<td>Retarders used to prevent sticking to shutters. Up to 1% plastocrete added to both mixes.</td>
<td>Retarders used to prevent sticking to shutters.</td>
<td>Retarders used to prevent sticking to shutters.</td>
</tr>
<tr>
<td>W/C ratio</td>
<td>-</td>
<td>50PH mix: W/C ratio 0.43</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Latent hydraulic binder</td>
<td>Yes</td>
<td>Yes in splash zone.</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>used</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other notes</td>
<td>Cube strength at 12 months was 70 N/sq.mm. No paint or coating applied.</td>
<td>Cube strength at 12 months was 60 N/sq.mm. No paint or coating applied.</td>
<td>No paint or coating applied.</td>
<td>No paint or coating applied.</td>
</tr>
<tr>
<td></td>
<td>Platform E</td>
<td>Platform F</td>
<td>Platform G</td>
<td>Platform H</td>
</tr>
<tr>
<td>------------------------</td>
<td>-------------------------------------------------</td>
<td>-------------------------------------------------</td>
<td>-------------------------------------------------</td>
<td>-------------------------------------------------</td>
</tr>
<tr>
<td><strong>Concrete grade</strong></td>
<td>Minimum grade to be C45.</td>
<td>Ground blast furnace slag was not used in the concrete.</td>
<td>Minimum grade to be C50</td>
<td>Four mixes used, cement content 410-450 kg, sand/coarse aggregate ratio 45/55 or 42/58.</td>
</tr>
<tr>
<td><strong>Concrete mixes and type</strong></td>
<td>OPC (PC330) used throughout. In general sand/coarse aggregate ratio of 50/50 used.</td>
<td>-</td>
<td>Two mixes: (1) 425 kg/cu.m cement with 105 kg/cu.m PFA (2) 375 kg/cu.m cement with PFA.</td>
<td></td>
</tr>
<tr>
<td><strong>Aggregate</strong></td>
<td>Glaciofluvial origin.</td>
<td>-</td>
<td>Glacoid morain used.</td>
<td>Sand from Hedgrev, coarse aggregate from Halden.</td>
</tr>
<tr>
<td><strong>Admixtures</strong></td>
<td>Plasticisers and retarders used throughout. Vinsol resin used in splash zone mixes.</td>
<td>Plasticisers and retarders used in mixes for slipforming.</td>
<td>Plasticisers and retarders used throughout. Melment used for precast units.</td>
<td>3 used: Sika BV 40, Sika Ret. and Sika AER.</td>
</tr>
<tr>
<td><strong>W/C ratio</strong></td>
<td>Submerged zone 0.45. Other zones 0.40</td>
<td>-</td>
<td>-</td>
<td>Either 0.40 or 0.45</td>
</tr>
<tr>
<td><strong>Latent hydraulic binder used</strong></td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td><strong>Other notes</strong></td>
<td>No paint or coating applied.</td>
<td>Cores taken indicate a compressive strength 25% greater than 28 day strength.</td>
<td>-</td>
<td>Mixes gave slumps of 100-150 mm. 28 day cylinder strength with air entrained was 52 N/sq.mm, without 55 N/sq.mm. 90 day cylinder strength with air entrained was 63 N/sq.mm, without 63 N/sq.mm.</td>
</tr>
</tbody>
</table>
The design of the majority of structures did not make use of age factors, nor was any allowance made for loss of strength due to deep immersion. There were two exceptions to this generalisation:

- One structure used the anticipated one year cube strength for assessment of implosion due to deep immersion for deck mating.

- For one structure, loss of concrete strength due to pressure effects was considered as part of the design, with a 25% strength reduction considered.

Cores taken from one structure several years after installation indicated a compressive strength approximately 25% greater than the 28 day cube strength.

Only limited data was available on curing methods used but generally curing was minimal. Several operators commented that no special curing precautions had been taken. However, only one structure exhibited any reported shrinkage cracking, but this was where the operator commented that "curing had not been carried out". A post construction report, however, for one structure indicated that:

- curing of the concrete skirts, domes and inside face of the shaft was by the application of a curing compound.

- curing of both faces of the cell walls was by water spray underneath the slipform. (But there is no mention as to whether this was a fresh or sea water spray).

- curing of the outside face of the shaft was by application of a commercial heavy curing membrane consisting of one part of water emulsifying epoxy, one part of water and two parts ordinary Portland cement. This was used due to the severe exposure conditions with one layer in the submerged zone but two layers in the splash zone.

Although all operators stated that no form of paint or external coating was applied to the concrete, this last curing membrane must have remained effective for several years. There is a view, held by several very experienced engineers responsible for marine works, particularly in the Middle East, that a temporary coating protecting the young concrete from chloride attack is particularly beneficial to the durability of the structure. As the coating disintegrates, over a period of years, the concrete becomes exposed to chloride attack but by that time it is fully matured and hence more resistant to loss of passivity.

Concern has been expressed over both hydrogen sulphide and stagnant water degrading concrete. Investigations were carried out on an area subjected to attack by hydrogen sulphide with oxygen but within a depth of 1 to 2mm the attack appeared to have ceased with no further penetration. Where stagnant water has been present, no degradation has been identified.

An analysis of concrete quality obtained for one particular structure indicates that a high degree of control was achieved with a standard deviation of 3.9 N/sq.mm and an overall 28 day mean strength of 54 N/sq.mm. However, the concrete in the splash zone achieved a characteristic strength of 42.5 N/sq.mm against the specified 45 N/sq.mm and consequently drilled cores were required for approval.
2.2.4 Service performance

No failures of concrete structures due to fatigue or any other causes, apart from accidental damage, have been reported. No signs of any corrosion or concrete degradation have been noted. Operators have indicated confidence in their concrete structures and believe them to provide a durable solution for their original design lives.

However, at the time of writing this report, the oldest structure was approximately 10 years old and many have only been installed for 5 or 6 years. This service experience therefore is short and perhaps problems due to corrosion, for example, would not be anticipated for 10 or 20 more years. There are examples, in other types of marine structure, of serious corrosion of pre-stressing tendons not becoming apparent for 20 years.

Despite these generalised comments, the following specific details were given for particular structures:

- water ingress along a crack was prevented only by mastic coating on the outside surface. (Presumably this crack was a through thickness crack).

- all temporary pipes and other equipment (e.g. ballasting and grouting lines) have been identified as potential leakage paths.

- cracking, generally less than 0.3mm but with a maximum of 0.6mm, has been noted. This is thought to be due to:
  - reversal of stress during float out
  - temporary overload during prestressing transfer
  - temporary overload during construction
  - shrinkage
  - normal stresses within the design load.

- rust staining is evident at water level which it is believed is due to the corrosion of tying wire.

- cracks, generally hairline, have been noted in external diaphragm walls.

- a very small amount of seepage has been noted as occurring into and through the pre-stressing ducts at the base of a tower.

- some problems have occurred with slight loss of concrete from surface during water jetting for removal of marine growth.

- leakage has been noted where voids construction joints have been formed or have been infilled using letter box shutters.

- the use of sacrificial anodes, for cathodic protection of external steelwork, has been substantially greater than envisaged at the design stage.
Specific problems have been encountered with loss of drawdown as described in Section 2.2.1 of the report. This has been due to both failure of drawdown systems and accidental impact damage. All platforms have experienced accidental ship impacts or impact from dropped objects but, in many cases, this has not caused any damage. Detailed studies of these impacts have been carried out and reported by others (see Section 3.1.1 of this report).

Where structures have suffered damage this has taken the form of a hole in cell roof or a leg and, in each case, a repair had been carried out. Despite fears to the contrary, it has proved possible to effect repairs to concrete structures both under and above water.

The degree of marine growth experienced has varied enormously from structure to structure. It appears that several structures have no marine growth (visible by means of video) whereas others have significant growths requiring major cleaning programmes prior to inspection. One operator noted that drilling mud covered many parts of the structure and that cleaning was necessary for inspection.

### 2.2.5 Maintenance

All operators carry out a programme of continuous inspection leading to the five year re-certification. The inspections are typically carried out using video and photographs, in some cases from remote controlled vehicles with some diver input and in other cases predominantly by divers.

Where honeycombing occurred or other repairs were required during the construction, concrete was invariably cut out and replaced. In some instances by epoxy resin but in others by structural concrete with epoxy bonding agents and paints. Concreting of boxouts has led to subsequent problems with leakage but otherwise repairs are reported to be successful.

All structures except one have reinforcement bonded to external steelwork which is cathodically protected thus cathodically protecting the reinforcement. For one structure it is claimed that an electrical discontinuity has been preserved between the reinforcement and external steelwork; however, the operator reported concern that consumption of anodes has been considerably greater than anticipated.

### 2.3 REVISIONS REQUIRED TO GUIDANCE NOTES

The overall extent of the revisions found desirable to the Guidance Notes varied for each of the various sections and topics. These revisions and, where appropriate, new sections were brought about because of the advances that the industry has taken as a result of research and experience following the first generation of platforms constructed in the North Sea. In particular, the Concrete-in-the-Oceans programme has brought a considerable extension of the research effort.
The design sections of the Guidance Notes required the most significant revision with additional clauses covering Shear, Fatigue, Impact, Implosion and Drawdown. The requirements for Cracking and Cover were amended substantially in the light of the valuable Concrete-in-the-Oceans data. For the materials section, no significant changes in contents were undertaken but a general updating and re-arrangement of the relevant sections was done. Concrete strength grades were clarified and clauses relating to concrete reconsidered in line with the latest thinking in the industry. The only clauses in the existing Guidance Notes relating directly to construction topics cover pre-stressed concrete, which was outside the scope of this review. References were considered necessary on general construction topics covering all phases of construction, tow-out, installation and commissioning, including some suggestions for inspection of structures during construction.

A reference standard was required. The relevant sections of the existing Guidance Notes refer to CP110: "The Structural Use of Concrete" supplemented by the FIP Recommendations 1974. Since the publication of the original Notes, BS 6235: "Code of Practice for Fixed Offshore Structures", which provides guidance on both steel and concrete structures, has been published. It seemed appropriate that a British Standard should be used as the reference document for the Guidance Notes and either CP 110 or BS 6235 appeared to be the most appropriate. CP 110, later revised to BS 8110, was selected.

The existing Guidance Notes were drafted, for concrete, by quoting variations to both CP110 and FIP requirements. In preparing the revised draft Notes, specific guidance has been put forward on all topics even where this is a reference to CP110. Hence direct guidance is provided by direction rather than by omission.
3. TECHNICAL CONSIDERATIONS

3.1 DESIGN

3.1.1 Limit state and load factor design

A.L.L. Baker (1949) first proposed the concept of Limit State design when he defined a Limit State as 'a condition of a structure at which it ceases to function in the manner for which it was designed, and which can only be defined in terms of the limits of values which are subject to scatter'. This publication became the basis of CP110 published in 1972. CP110 was, in 1985, superseded by BS 8110; however the Limit State Design approach is essentially unaltered.

BS 8110 states that structures designed by the Limit State method will not become unfit for their purpose by collapse, overturning, or buckling (Ultimate Limit State), or by deformation, cracking, vibration etc (Serviceability Limit State).

An example of the concepts involved can be illustrated by reference to CP2007, first published in 1938 but now superseded by BS5337. The requirements laid down were that the water retaining structure should stand up (equivalent to Ultimate Limit State) while at the same time should not leak (equivalent to Serviceability Limit State). It should be noted that the design procedure in CP2007 was not Load Factor design and the terminology was not that of Limit State design but the concepts are appropriate. In design, all relevant Limit States should be considered so as to ensure an adequate degree of safety and serviceability. It is possible to identify three levels at which the design process may be treated. These are:

- Level 1: a semi-probabilistic design process
- Level 2: a probabilistic design process with some approximations
- Level 3: a full probabilistic analysis for the entire structural system.

Because of the urgency of development, the construction of offshore structures has preceded the clear definition of the reliability required of them. Industry's approach has been to adopt the Level 1 design procedure, for which codes of practice have been written accordingly. Thus, the design load is obtained by multiplying the characteristic load, defined and calculated in accordance with CP3: Chapter V, by an appropriate Load Factor. The Load Factor is introduced to take into account unconsidered increases in load, inaccurate assessment of load effects, unforeseen stress redistribution, variations in dimensional accuracy and the importance of the Limit State being considered.
Work undertaken by CIRIA on a Level 2 design approach concludes that statistical reliability theory has been sufficiently developed to provide a basis for calculating probabilities of attaining a Limit State. Further work by Flint and Baker (1976) has shown that this approach can be adopted with a slight modification to offshore structures. It notes that the existing design directives have not been in use long enough for reliance in back calibration to be entirely satisfactory. This is an important consideration as any new code for previously existing forms of construction should not permit construction of markedly less reliability than the least acceptable in current practice.

Experience to date suggests that the design of offshore concrete installations using Level 1 design procedures has produced safe and serviceable structures. All present codes of practice quote Load Factors which have been derived principally from industry consensus and experience rather than by statistical derivation. Hence, the degree of safety achieved may be in excess of that required but, until further experience in the use of reliability indices is gained, caution should be exercised in the use of load and material factors significantly different from those presently set out. A detailed reliability study may provide a valuable assessment of the actual levels of safety being achieved and thus allow adjustment of the relevant factors on a rational basis.

Codes of Practice for offshore structures outline recommended values for Load Factors for both the Ultimate Limit State and the Serviceability Limit State design.

In a comparison of Loading Factors for the Ultimate Limit State, many of the recommended values differ numerically between codes. All offshore codes detail Load Factors for similar environmental and load categories. BS 8110, however, is a general concrete code and lists safety factors in a manner not directly appropriate to concrete offshore structures. Although BS 8110 and other codes are not directly comparable, the values are numerically similar if like with like is compared. Offshore related codes disagree on whether hydrostatic or deformation loads should be treated as separate categories. The present Guidance Notes consider hydrostatic loads separately and treat deformation loads as imposed loading. FIP and others do the opposite, treating deformation loads separately and hydrostatic loads as dead loading.

A similar situation occurs for Loading Factors for Serviceability Limit State design. All offshore related codes give identical values for similar loading categories for the loading condition applicable for this Limit State. BS 8110 again considers Load Factors in a different manner and, as before, in a way not appropriate to concrete offshore structures.

The codes also take into account the effect on the Load Factors of other loading types such as differential settlement, creep, shrinkage, temperature, impact, implosion and fatigue. These effects may be significant in one Limit State but not in the other; for example, temperature is predominantly a Serviceability consideration whereas fatigue may be considered in both Ultimate and Serviceability Limit State design.

The design engineer must consider all relevant load categories at appropriate Limit States with the characteristic load value being determined and used together with an appropriate Load Factor. No detailed statistical reliability studies have been carried out as part of this or other studies to define such loads and load factors. Until such studies are available, factors derived by experience and consensus must remain in use as part of a Level 1 design.
3.1.2 Cracking/cover/durability

An assessment of appropriate requirements to assure durable concrete cannot be divorced from the particular environmental and in-service conditions involved, the design life, nor from the direct and consequential effects on safety and cost of any premature deterioration. A durable concrete is a dense material with the ability to withstand aggressive natural and industrial environments for very long periods.

This durability of reinforced concrete structures is, in general terms, affected by three parameters under the control of designer: cover, crack widths and concrete quality. In recent years much research has been carried out, most notably as part of the Concrete-in-the-Oceans programme, to investigate the effect of these parameters on corrosion rates as well as to study the underlying corrosion mechanisms. Many of the conclusions drawn are incompatible with each other and cannot, in the present state of knowledge, be wholly accepted for design. Nevertheless, they add significantly to the understanding of the influence of these parameters and allow present ‘good practice’ to be further refined.

As part of the assignment to revise the Guidance Notes, a substantial number of references have been identified and studied. Several references are, it is believed, of particular relevance to this topic and these are summarized in Table 14.

Steel reinforcement in concrete is protected by an iron oxide (Fe₂O₃) film around the reinforcement formed in the alkaline conditions. So long as this alkaline environment remains, typically with a pH of greater than 12.5, the steel is passivated and thus protected from corrosion. However, when this passivity has been lost due either to the lowering of alkalinity associated with carbonation of the concrete or to the ingress of chlorides, the basic electrochemical corrosion process can start; this has been investigated by many workers.

Due to small variations in the condition of the steel along the reinforcing bar and variations in the adjacent concrete, the pore fluid of which provides the electrolyte, the equilibrium conditions along the bar are not uniform and anodic and cathodic areas are allowed to develop. The corrosion cells are set up between these anodes and cathodes. The anodic reaction is the oxidation of iron where iron is dissolved away with electrons moving through the steel to the cathode. The complementary cathodic reaction is the reduction of oxygen which is the addition of electrons, arriving from the anode, to form hydroxyl ions. These hydroxyl ions move through the pore fluid to the region of the anode where they combine with the ferrous ions, released into the electrolyte at the anode, to form ferrous hydroxide, which in the presence of oxygen, will rapidly form rust (ferric oxide).
## Table 14
Key references for cover, cracking and durability (sheet 1 of 4)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beeby AW</td>
<td>A review of cracking and crack control to provide design assistance for offshore concrete structures. Methods of crack prediction were reviewed and data from long term exposure tests accumulated.</td>
<td>The main conclusions are: . crack widths may significantly affect corrosion in the short term but have insignificant effect in the long term . checks on crack widths serve no useful purpose during design.</td>
<td>(1) Conclusions reached suggest that no crack width requirements should be included in the Guidance Notes. (2) Further research on chloride penetration and longitudinal cracks must be considered. (3) Reference should be read in conjunction with Wilkins et al 1987 for underwater concrete. (4) Data used relates primarily to splash and atmospheric zones.</td>
</tr>
<tr>
<td>Fidjestol P et al</td>
<td>To identify possible problem areas on offshore concrete structures for corrosion of reinforcement. Discussion based on review of present day design procedures.</td>
<td>Major corrosion area is splash zone and with significant corrosion due to hollow leg effect. The pH value of water in storage tank must also be considered.</td>
<td>A general discussion extending the theoretical work of other Concrete in the Oceans projects.</td>
</tr>
</tbody>
</table>

Concrete in the Oceans Technical Report No 1 1978

Concrete in the Oceans OTH 87 237 1987
Table 14  
Key references for cover, cracking and durability (sheet 2 of 4)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Stillwell JA  
Exposure tests on reinforced concrete in seawater.  
Concrete in the Oceans Technical Report No. 23 1987 | To carry out exposure tests, at differing locations, to determine the rate of chloride penetration into concrete and rate of reinforcement corrosion. | The main conclusions are:  
- for submerged concrete significant corrosion of reinforcement will not occur  
- where cracks occur in the splash zone, corrosion is almost certain  
- concrete strengths should be 10 N/sq.mm above design requirements  
- cover may not be critical for determining corrosion rate. | Conclusions drawn differ in some marked respects from those drawn by Beeby (1978) and Wilkins et al (1987). |
| Wilkins NJM et al  
Fundamental mechanisms of corrosion of steel reinforcement in concrete immersed in sea water.  
Concrete in the Oceans Phase 1 Final Report 1980 | Fundamental research to investigate electrochemically, the corrosion mechanism of reinforcement in concrete immersed in sea water. | Steel embedded in sound concrete showed passive behaviour and concrete quality had little effect on corrosion. Fine cracks did not cause significant corrosion but steel exposed in slots corroded as bare steel. | (1) Provides data (and explanation) of the electrochemical corrosion of reinforced concrete.  
(2) Some of the conclusions drawn conflict with other Concrete in the Oceans reports.  
(3) Underwater corrosion only. |
### Table 14

**Key references for cover, cracking and durability (sheet 3 of 4)**

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Wilkins NJM et al.  
Fundamental mechanisms of corrosion of steel reinforcement of concrete immersed in sea water: results from Phase II.  
Concrete in the Oceans OTH 87 238  
1987                                                                          | As a contribution to definition of crack and corrosion control, experiments were carried out to investigate:  
- factors determining the rate of cathodic reactions on steel in concrete  
- effect of crack widths on steel passivity  
- effect on steel corrosion of concrete age and composition. | General conclusions were:  
- cracks do not lead to localised corrosion without external polarisation. Wide cracks with polarisation will lead to severe and continuing corrosion  
- there is a possibility of severe uniform corrosion in the negative active region  
- sacrificial corrosion of reinforcement will not necessarily be suppressed by normal cathodic protection. | (1) The report continues the research undertaken in the Phase 1 programme.  
(2) Overall (very generalised) conclusion may be that presently accepted "good practice" will lead to a sound durable structure.  
(3) Underwater corrosion only. |
| Taylor Woodrow  
Marine durability of the Tongue Sands Tower.  
Concrete in the Oceans Phase 1 Technical Report No 5  
1980                                                                      | Survey of an existing structure, 34 years old, to determine performance of reinforced concrete. | Concrete and reinforcement in the tidal and underwater zones were in good condition.  
Significant corrosion of the reinforcement was restricted to the splash zone. | |


<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fidjestol P et al</td>
<td>Review cover and crack width requirements for the submerged zone and put forward design recommendations.</td>
<td>(1) Cover should be that required for structural requirements. (2) Crack width requirements should be relaxed and that 0.6 mm should be regarded as an acceptable characteristic crack width. (3) Crack width calculations should be using a modified CEB-FIP formula with allowance for transverse cracks.</td>
<td>(1) Conclusions are for the submerged zone only. (2) Conclusions on methods of crack width calculations differ from Belsey (1978).</td>
</tr>
</tbody>
</table>
This process is indicated, in idealized form, in Figure 2, as the reduction of oxygen; this reaction is one of several that can occur but is, perhaps, the only one of significance for normal concrete structures.

This basic corrosion process has been studied by three alternative approaches:

- **Taylor Woodrow (1987)** have undertaken surveys of existing structures to investigate the actual deterioration that has occurred and, as a separate exercise, Stillwell (1987) has monitored the exposure effects on concrete specimens. Both of these prototype studies considered all exposure zones of a structure.

- **Beeby (1983)** carried out a desk study, based on published data for splash and atmospheric zones, to consider the effect of cracking and cover on corrosion. As a final project in the Concrete-in-the-Oceans programme, Fidjestol et al (1987) also undertook a desk study into cover and cracking parameters, based on other work on fully submerged concrete undertaken within the overall programme.

- **Wilkins et al (1980 and 1987)** have carried out controlled laboratory experiments to investigate the fundamental mechanisms of the corrosion process for underwater concrete. In addition, Taylor Woodrow (1987) have carried out a laboratory investigation into various parameters determining the effectiveness of cover.
The amount and, to some extent, nature of the corrosion processes varies according to the exposure zone being considered. For this reason, investigations carried out for any one particular zone should be used with caution in other zones.

Little, if any, work has been undertaken in the Concrete-in-the-Oceans programme on the materials aspects of concrete and their effect on durability. Several workers have used specimens of differing concrete qualities, as measured by cement content and/or crushing strength, in tests on overall corrosion of reinforced concrete, but the results are not conclusive and the range of properties tested was small. The materials aspects of concrete are discussed more fully in Section 3.2.

The normal basic requirements of concrete specifications are a minimum cement content and a maximum water/cement ratio, a characteristic strength, and appropriate cement type, all coupled with the use of sound aggregates and clean water. However, these measures can only create a certain potential durability and the extent to which this potential is realised or impaired, in the finished work, is dependent upon various other factors such as mix design, production control, handling and placing, compaction, curing, design, workmanship and supervision generally. There is evidence, Taylor Woodrow (1987), that the most significant factor is curing, although the mechanisms whereby this increases the potential durability are not fully understood.

From their surveys Taylor Woodrow concluded that the 34 year old Tongue Sands Tower was in 'remarkably good condition' and noted that 'deterioration of concrete and significant corrosion of the embedded reinforcing steel was restricted to areas of the splash zone in which either the specified cover (32mm) had not been met or where the localized use of an inferior quality mortar mix had resulted in both rapid ingress of oxygen and chlorides and carbonation of the concrete cover'. This evidence, it could be argued, demonstrates that good practice will lead to sound durable structures with the required economic design life. However, this view must be tempered with the understanding that present day practice differs from that of 30 or 40 years ago and, hence, evidence of 30 year old structures may not be fully representative of presently constructed structures in 30 years time. In addition, modern cements are different from those which were being used 30 years ago.

Over a period of 5 years (covering both Phases 1 and 2 of the Concrete-in-the-Oceans programme) Stillwell (1987) undertook a series of exposure tests, at differing locations, on prepared specimens to determine the rate of chloride penetration into concrete and the corresponding corrosion. These tests have demonstrated the differences between the submerged and splash zones but, again, have drawn generalised conclusions that in the submerged zone there is 'no evidence to suggest that significant corrosion of reinforcement will occur'. For the splash zone, however, Stillwell identified corrosion at cracks and suggested that cracking rather than cover and concrete grade is the parameter which will most affect corrosion.

Beeby (1978 and 1983) published the results of studies into cracking and its effect on corrosion. His conclusion, that crack width has an insignificant effect on long term corrosion, would appear to be contradictory to those of Stillwell. However, whilst Beeby was considering long term corrosion (say longer than 10 years), Stillwell's conclusions were based on a maximum of 5 years exposure; these conclusions are perhaps, therefore, in agreement that short term corrosion is significantly influenced by crack widths but long term corrosion is not.
Both Taylor Woodrow (1987) and Stillwell (1987) have produced evidence that chloride penetration can create concentrations of greater than 0.50% by weight of cement at a depth of 50mm in sound concrete in less than five years. Measurements at the Tongue Sands Fort indicate that, in the long term, chloride concentrations of greater than 0.50% by weight of cement will penetrate to a greater depth than any reasonably specified cover.

As part of their investigations into the efficiency of cover, Taylor Woodrow (1987) have measured chloride penetrations over a period of two years and then, using a parabolic rate law, extrapolated these figures to predict the ingress that may occur over a 30 year life. These measurements and extrapolations indicate the reduction in chloride ingress in higher strength and well cured concretes. Also, they throw doubt on the commonly held view that chloride ingress can be modelled by means of a constant diffusion coefficient. However, the findings and conclusions are of limited value to real structures since they relate to uncracked relatively high grade concrete specimens.

From the various aspects of research it is apparent that the rate of transport, by diffusion or other means, initially, chlorides then oxygen is more important than the permeability of the concrete. Curing of the concrete forming the cover layer, as demonstrated by Browne (1986), is very important in reducing the diffusion coefficient.

Whilst not explicitly stated in his report, Beeby based his conclusions primarily on data relevant to the splash and atmospheric zones. Fidjestol et al (1987) have undertaken a desk study of conditions in the submerged zone and, in some respects, have produced similar conclusions in that crack width requirements should be relaxed from those presently used. Also, the report perhaps somewhat dramatically, suggests that only the cover required for structural reasons (eg bond) should be provided and that this would be adequate for durability. Whilst this would not significantly affect the amount of cover actually provided on structures, of typically 1.5 times the bar diameter, it does change the basic underlying thinking on the need for cover. But there is a difference in thinking between Beeby and Fidjestol et al concerning the appropriate methods to be used in calculating crack widths.

For the submerged zone only, Wilkins et al (1980 and 1987) have investigated the fundamental mechanism of corrosion using laboratory specimens maintained submerged in test tanks. These investigations consisted of electrochemical research and the reports present the findings as such. They tend, therefore, to be the most difficult for practising engineers to assimilate but contain nevertheless some explanations of the corrosion phenomena and have resulted in much debated conclusions. As previously described, corrosion of reinforced concrete is an electrolytic process. The tests measured potentials and currents in the specimens under varying conditions both naturally occurring and artificially induced by external polarization. On completion of the tests, the specimens were broken open so that the physical corrosion could be quantified and conclusions drawn on the electrochemical behaviour.

The extent of any corrosion process is influenced by the relative areas of reinforcing steel and concrete. The tests carried out by Wilkins et al (1980 and 1987) were on specimens with a significantly lower percentage of reinforcement than would occur in a full size structure; hence, the corrosion seen in these specimens may not occur in practice to the same extent.
Corrosion of a metal can occur if its potential is displaced above its equilibrium potential. The current-potential relationship of the two basic reactions (anodic and cathodic) can be represented graphically as shown by the solid lines on Figure 3. The current associated with metal dissolution (anodic reaction) theoretically must equal the current associated with oxygen reduction (cathodic reaction) as the driving force for the reaction to take place. The corrosion potential ($E_{corr}$) is defined at the intersection of the two current-potential lines.

![Graph showing corrosion potential](image)

**Figure 3**
Electrochemical system of corrosion cell

This corrosion potential must lie on the anodic polarisation curve, also indicated by a solid line on Figure 3, which defines the behaviour of the system under consideration. Two distinct regions can be identified on the anodic polarisation curve - the passive and active regions.

The passive state is associated with low current densities, less negative potentials and the formation of a protective film of corrosion products on the steel surface; in this condition, there is a negligible corrosion rate after the protective film becomes established. The active state is associated with more negative potentials and the protective surface film on the reinforcement is broken down; in this condition there may be significant corrosion. When the oxygen supply is limited, for example in permanently wet concrete, corrosion can continue, as shown by the dotted lines on Figure 3 but at a very reduced level and at a very negative potential; the region in which this phenomena occurs has been called 'negative active'.
The equilibrium potential of reinforcing steel in the active region will typically be -800mV or more negative whereas that for reinforcement in the passive region will be less negative than -550mV, and that for steel exposed to seawater may be typically -650mV. Therefore, in the passive state the embedded steel is the cathode to the anode at the exposed steel but in the active (or negative active) conditions the exposed steel becomes the cathode to the anode at the embedded steel and therefore, in the active or negative regions, embedded steel will corrode sacrificially to exposed steel electrically connected to it.

All of the specimens tested by Wilkins et al. were submerged but some were pressurised to represent depths of greater than 300 metres. These deep specimens were shifted into the negative active region but some readily reactivated when brought to the surface (depressurised). Most of the specimens submerged at the surface (i.e. not pressurised) remained in the passive region but some were shifted into the active. The reasons why some were shifted one way and some the other are not, however, understood.

From the Phase 1 experiments, Wilkins et al. (1980) concluded that the ‘overall corrosion rate of steel in fully immersed concrete is likely to be very low but the distribution of corrosion and therefore the corrosion damage depends on the relative anodic and cathodic areas’. Tests with cracked specimens, over the two year period of the tests, indicated that narrow cracks had little effect on corrosion. The definition adopted of very low corrosion rate was 1mm per 100 years but this represents, perhaps, a corrosion scientist’s view rather than an engineer’s view.

The Phase 2 tests tended to confirm the conclusions drawn at the end of Phase 1. The corrosion potentials of reinforcing steel in sound or finely cracked submerged concrete were identified as being either in the passive or negative active regions. Corrosion rates in the passive region are negligible. However, the possibility of ‘uniform corrosion in the negative active region up to several hundredths of a millimetre per year’ was hypothesised and ‘cannot be excluded’. This last conclusion was much debated in the Concrete-in-the-Oceans Steering Group although it was finally accepted that actual corrosion rates for typical offshore structures are likely to be negligible.

Wilkins et al. (1987) also concluded that corrosion of reinforcement in the negative active region may be accelerated by electrical coupling to reinforcement directly exposed to seawater. Normal cathodic protection systems, with operating potentials between -800mV and -900mV Ag/AgCl, may not suppress this action at the least negative end of the range although in the -850mV to -900mV range the action will be suppressed.

Where corrosion occurs at cracks in the submerged zone, some of the corrosion product migrates out through the cracks. There is not therefore the same tendency for a build up of expansive corrosion products at the reinforcement surface which occurs in structures exposed to air. This has consequences for the inspection of underwater sections of structures as serious corrosion could be occurring without associated cracking and spalling. The only visual evidence of corrosion would be surface rust staining and deposits. In situations where the rate of seawater flow is high, the corrosion products could be swept away and oxidised remote from the concrete, rather than being deposited near the crack. In these circumstances there would be little visual evidence that corrosion was occurring.
Carney et al (1987) investigated the conditions of crack geometry, electrochemical potential and seawater flow under which there might be no external visual evidence of corrosion. Slabs with different longitudinal and transverse crack widths were exposed in a channel through which seawater was pumped at controlled velocities. Some specimens were left unpolished and the remainder were polished under galvanostatic control at currents estimated to give potentials as near as possible to -200mV and -400mV against silver/silver chloride half cells. Carney et al found that for linear flow rates of 0.25 metres per second, or less, corrosion was readily detected as rust staining and deposits of corrosion product were visible at the mouth of the crack. At higher seawater flow rates there was no such clear visual evidence of corrosion. They concluded that visual inspection of underwater concrete surfaces is not a reliable method of confirming the absence of corrosion in underwater structures and that the only reliable criterion for dismissing localised corrosion at cracks is an overall underwater potential of -700mV Ag/AgCl or more negative. This potential criterion should be met if the reinforcement is cathodically protected.

All of the above work has been carried out in relation to static cracks; where laboratory specimens have been prepared, they have had artificially induced cracks held open at the required width for the duration of the tests. In practice, the majority of cracks on an offshore structure will be dynamic cracks opening and closing elastically as the load is applied and reduced. Hodgkiess et al (1987) have investigated the effects of dynamic cracks as part of an investigation into fatigue of reinforced concrete and shown substantially increased corrosion. They concluded that in comparison with static loads, the effect of dynamic loading can seemingly allow corrosion to be initiated and certainly allow corrosion to continue (once initiated) at lower crack openings than under static conditions.

The experiments set up by Hodgkiess et al (1987) were to investigate fatigue and the corrosion implications were only realised towards the end of the test programme. Reverse bending tests were carried out on reinforced concrete beams with the mid-span section submerged and the ends exposed such that all cracks were fully submerged and uncracked sections exposed; this test arrangement led to increased corrosion with cathodic areas being established at the exposed ends of the beams to anodic areas in the submerged mid-span sections. Initially a high rate of corrosion was experienced but this decreased with increasing number of load cycles and time. All cracks, formed by the loading, became blocked leading to stiffening of the beam and increased fatigue resistance and, presumably, resulted in a reduced corrosion rate.

Beeby, in a report for the Department of Energy, has reviewed results from the Hodgkiess et al research. He has drawn the conclusion that, under certain circumstances, serious local corrosion can occur at cracks in the submerged zone when subjected to cyclic opening and closing. He has also concluded that there may be a difference in corrosion behaviour depending on the maximum reinforcement stress. In the Hodgkiess et al tests, at a lower stress range, (216N/mm²) the extent of corrosion was limited to positions close to cracks but there were losses of reinforcement section of 27% and 46% in the two beams examined. At higher stress ranges (290 and 363 N/mm²) corrosion covered a much larger area, again close to cracks, but was of insignificant depth. The difference in corroded length at cracks is attributed, in the Hodgkiess et al report, to the debonding, between concrete and reinforcement, which occurs close to cracks at high stress values thus exposing more steel to attack. Beeby notes that it would be reasonable to conclude, from the Hodgkiess et al results, that the lower the stress the smaller the cracks, the more localised is corrosion. Also he
concluded that, the more localised the corrosion is, the larger is the corrosion depth and also that there would not therefore appear to be any logical reason, on the current evidence, for specifying limits on crack widths under dynamic loading as this could result in more intense localised corrosion.

Whilst the Hodgkiess et al tests indicate the possible importance of dynamic cracking under cyclic loads, they do not allow design rules to be formulated. Some of the phenomena are time related, such as the crack blocking and corrosion process, whilst others are cycle dependent and further testing is, therefore, required to understand these mechanisms. These conclusions, however, are counter balanced by findings reported by Nilsen et al (1985) which tend to indicate that dynamic cracking on fully submerged members has no effect on the corrosion behaviour. There is an anomaly between the results of Hodgkiess et al and Nilsen et al in that both sets of specimens reached the same potentials but where the former showed significant corrosion damage the latter indicated none.

The research that has been undertaken supports the experienced view that corrosion mechanisms and rates are different in the various zones of a structure. However the proportions of the structure in each zone are important. Fidjestol et al (1987) have considered this ratio and put forward definitions for 'permanently wet' and 'interactive zones' for a typical deep water concrete platform. Also, the extent of corrosion can be influenced, within the submerged zone, by the hollow leg effect. Nevertheless it is appropriate to consider separate parameters for these zones, the atmospheric, splash, tidal and submerged zones:

- **The submerged zone** - The general body of experience and research evidence would suggest that corrosion in this zone is limited and that the presently accepted parameters of cover and crack width can be significantly relaxed. Fidjestol et al (1987) have concluded that "there does not appear to be any clear influence of cover quality and depth as far as corrosion protection of reinforcement is concerned" and also perhaps that crack widths, for submerged concrete, may have little effect on corrosion.

Other researchers have also concluded that there is no benefit to be obtained from checking for and limiting crack widths but others have found, certainly in the short term, corrosion being initiated and accelerated at cracks. Until these phenomena can be fully explained it does not seem appropriate to abandon crack width parameters. Various authors (e.g. Wilkins et al 1980) have concluded that concrete mix has little effect on corrosion and others (e.g. Stillwell 1987) have questioned the significance of cover. Again these conflicts do not allow any substantial change from present good practice with any confidence and particularly when Wilkins et al (1987) conclusions are considered.

Underwater, corrosion products can be (or are) washed away by water action preventing build up and spalling. In fact, research by Carney et al (1987) upon visible signs of corrosion suggests that significant reinforcement corrosion can take place without any external evidence.

The possibility of corrosion occurring at dynamic cracks, particularly in the upper region of the submerged zone, cannot be ignored based on the evidence available to date.
• **The tidal and splash zones** - These should perhaps be considered together. Evidence from the various research projects and from experience suggests that this is the most vulnerable area of a structure and over which corrosion is most likely to become apparent.

Corrosion of steel in submerged concrete is suppressed by oxygen limitations caused by the low diffusion rate through the saturated concrete. The diffusion rate of dry concrete is greater and there is an abundant oxygen supply at the concrete boundary allowing for a greater activity of the corrosion reaction in the splash zone.

Taylor Woodrow (1987) and Stillwell (1987) have demonstrated that chloride levels will, in a relatively short period, be sufficient to initiate corrosion and that the presence of cracks will speed up the ingress of chlorides. Corrosion becomes apparent and is rapidly accelerated when spalling occurs. However greater cover will require larger bursting forces, from corrosion products, before spalling occurs.

For these reasons it is concluded that no substantial change is appropriate to the basic design parameters of cover, crack width and concrete quality. Present practice is seen to provide reasonably durable structures except that, the possible consequences of dynamic cracking should be recognised.

• **The atmospheric zone** - This is that area of the structure above the splash zone and the zone for which there has been least recent specific research. Therefore, as with the other areas, there appears to be no rationale for changing the parameters.

### 3.1.3 Impact

Damage due to impact may occur either from objects dropping off the working areas of a platform and impacting the lower structure before coming to rest on the sea bed, or alternatively from a ship colliding with the structure. These aspects are considered separately.

The treatment of accidental loads, such as impact, requires special consideration with respect to Limit State Design to ensure that an appropriate level of reliability is retained. Accidental loads differ from other loads in that, in a perfectly ordered system, they would not occur whereas other loads definitely will occur; but, nevertheless, recognition must be given to the fact that they can and do occur. DnV (1981) have considered an appropriate design philosophy involving the concept of a Limit State of Progressive Collapse, where two levels of loading are envisaged corresponding to a 'no damage' working stress criteria and a 'local failure' ultimate stress criteria. Such a philosophy implies the happening of a large event which may cause the total collapse or failure of the structure.

**Dropped Objects** - A variety of shapes and sizes of objects fall by accident into the sea. These may cause damage to pipelines resting on the seabed, or to caisson roofs. Consideration should be given to protection against a range of objects and also to the mode of failure which would influence the repair to be undertaken. No attention appears to have been given to quantifying these problems until quite recently, (Dowrick
## Table 15

**Key references for impact due to dropped objects**

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wimpey Laboratories Ltd</td>
<td>Compile data on caisson configuration including reinforcement and prestress details.</td>
<td>There is a 'real' risk of damage to caisson roofs.</td>
<td>Several recommendations for further empirical work.</td>
</tr>
<tr>
<td>DnV Design against accidental loads. DnV Technical Note TNA 101 1981</td>
<td>Recommendations for design for accidental loads.</td>
<td>Existing offshore codes are inadequate. Procedures recommended for several individual cases.</td>
<td>Useful conceptual framework for this type of loading.</td>
</tr>
<tr>
<td>Taylor Woodrow The development of methods for structural reinstatement of concrete structures. Taylor Woodrow Research Laboratories March and April 1983</td>
<td>To investigate methods of repairing damaged concrete and testing representative scale models.</td>
<td>Successful repairs can be made to severely damaged localised sections.</td>
<td>Includes punching shear tests carried out on model sections.</td>
</tr>
</tbody>
</table>
1979). Subsequently, there have been several research projects undertaken and, as part of the process of revising Guidance, these have been identified and studied. Key references are summarised in Table 15 in addition to a full bibliography in the database listing as an Appendix to this report.

Analysis of the problem requires consideration of several component aspects. These include establishing a 'design' object, from a range of objects which may be dropped from a platform, calculating the impact velocity with due allowance for free fall through air and impact with and fall through water, and analysis of the concrete walls or slab for impact damage. Damage may take the form of spalling of the outer face, scabbing of the inner face or perforation of the object through the concrete member.

Wimpey (1987) undertook a study to review all these aspects and derive appropriate design formulae. Details of caissons and of actual dropped objects were compiled and an assessment of impact damage was made. However, the data on experimental work was considered to be insufficient. Recommendations for design formulae were, therefore, tentative with additional work being considered essential before adequate understanding of the problem could be obtained. This would involve further investigation corresponding more closely to actual conditions; for example, dynamic response of caisson roofs, variations of hydrostatic pressure and validity of dimensional analysis. The conclusions reached on completion of the study were:

- Data on impact velocities for a variety of slender and bulky objects both broadside and end-on was compiled.

- There was insufficient empirical evidence to support the design methods considered; that is, empirical formula and dimensional analysis.

- Order of magnitude calculations showed that for a slender object at a maximum design impact velocity of 25 m/sec and reinforcement say 0.5 to 1.5% the slab thickness required to prevent scabbing was 750mm and to prevent perforation 400mm. This excluded several mitigating effects which have not been quantified.

- Non-linear finite element analysis showed that damage to a typical caisson roof of say 500mm by a bulky object would only occur under particularly unfavourable circumstances.

Two categories of objects were identified: slender and bulky objects. Information on empirical testing of slender objects was considered limited. This was based on the results of military ballistics (Kennedy 1975) and dimensional analysis (Davies et al 1979). Only three cases of observed damage to caisson roofs were available. For bulky objects, a study by Jensen (1979), on the impact of falling loads on submerged concrete structures was considered, but the validity of several assumptions made in this study were found questionable. The findings made in this category were, therefore, based on a non-linear finite element analysis which concluded that damage to caisson roofs from bulky objects would only occur under particularly unfavourable combinations of circumstances.

Figures 4 and 5 indicate experimental data for impact damage for a selection of slender and bulky objects respectively. These were produced from design parameters based upon several publications (NDRC 1946, Berriaud 1978 and Davies et al 1979) for a 280mm OD drill collar and a 915mm drain caisson. The data on the drill collar shows
Figure 4
Assessment of possible damage by a Christensen 280mm OD drill collar
(after Wimpey 1987)
Figure 5
Assessment of possible damage by a 915mm OD 22mm WT drain caisson (after Wimpey 1987)
the relative differences of the alternative methods available. This data is based on several empirical formulae and dimensional analysis derived from tests on ballistics and models. No account has been taken of response of the structure, effect of water or shape of missile. These figures give an empirical assessment of the slab thickness required to prevent scabbing or perforation. It should be noted that the NDRC formulae are considered conservative with respect to damage observed in reported incidents. This may be attributed to the lack of suitable data at an appropriate velocity range. The above data should be regarded as provisional until a more satisfactory approach has been developed following additional experimental work at velocities below 30m/sec.

For bulky objects a study by Jensen (1979) was reviewed by Wimpey (1987) but several assumptions were found questionable. Wimpey (1987) therefore used a static non-linear finite element analysis to model the initial stages of impact and a simple yield line model was used to predict the extent of yield. The analysis covered a generator (10.0 tonne), mud pump (31.0 tonne) and blow out preventor (20.6 tonne) which suggested that damage to caisson roofs from bulky objects was unlikely.

The Wimpey (1987) work was supplemented by Brown et al (1987) whose empirical work on prestressed slabs, rigid blocks and reinforced domes provided some basic information on the effect of fender layers for protection against impact, as well as water saturation and impactor shape. New empirical formulae were proposed from a simple mechanical model using springs. These formulae are similar to those proposed by Wimpey (1987) and use the same data and are therefore subject to the similar limitations.

Apart from this, further conceptual refinement of the mechanical model is required before its full potential for predicting accurate impact behaviour is realised. On the other hand the formulae do give some indication of the likely penetration, scabbing or shear plug formation.

In conclusion, Wimpey (1987) and Brown et al (1987) have drawn together all the relevant documentation and therefore represent the latest available information on the subject. It may be generally concluded that a significant basis of design for impact damage has been provided. A selection of objects likely to cause damage has been analysed and impact velocities determined and these are summarized in Figures 6 and 7. Formulae for determining damage due to impact have been derived but several limitations are noted: dynamic response of caisson roofs, effect of hydrostatic pressure and uncertainty regarding experimental data.

**Ship Collision** - Several incidents of collision of supply vessels with offshore structures have been experienced and therefore such circumstances require detailed consideration by the designer. Consideration should include a major collision by a large ship causing overall collapse of the platform and a minor collision by a supply boat causing minimal local damage. The subject comprises several sub-categories which require attention as noted by Dowrick (1979) in his appraisal of the problem. The probability or risk of collision varies with the type of vessel and should be quantified. The impact force for a given design loading must be calculated and the resistance of the structure to both local failure and overall stability must be determined.

Several references have been identified and assessed for this subject. Key references are summarised in Table 16.
Figure 6
Velocity of slender objects falling from 35m (after Wimpey 1987)
Figure 7
Velocities of falling bulky objects (after Wimpey 1987)
**Table 16**

**Key references for ship impact**

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Davies et al Assessment of the damage from collision between ships and offshore structures. Integrity of Offshore Structures, Glasgow Symposium July 1981</td>
<td>Method of realistic appraisal of consequence of accident.</td>
<td>Results provided for variation interaction force for work boat with respect to beam-on stern-to collision.</td>
<td>Calculation of damage based on impulse - momentum principles.</td>
</tr>
<tr>
<td>DnV Design against accidental loads. DnV Technical Note TNA 101 October 1981</td>
<td>Recommendations for design for accidental loads.</td>
<td>Existing offshore codes are inadequate. Procedures recommended for several individual cases.</td>
<td>Useful conceptual framework for this type of loading.</td>
</tr>
</tbody>
</table>
The risk of collisions is real and may occur at different stages in the life of the structure, for example during construction, towing to the site, or from supply boats, fire boats, tankers and commercial shipping at the final location. However, ship impacts with a low probability of occurrence may be very high in magnitude and consequently unreasonable to design against. A level of 0.5% of the maximum ship momentum is considered as a reasonable design load when assessed in conjunction with risk (Dowrick 1979).

The forces between the ship and concrete structure must be evaluated to determine damage. If the duration of the collision lies between 0.4 and 2.0 seconds then contact forces between 5MN and 25MN may result using conventional principles of momentum (Dowrick 1979). However, the collision force is essentially a function of the deformation characteristics of the vessel, which is a function of the area over which impact occurs. This, naturally, varies over the duration of the impact.

The mode of local failure of the structure, on the basis of tests on cylinders, appears to be due to punching shear, although cracking occurs at lower loads and should be considered for serviceability requirements (Dowrick 1979).

Wimpey (1987) undertook a comprehensive study on the impact of offshore concrete structures, by including an assessment of the probability of collision, the resulting forces and the local failure loads on the structure, the calculation of global failure loads and a review of the structural integrity of the damaged platform.

They considered that an inelastic impact analysis, necessary for the investigation of local bending failure, was not possible and the study of failure mechanisms was therefore limited to punching shear failure only. The available tests did not however include the prestressed case and did not measure membrane stresses.

They concluded that a testing programme of the punching shear strength of cylindrical shell elements was required which would include a study of impact effects and would be supported by three-dimensional computer analysis. Other major areas in which further work was needed was the load/deformation characteristics of vessels, inelastic analysis to determine local bending failure of prestressed towers and finally the inelastic analysis of damaged concrete cylinders, to investigate conditions under axial loading and applied bending moments.

Various conclusions were drawn from the study. These were:

- The probability of a ship collision with a prestressed concrete tower would be 0.02 to 0.03 per installation per year for a supply vessel and 0.002 for a tanker engaged in offshore loading.

- Concrete platforms would withstand minor collision from small vessels (eg fishing boats) without damage. Sideways collision of an offshore supply vessel during normal operating conditions may cause some damage and larger ships, such as drifting offshore loading tankers, would cause major damage.

- Significant local damage was possible before global collapse of a tower occurred.

Two methods of determining impact forces have been proposed from the various studies done on the subject, including Wimpey (1987). These are the energy method and the impulse - momentum method.
The energy method is based on the principles of kinetic energy and strain energy. Absorption energy obtained from curves representing load deformation characteristics for the ship and platform are equated to the kinetic energy of the vessel, hence the collision characteristics may be established (Carlin et al 1977, Furnes et al 1979, Haywood 1978). The difficulty is in deriving the load deformation curves. Procedures for deriving these curves, using an elasto-plastic model and comparing with ship model tests representing a variety of hull strengths and using simplifying assumptions, are described by Furnes et al (1979) for the case of a sideways collision with a fixed offshore structure and Haywood (1978) for a range of vessels assuming a bow collision with infinitely rigid barriers.

The impulse - momentum method (Davies et al 1981) is based on the equality of momentum and impulse and the solution to the equation of motion. The determination of the forcing function assuming an infinitely rigid platform structure is described with a refinement comprising 2 degrees of freedom. It is claimed that this approach is somewhat more accurate than the kinetic energy/strain energy method. This method may be used to define the time related variation of the interaction force between ship and structure and to indicate the way in which both the ship and structure distort as the collision develops. More accurate predictions of damage (e.g. cracking of concrete and yielding of reinforcement) can be made using a finite element method based on dynamic relaxation whilst assessments of global effects may also be made using suitable finite element models.

Tables 17, 18 and 19 provide a comparison of collision characteristics based on the two methods.

<table>
<thead>
<tr>
<th></th>
<th>Supply vessel</th>
<th>Small cargo ship</th>
<th>Channel ferry</th>
<th>Large cargo ship</th>
<th>Oil tanker</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement (tonnes)</td>
<td>875</td>
<td>2500</td>
<td>8900</td>
<td>21300</td>
<td>60000</td>
</tr>
<tr>
<td>Ship Distortion (m)</td>
<td>2.0</td>
<td>large</td>
<td>2.0</td>
<td>2.0</td>
<td>0.31</td>
</tr>
<tr>
<td>Damage Horizontal Dimension (m)</td>
<td>3.17</td>
<td>5.90</td>
<td>2.90</td>
<td>4.28</td>
<td>2.15</td>
</tr>
<tr>
<td>Impact Velocity (m/s)</td>
<td>3.03</td>
<td>1.50</td>
<td>1.15</td>
<td>2.05</td>
<td>0.77</td>
</tr>
<tr>
<td>F.O.S. (failure)</td>
<td>4.15</td>
<td>1.55</td>
<td>4.10</td>
<td>1.78</td>
<td>1.00</td>
</tr>
<tr>
<td>Remarks</td>
<td>No Failure</td>
<td>No Failure</td>
<td>No Failure</td>
<td>No Failure</td>
<td>Punching Shear Failure</td>
</tr>
</tbody>
</table>

Table 17
Summary of ship collision characteristics based on energy methods
(after Caldwell et al 1981)
Taylor Woodrow (1983) studied the development of methods for structural reinstatement of concrete structures, including the testing of semi-cylindrical concrete models to determine the failure mode due to localised loading. They concluded that initial failure took the form of a typical shear cone and led to large diagonal cracks through the thickness of the concrete. Subsequent to this the reinforcement and prestressing tendons were capable of carrying considerable load before themselves suffering ultimate failure. The severely damaged sections were then repaired before being retested in the same manner. The repaired section was found to be capable of resisting considerable load before suffering failure again.

The study showed that a typical reinforced concrete section as found on an offshore structure had considerable punching shear strength similar to the findings of Dowrick (1979). However the tests were principally concerned with the repair of severely damaged sections and as such the information contained in the report with regard to shear punching was not sufficient to enable the conclusions arrived at by Wimpey and others to be refined.

Table 18
Summary of ship collision characteristics based on impulse - momentum method (after Davies et al 1981)

<table>
<thead>
<tr>
<th>Collision Speed (m/s)</th>
<th>Beam-on collision by work boat</th>
<th>2500t vessel with 900t added mass</th>
<th>All vessels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interaction Force (MN)</td>
<td>0.5</td>
<td>2.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Ship Distortion (mm)</td>
<td>22.5</td>
<td>65.0</td>
<td>97.0</td>
</tr>
<tr>
<td>Caisson Distortion (mm)</td>
<td>41</td>
<td>180</td>
<td>435</td>
</tr>
<tr>
<td>Remarks</td>
<td>Slight non significant cracking of caisson</td>
<td>Significant but not serious cracking of caisson</td>
<td>Considerable damage to caisson</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Further detailed studies were being carried out, at the time of writing this report, by Lloyd's and NMI with regard to ship impact on offshore structures, to allow a new section of the Guidance Notes to be finalised. These studies will not deal specifically with concrete structures but with all offshore structures in an attempt to derive overall design rules. There are, however, differences appropriate for ship impact on differing types of structure and perhaps the concrete sections of the Guidance Notes should contain some requirements specifically for concrete structures.

The following general conclusions may be considered appropriate for concrete structures but these must be reviewed when results are available from other ongoing research studies:

- Design for ship collision should be divided into several component categories; these include risk of collision, design loading, force of impact, impact capacity, limit state analysis for damaged structure.

- Information on damage and load capacity of concrete cylinders requires further improvement. In particular, punching shear tests on cylindrical shells and investigation of local bending failure are urgently required to provide an adequate basis for design. Further to this, the relative merits of two methods for calculating impact forces, that is, kinetic energy and impulse-momentum, require assessment.

### Table 19
**Collision forces from 2500 tonne supply vessel based on energy methods (after Furness et al 1979)**

<table>
<thead>
<tr>
<th>Proportion of energy absorbed by boat (%)</th>
<th>Max force developed (MN)</th>
<th>Contact chord (m)</th>
<th>Contact vert (m)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>5.3</td>
<td>0.75</td>
<td>0.9</td>
<td>Impact area corresponds to the deck zone at a transverse frame</td>
</tr>
<tr>
<td>100</td>
<td>6.4</td>
<td>0.95</td>
<td>0.9</td>
<td></td>
</tr>
</tbody>
</table>

**Case (a) 'Operating' collisions. 2500 tonne boat, 0.5m/sec, added mass 0.4, energy = 0.43 MN.m**

**Case (b) 'Accidental' collisions. 2500 tonne boat, 1.5m/sec, added mass 0.4, energy = 4.0 MN.m**

| 50                                       | 17.0                     | 5.4               | 5                | Impact area corresponds to the contact over full height of ship's side. |
| 100                                      | 20.0                     | 5.9               | 5                |         |
3.1.4 Implosion

Concrete offshore structures generally comprise cylindrical members which must withstand high hydrostatic pressures during both construction and operation. This section considers failure of this structural configuration by implosive buckling or implosion. Key references identified are given in Table 20.

As reported by Haynes (1979), initial research into implosion effects was begun in the mid-1960's by the Civil Engineering Laboratory, Pat Hueneme, California, USA. Initial research was based upon concrete spheres and tests on cylinders started about 1970. The early cylinder models had an outside diameter of 406mm. Parameters such as cylinder length, wall thickness, end closure conditions and concrete compressive strength were investigated. With the start of North Sea development and concrete structures being placed in greater water depths, a programme of testing was developed culminating in the formulation of design guides for implosion of concrete cylinders.

Chrapowicki et al (1987), reviewed current design methods and related these to the results of experimental work available. The study thus included:

- review of experimental work on long thin cylinders
- survey of shells used in offshore design
- survey and assessment of existing methods of analysis of prestressed and reinforced concrete shells
- study of effects of parametric variations in partial cylinders of slenderness numbers
- guidance on design

These objectives were generally achieved and no specific recommendations were made regarding further work to be undertaken. However, it was noted that both analytical tools and experimental data on partial cylinders was limited.

The format of the report by Chrapowicki et al (1987), which is essentially the documentation of the results of a survey, did not lend itself to the drawing up of final conclusions. However, recommendations have been made in respect of the various design methods reviewed. These were divided into two categories: semi-automatic or desk calculation methods, and computer programs. The methods which were compared are summarized in Table 21.

The comparison of analytical methods using semi-automatic or desk calculation methods suggested that the Haynes method (Haynes et al 1979) was preferable although partial cylinders and shell bracings were not covered. The Haynes method is based on three equations which describe the failure of concrete cylindrical structures under hydrostatic loading. The approach for thick wall cylinders is based on the average stress distribution across the wall of the cylinder at implosion. The wall stress at implosion may be considered as a function of the implosion pressure for a given diameter and wall
### Table 20

**Key references for implosion**

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chrapowicki KA et al</td>
<td>. Review of experimental work on long thin cylinders.</td>
<td>Recommends 'Haynes' method for desk calculations.</td>
<td>Study is based on cylindrical shells in their simplest form. Variations commented upon.</td>
</tr>
<tr>
<td></td>
<td>. Guidance to design.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Haynes HH</td>
<td>To present design guides for implosion based on test results from both thin and thick walled cylinders.</td>
<td>Analytically, the behaviour of the cylinder was predicted with gooc accuracy. Out-of-roundness was an important parameter reducing implosion strength.</td>
<td></td>
</tr>
<tr>
<td>Design for implosion of concrete cylinder structures under hydrostatic loading. Civil Engineering Lab (Navy), Port Hueneme CA</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1979</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 21
Comparison of desk and computer calculation methods for implosion
(after Lloyds 1987)

<table>
<thead>
<tr>
<th>Method</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Semi-automatic or deck calculation methods</strong></td>
<td></td>
</tr>
<tr>
<td>1. Tangent modulus</td>
<td>Easy to apply. Requires advance computation of $f$. May need calibration.</td>
</tr>
<tr>
<td>2. Adjusted equilibrium</td>
<td>More accurate than 'tangent method' but application complex. Also requires computing $f$.</td>
</tr>
<tr>
<td>3. Limit state</td>
<td>Results close to experimental values, requires computing $f$. Long term effects need refinement.</td>
</tr>
<tr>
<td>4. Secant modulus</td>
<td>Appears to be suitable for values of $f &gt; 500 \gamma_m$: that is, mainly outside range applicable to offshore.</td>
</tr>
<tr>
<td>5. Haynes et al</td>
<td>Many effects joined in 'plasticity factor'. Version 1979 shows good agreement with experimental data.</td>
</tr>
<tr>
<td><strong>Computer programs</strong></td>
<td></td>
</tr>
<tr>
<td>1. BOSOR 5</td>
<td>Suitable for complex or advanced analysis but further adaptation work needed.</td>
</tr>
<tr>
<td>2. BOSOR 4</td>
<td>Material non-linearity not considered. Useful in conjunction with semi-automatic methods for computing $f$.</td>
</tr>
<tr>
<td>3. STAGS - C</td>
<td>Suitable for complex or advanced analysis. Adaptation work needed.</td>
</tr>
<tr>
<td>4. BUSH</td>
<td>Non-linear effects not considered. May be used in conjunction with automatic methods.</td>
</tr>
<tr>
<td>5. CONSYM 1</td>
<td>Handles material non-linearity, creep, cracks, reinforcement. Limitations to axisymmetric loading and geometry.</td>
</tr>
</tbody>
</table>
thickness and can also be expressed as the ultimate compressive strength of concrete times an empirical factor. It therefore follows that the implosion pressure may be expressed as a function of wall thickness and diameter and, further, that this relationship may be determined empirically. The approach taken for thin wall cylinders considers separately, moderately long cylinders in which stability is influenced by end-closures which restrain the cylinder from collapse, and long cylinders which are unaffected by end-closures.

Elastic buckling stresses are determined by Donnell's equation and Bresses equation respectively. An empirical plasticity factor is included in each equation. This is calculated as the experimental stress at implosion divided by the calculated elastic stress at buckling and allows for the inelastic behaviour of concrete and out-of-roundness. An advantage of this approach is that the empirical factor may easily be modified to correspond to the most recent experimental data available.

The recommendation of the above method was supplemented with a full description of a graphical approach. This graph is directly applicable to both complete and partial cylinders in simple design situations. The method is based on the Haynes method for complete cylinders. Data for partial cylinders has been generated using BOSOR 4, a finite element computer program.

The computer programs which were reviewed offered a wide range of particular features such that any application would be dependent on the analysis required. These programs would be reserved for final design or complex details (e.g. branchings, multi-lateral composition, asymmetric loadings, temperature loadings, stage loadings, etc).

Consideration was also given to other aspects of the idealised case:

- **Imperfections.** Reduction in buckling strength is generally accepted as occurring for imperfections of the order R/200. This is supported by experimental work (Haynes 1979).

- **Reinforcement.** Reinforcement is normally provided to prevent minor cracking and is taken into account by most methods. The contribution of the reinforcement after yielding to the buckling strength is still a matter of debate (Zaleski - Zamenhof 1976).

- **Prestressing.** The effects of prestressing have been considered in some detail (Zaleski - Zamenhof 1976). The modulus of elasticity of concrete at implosion depends not only on the critical stress but on the sum of the critical stress and direct prestressing effects. This implies modification of the buckling formula for reinforced untensioned cylinders.

- **Long term phase loading.** Data illustrating in graphical form a relationship between the buckling load obtained as a result of phase loading and load duration is available (Zaleski - Zamenhof 1976). This also includes a correlation of concrete strains and load duration for various concrete strengths.

- **End closures.** In most cases a simple adjustment to buckling strength may be obtained by the use of suitable factors. Tables showing relative factors for different types of closure are available (Leick et al 1978, Haynes Sept 1976).
• **Saturation.** Useful data on saturation of submerged concrete and variation of compressive strength with age of concrete is available (Haynes Sept 1976).

• **Experimental results.** Results from several sources (Runge et al 1972, Albertson 1973) are provided in the Final Report P2A of the Concrete-in-the-Oceans programme.

• **Thick cylinders and spheres.** Comprehensive guidance based on extensive tests is provided in several publications. (Haynes et al 1976, Haynes 1979, Albertson 1973). The testing includes a domed cylindrical specimen approximately 6m long which was tested in the deep sea.

Sjursen et al (1979) describe the results of a test on a one fifth scale model of the Condeep Platform Stratford A. This comprises a load capacity test on a cell model exposed to hydrostatic pressure. Both reinforcement and prestressing were modified to correspond to the one fifth scale model adopted. Failure corresponding to a shear failure occurred at a net hydrostatic pressure of 164 metres. A finite element analysis was also completed and comparison made with actual strains occurring during the test. An acceptable correlation was obtained but details of the programme are not provided.

Finally, it may be concluded that the review described above, has adequately drawn together all the latest information on the subject. The following general conclusions may therefore be drawn:

• Guidance on design comprising charts for preliminary sizing, semi-automatic/desk calculation and computer programmes have been provided from the Concrete-in-the-Oceans programme. The Haynes method which is semi-empirical and based on laboratory tests may be recommended for general use. Complex configurations may be analysed using a variety of computer programs which have been developed for various specific applications.

• General design guidelines have been given on a variety of component aspects of design. These include, for example, an assessment of various non-linear effects such as geometry, long term effects, different combinations of loading, partial cylinders.

• Comprehensive empirical work is available in support of the Haynes method (Haynes 1979). Testing of partial cylinders is, however, outstanding and further work is needed in this area.

3.1.5 **Temperature effects**

Existing configurations for offshore concrete oil storage structures, including production platforms, are subject to significant temperature gradients. Oil flowing from a production well may typically be between 70° and 100°C whereas the sea water surrounding the outside of the structure is at a constant temperature of approximately 5°C. Differential temperature gradients are set up in the concrete walls of the storage vessel when the oil is pumped into or out of the storage cells displacing the sea water. The walls of the storage cells will, therefore, be subjected to cyclic temperature differentials with the inside face alternating between 40° and 50°C due to hot oil and 5°C after cooling while the outside face remains constant at 5°C. The duration of these cycles will depend on the operating characteristics of the platform but a storage vessel might be filled in a week and emptied in a day.
As part of the Concrete-in-the-Oceans and other programmes, research has been carried out into these temperature effects. Key references identified during the project are given in Table 22.

The behaviour of concrete structures subjected to high temperature has already been well researched in the context of containment vessels for nuclear reactors. In this instance, however, temperature is generally in a steady state and varies only for the occasional shut down. Early work in this category was undertaken by Monterieff et al (1969). Tests were made on prestressed concrete vessels subjected to a temperature gradient of 40°C. During the first two years of operation, the stress on the inside (high temperature) face reduced by approximately 25% and subsequently to 65% after 25 years. Stresses still present after shutdown of the vessel are attributed to irrecoverable creep. The ACI Committee 349 (1980) has also reported on thermal effects regarding nuclear power structures; this report is significant for the approach taken with respect to cracking of members subject to combined mechanical and thermal loading.

The consideration of temperature effects for nuclear reactors, however, did not adequately represent the more complex loadings experienced with respect to concrete platforms. Further studies were considered necessary in which the stresses are generated due to a difference in temperature between the top and bottom of a cell, through the walls of the cell and due to one cell being adjacent to another at a different temperature.

Richmond (1976) showed that there is a marked redistribution of stresses due to creep. The stress distribution through an element of wall subject to a steady gradient and a fixed level of prestress changes rapidly until eventually, the stress distribution tends towards the initial prestress; that is, the thermal stresses tend to disappear. Similar work was undertaken by England (1976), who studied the behaviour of prestressed beams under cyclic temperature differences. The temperature regime was simplified to a simple step change so that the transition period was ignored from one gradient to the next. The concept of pseudo-time was developed in these contributions. These studies were subsequently taken a stage further by England et al (1979) in which the effects of cyclic filling and emptying of a cylindrical oil storage vessel were studied. Note should also be made of a specific analysis undertaken recently for thermal loading of oil storage tanks (Brakel et al 1981) but this concentrated on the development of successive cracks under steady static conditions without taking creep into account.

Experimental work on the effects of temperature is minimal. Work has been carried out on beams (Ross et al 1965), reinforced slabs (Kayrcheine 1978) and prestressed slabs (Dubois et al 1970) but additional work has, however, been completed with respect to containment vessels for nuclear reactors and concrete chimneys and the results briefly documented by Clarke et al (1987).

Further experimental work specifically on temperature problems arising from oil storage platforms has been undertaken by Clarke et al (1977 and 1987). In the first series of tests, it was concluded that:

- Under current operating conditions, with a realistic level of drawdown, cracking due to local temperature effects was unlikely to be a problem.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clarke JL et al</td>
<td>Subject wall sections to temperature gradients typical of oil storage structures.</td>
<td>Cracking due to local effects is unlikely to be a problem under current operating conditions.</td>
<td>Useful data on creep. Followed up by further work in Phase II.</td>
</tr>
<tr>
<td>Clarke JL et al</td>
<td>Subject reinforced concrete cylinders to temperature gradients typical of oil storage structures.</td>
<td>The provision for minimum reinforcement in the Lloyds regulations and DnV rules would appear to be adequate.</td>
<td>Several recommendations for further empirical work.</td>
</tr>
<tr>
<td>Richmond B et al</td>
<td>Describe methods for calculating stresses and strains, discuss design aspects with worked examples.</td>
<td>As noted in ‘Objective’.</td>
<td>Comprehensive analytical treatment.</td>
</tr>
</tbody>
</table>
• In future designs it should be possible to increase temperatures. This would reduce the need to cool the oil before it was pumped to the storage vessels.

However, a further series of tests was conducted on cylindrical reinforced concrete specimens, subjected to cyclic temperature gradients, as previous tests on beams were considered inadequate after the onset of cracking at the supports, leading to anomalies in the end constraints. The objectives of the second series of tests were to determine conditions under which cracks form, the influence of reinforcement on the number and distribution of cracks, development of cracks during the heating cycle and subsequent cycles, influence of the rate of heating on the onset of cracking, influence of the specimen size on the crack pattern.

It was concluded that low steel percentages produce small numbers of wide cracks whereas high percentages lead to more, but thinner cracks. Provisions for minimum reinforcement in present offshore Codes was considered adequate if slightly conservative. Provision of inner steel appeared to reduce cracking on the outer face for small specimens although this did not apply to larger specimens. It was nevertheless recommended that minimum steel be provided to both faces. The findings of these tests are summarized in Table 23 where a comparison is made between actual reinforcement and reinforcement required under various rules.

Table 23
Comparison of experimental results for temperature effects
(after Clarke et al 1987)

<table>
<thead>
<tr>
<th>Series/specimen</th>
<th>% Reinforcement in specimen (outside face)</th>
<th>% Reinforcement required.</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DnV</td>
<td>Lloyds</td>
<td></td>
</tr>
<tr>
<td>Small Cylinders</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/1</td>
<td>0.34</td>
<td>0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>1/2</td>
<td>0.07</td>
<td>0.32</td>
<td>0.41 Cracks</td>
</tr>
<tr>
<td>1/3</td>
<td>0.52</td>
<td>0.21</td>
<td>0.37</td>
</tr>
<tr>
<td>1/4</td>
<td>0.34</td>
<td>0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>1/5</td>
<td>0.07</td>
<td>0.32</td>
<td>0.41 Cracks</td>
</tr>
<tr>
<td>1/6</td>
<td>0.54</td>
<td>0.18</td>
<td>0.33</td>
</tr>
<tr>
<td>1/7</td>
<td>0.34*</td>
<td>0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>1/8</td>
<td>0.07</td>
<td>0.32</td>
<td>0.41 Cracks</td>
</tr>
<tr>
<td>1/9</td>
<td>0.34</td>
<td>0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>1/10</td>
<td>0.07</td>
<td>0.32</td>
<td>0.41 Cracks</td>
</tr>
<tr>
<td>1/11</td>
<td>0.34*</td>
<td>0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>1/12</td>
<td>0.34</td>
<td>0.18</td>
<td>0.29</td>
</tr>
<tr>
<td>Large Cylinders</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2/1</td>
<td>0.35</td>
<td>0.32</td>
<td>0.44</td>
</tr>
<tr>
<td>2/2</td>
<td>0.11</td>
<td>0.32</td>
<td>0.37 Cracks</td>
</tr>
<tr>
<td>2/3</td>
<td>0.35*</td>
<td>0.18</td>
<td>0.24</td>
</tr>
<tr>
<td>2/4</td>
<td>0.45*</td>
<td>0.18</td>
<td>0.24</td>
</tr>
</tbody>
</table>

Note: DnV values are based on the 1977 regulations. (* both faces)
Comparison of the values in Table 23 shows that cracks appeared in all specimens with 0.07% reinforcement which is well below regulation requirements. Additionally, although actual reinforcement was in some instances below DnV requirements, no failure occurred. It may therefore be concluded that the provision for minimum reinforcement in both the Lloyds and DnV rules appears to be adequate.

The DnV formula includes a water pressure term which was not allowed for in the experimental programme, since all tests were conducted at atmospheric pressure. Similarly, comparison with the chimney codes was difficult as the actual reinforcement percentage adjusted did not cover the low range provided by the chimney codes. Comments are included on differences between the model and the prototype, suggesting that stress will be less critical in the prototype when prestress, heating cycle and wall thickness are taken into account.

Richmond et al (1980) undertook a thorough review of design for temperature effects. This work suggested that methods for calculating thermally induced stresses may be divided into three main categories:

- Elastic analysis for temperature distributions constant with time. This may be achieved through standard methods of structural mechanics ranging from beams on elastic foundations for cylindrical shells corresponding to one-dimensional systems, to Fourier series for a two-dimensional system such as a square cell structure (Timoshenko et al 1970). Finite elements are applicable to problems involving more complex forms.

- Transient temperature and stresses. Stress distribution due to temperature variations in both cracked and uncracked sections may be calculated. An empirical method is also available for calculating crack widths (Hughes June 1971 and Dec 1971).

- Calculation of stresses resulting from creep strains. The basis for calculating effects of temperature induced creep are provided by England (1966) in a description of the steady state solution with respect to the increase in the rate of creep with temperature, assuming that loads and temperature distribution remain constant.

The above contribution is supported by several references providing both theoretical and empirical background. For example, Richmond (1976) plotted stresses for inner and outer faces of a cylindrical storage vessel where the top half was full of oil at 40°C, the bottom full of water at 5°C and the outside was maintained at 5°C. It was shown that there is a marked redistribution of stress due to creep such that after 200 days the stress distribution was tending to the initial uniform prestress. Also, England et al (1979) studied the effect of cyclic filling and emptying of an oil storage vessel.

To summarize, it may be concluded that sufficient experimental work has been completed to provide an understanding of the behaviour of concrete subjected to temperature variations. The results indicate that the DnV regulations and Lloyds rules allow for adequate reinforcement and hence it is recommended that these guidelines be adopted until the results of further more comprehensive empirical work is made available.
3.1.6 Fatigue

Fatigue of reinforced concrete has been the subject of extensive research in several countries during recent years. The use of concrete as a construction material for offshore and marine structures has given added impetus to this work due to the extreme and cyclic nature of wave loading and the aggressive environment in which the materials are located.

The motivation for devoting resources to the understanding of fatigue does not come from observed failures in offshore installations (there appear to be no published records of failures attributable to the fatigue of reinforced/prestressed concrete); rather it results from concern that extreme wave loading conditions coupled with a reduced fatigue life of materials due to the marine environment could lead to a lack of safety in the future.

There has been a significant amount of research undertaken in relation to fatigue of concrete structures. As part of the assignment to revise Guidance for concrete offshore structures, a comprehensive review of published data has been undertaken and the key references summarized on Table 24.

The steps required in undertaking a fatigue analysis/design have been illustrated by Price et al. (1987) in a flow chart, reproduced as Figure 8. From this it will be seen that information is required on the following:

- Loading and frequency data
- Static and dynamic response of the structure to this loading, leading to stresses in individual members and details
- S-N curves for concrete, reinforcement and prestressing tendons
- A method of assessing cumulative damage due to a range of different cyclic loads and an acceptance criterion
- Knowledge of the suitability of detailing practices e.g. laps, welds, bends, bond etc
- Life and maintenance policy for the installation

The principal cyclic loading on offshore structures is generally that due to wave action although for individual components of the structure other loadings may be significant and fatigue analysis should not necessarily be limited to wave loading only. Which, if any, additional loadings to include when considering fatigue, must rest with the judgement of the designer.

To carry out a fatigue analysis for wave loading, a spectrum of wave heights and associated number of occurrences for a particular location is required. The stresses induced in a given member vary with wave direction and an assessment must be made of the proportion of waves coming from each direction which produce significant stresses. The determination of a wave spectrum suitable for carrying out a fatigue analysis is discussed by Price et al (1987).
Table 24
Key references for fatigue (sheet 1 of 2)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Price WJ et al</td>
<td>To review the magnitude of the problem of fatigue in design and in practice for offshore structures.</td>
<td>No evidence of actual fatigue failure was recorded. A fatigue analysis for various types of structures and members was carried out. Offshore platform types of structure were in general not prone to fatigue.</td>
<td>The paper contains very useful information and guidance for fatigue analysis.</td>
</tr>
<tr>
<td>Concrete in the Oceans</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Technical Report No. 12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>November 1987</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gerwick BC et al</td>
<td>A general review of fatigue in offshore concrete structures.</td>
<td>Cumulative fatigue damage is low at the high cycle end of the spectrum. The low cycle high amplitude end of the spectrum is more likely to be critical. Principal tensile stresses due to shear can be controlled by prestressing.</td>
<td>Curves are given for likely stress ranges versus number of occurrences for a typical North Sea environment. Wohler curves are given for concrete, reinforcement prestressing stress strength.</td>
</tr>
<tr>
<td>High- and low-cycle fatigue</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>behaviour of prestressed concrete</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>in offshore structures.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Offshore Technology Conference</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>May 1979</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tilly GP</td>
<td>A general view of fatigue of reinforcement, written from the standpoint of bridge design examples.</td>
<td>Detailed review of the different parameters affecting fatigue life and their relative influence. A large variability of results was observed.</td>
<td>The paper contains a section on corrosion fatigue somewhat superseded by recent research.</td>
</tr>
<tr>
<td>Fatigue of steel reinforcement bars in concrete - a review.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatigue of Engineering Materials and Structures</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vol 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1979</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reference</td>
<td>Objective</td>
<td>Conclusion</td>
<td>Comments</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------</td>
<td>------------</td>
<td>----------</td>
</tr>
<tr>
<td>Paterson WS et al</td>
<td>Observation of realistically dimensioned beams under cyclic flexural load in a submerged and partially submerged condition in seawater. Determination of an S-N curve for Torbar reinforcement.</td>
<td>A seawater environment causes a significant reduction in fatigue strength of reinforcement. No fatigue limit was established. Crack blocking with magnesium hydroxide was observed.</td>
<td>A difference in conclusions between this work and work undertaken by Arthur et al. Currently being resolved by the Concrete in the Oceans Fatigue Steering Group.</td>
</tr>
<tr>
<td>British Steel Corporation</td>
<td>Establishment of a data bank for reinforcement fatigue strength. Derivation of guaranteed performance curves for reinforcement.</td>
<td>A computerised data bank for a wide range of reinforcement now exists. Equations for guaranteed fatigue performance are given. For corrosive environments no fatigue limit is guaranteed.</td>
<td>Access to the data bank is via the British Steel Corporation.</td>
</tr>
<tr>
<td>Waagaard K</td>
<td>S-N design curves for concrete in compression/compression and compression/tension including effects of hydrostatic pressure.</td>
<td>Compression/tension significantly more damaging than compression/compression. Presence of water crack is critical, not pressure.</td>
<td>Specimens tested were heavily reinforced (3.6%) and splitting along reinforcement was common failure mode.</td>
</tr>
<tr>
<td>Offshore Certification Bureau</td>
<td>Examination of applicability of draft guidance on fatigue, examination of the assessment process and susceptibility of existing designs to fatigue.</td>
<td>No evidence of critical fatigue lives for present platforms. Concrete subject to compression/tension could be critical in fatigue.</td>
<td>More information required on stress concentration factors applicable to reinforced concrete.</td>
</tr>
</tbody>
</table>
Figure 8
Fatigue analysis procedure (after Price et al 1987)
Waagaard (1982) recommends dividing the wave spectrum into loads of average amplitude. A fatigue check for each of these amplitude ranges and the associated number of cycles can then be carried out and the cumulative fatigue effects evaluated using Miner's hypothesis.

The calculation of member forces and stresses, given the wave loading, can be done using static methods where the natural period of the structure is much shorter than the period of the cyclic loading. If the cyclic load produces a significant dynamic response in the structure, dynamic methods of analysis have to be used. The member stresses may also be determined by model testing.

Particularly high stresses may occur at sudden changes in cross-section and at openings. These so-called 'hot-spots' are the places where fatigue problems are most likely and should be the subject of a fatigue analysis. However, the extent to which stress concentration factors should be applied to reinforced concrete is unclear. Careful detailing can mitigate the problems to some extent.

Price et al (1987), referred to above, carried out fatigue analyses for some typical designs of offshore oil platforms and also for wave energy converters. Generally they found that the wave energy converter designs were much more prone to fatigue than the platform designs. Gerwick and Venuti (1979) concluded that for wave conditions such as those in the North Sea, low amplitude loading with a high number of cycles (≈ 1 million) was not critical. They suggested that the high amplitude low cycle end of the loading spectrum was the area requiring careful attention. This statement should however be viewed with caution in the light of the work on corrosion fatigue of reinforcement by British Steel Corporation (1987). Work by Waagaard et al (1986) suggests substantial fatigue usages in submerged concrete at the low stress/high cycle end of the spectrum, particularly in the case of stress ranges exceeding crack opening and pumping of water in the crack.

Most of the work undertaken on the subject of fatigue of reinforced concrete has been directed towards establishing the separate fatigue lives of the reinforcement and the concrete. The fatigue strength of reinforcing bars for concrete has been the subject of many research programmes and Tilly (1979) has carried out a review of research on a range of bars commonly in use in the United Kingdom. Factors which affected the fatigue life of the bars were commented on as follows:

- The bar geometry (smooth, ribbed, twisted etc) has a marked effect on the fatigue life of reinforcement. Plain round bars have considerably longer fatigue lives due to the absence of stress 'hot-spots' at the base of ribs. It was also noted that the sharpness of the radius at the root of the rib had an effect. Bars produced using worn rollers had an improved performance due to the greater radius at the root of the rib. Surprisingly perhaps, manufacturer's markings in the form of raised features caused a considerable reduction in the fatigue life. In cases where the markings had been filed off the fatigue life was increased by as much as 100%.

- The fatigue strength of reinforcement decreases with increasing diameter. This decrease is much more pronounced for ribbed bars than for plain bars.

- It has been found that fatigue strength does not increase in proportion to static strength. There is some increase in fatigue strength with increasing static strength, but higher strength bars are likely to have a lesser fatigue strength as a proportion of their static strength.
• Reductions in fatigue strengths due to corrosion in the order of 20% - 40% for tests up to 10 cycles were observed from the research reviewed. The presence of good concrete cover lessens the problem to some extent.

• Axial tests on bars tend to produce somewhat more conservative results than the flexural beam tests. The reason put forward for this is that the probability of an inherent weakness occurring within an area of high stress is greater for the axial tests than for flexural tests, due to the stress gradient through the bar in the latter tests. Tilly quotes a maximum difference of 10% between the two methods of testing.

Paterson et al (1987), have carried out tests on bars of a realistic diameter and geometry, in beams with cover and concrete grade of the type used in offshore work, in order to eliminate some of the many variables referred to above.

As part of this programme, Paterson et al (1987) have undertaken a series of flexural beam tests using 32mm diameter bars (Torbar) with 60mm cover and 1% reinforcement content. The concrete grade was 50-60 N/sq.mm at 28 days. Tests were carried out in air, partially submerged in seawater, and under pressure to simulate beams submerged at a depth of 30m. The stress ranges used in the tests varied from 360 N/sq.mm down to 100 N/sq.mm, the mean stress being such that tension was always present in the reinforcement.

The results of these tests are summarized in graph-form in Figure 9. A phenomenon noted during the course of this test series was crack blocking where magnesium hydroxide was found to be forming in the cracks causing the minimum beam deflection to increase as the number of cycles increased. This in turn had the effect of raising the minimum stress, thus reducing the stress range and of raising the mean stress level. The amount of crack blocking cannot however be accurately predicted. As the results are for 32mm diameter ribbed bars they can safely be used for smaller bar diameters of the same steel quality. The largest number of tests was carried out for a partially submerged environment and this may be considered equivalent to the splash zone. The results from these tests show a marked reduction in fatigue strength when compared with those for an air environment. The number of tests carried out for fully submerged beams at a pressure equivalent to 30m of seawater was only four, and it is not clear whether this category is different to the partially submerged one.

The difference in fatigue strength between tests in air and tests partially submerged in seawater becomes more significant at lower stress ranges.

In addition to the differences in fatigue strength caused by environment, there also appear to be marked differences due to the frequency of loading. Frequencies of 3Hz and 0.1Hz were used in the tests and the 0.1Hz frequency corresponds quite closely to typical North Sea wave frequency.

Two modes of fatigue fracture were observed in these tests: mechanical initiation at rib intersections and corrosion initiation. In the longer duration tests in seawater, corrosion initiation superseded the mechanical initiation mode.

Hodgkiess et al (1987) have reported on a test where the main reinforcement was 10mm dia (Torbar) and the beams were tested in reverse bending. The results from this research show virtually no reduction in the fatigue life of reinforcement in concrete beams in a seawater environment when compared with an air environment.
Figure 9
S-N curves for reinforcement (after Paterson et al 1987)
The apparently contradictory results from the two above sets of research have been discussed by Leeming (1987) and an explanation has been postulated based on differences in the rate and type of corrosion caused by the two different test arrangements. In the tests by Paterson et al (1987) the full length of the beam was submerged or partially submerged whereas for the Hodges et al (1987) tests, only the central portion was surrounded by seawater, and the ends were in air. It is believed that the differences in electrochemical potential thus generated lie at the root of the different corrosion fatigue behaviour. The Hodges et al (1987) tests exhibited an accelerated rate of corrosion which caused blunting of any fatigue crack in the reinforcement and prevented its propagation, thus leading to fatigue lives no shorter than in an air environment. The test arrangement used by Paterson and Dill is now believed to represent more closely the actual conditions in the splash zone of an offshore platform.

The S-N curve method of representing data does not take into account the mean stress level, thus, for example, a test cycling between +250N/sq.mm and +350N/sq.mm with a stress range of 100N/sq.mm would have the same ordinate as a test cycling between -50N/sq.mm and +50N/sq.mm. A minimum or a mean stress level must be defined in order to make the test unique. However, work by British Steel Corporation (1987) has shown that, within the normal working stress limits in reinforcement, the mean stress does not significantly affect the fatigue life of bars in a corrosive environment. Where no fatigue limit can be guaranteed, British Steel Corporation have proposed an equation for a lower bound guaranteed performance curve, in which the effect of environment and bar diameter can be taken into account.

Paterson et al (1987) have reported on the fatigue behaviour of beams with a simulated cementitious repair, beams made using PFA (20% cement replacement) and beams with cathodic protection. From the limited number of tests carried out, these results appear to lie within the normal scatter of results for the standard beams (i.e. no prominent advantage or disadvantage was observed). Gerwick (1979) gives S-N curves for prestressing strand, indicating failure at stress ranges of not more than 15% of the static strength. This agrees with Tilly's comments that the fatigue strength does not increase in proportion to static strength. Paterson et al (1987) have also reported on a set of fatigue tests on prestressed concrete beams in seawater. It should be noted that the cyclic stress ranges in concrete structures will generally be low relative to the high static strengths of post-tensioning tendons. The importance of tendon bonding is emphasized by Price (1981).

Price also quotes experimental and design S-N curves for concrete under variable compressive stress. These are reproduced in Figure 10. The paper describes the failure mode as: 'For plain concrete, fatigue is a process of progressive permanent internal structural change in a material subject to repetitive stresses and strains. These changes may be damaging and result in progressive growth of cracks and eventually fracture, if the number of repetitions of stress is sufficiently great. If the fatigue strength of concrete from tests is expressed as a fraction of the static strength that it can support repeatedly for a given number of cycles under particular conditions, then this strength is essentially the same whether the stress regime is tension, compression or flexure. The effects of a wide range of variables, including mix proportions, loading and environmental conditions can be approximated by a single mean endurance or S-N curve'. Work by Ralihby (1979) corroborates this concept of describing the fatigue strength of concrete by a single curve.
Figure 10
S-N curves for concrete (after Price et al 1987)
Waagaard (1986) reports that when pre-cracked reinforced concrete specimens are subjected to constant amplitude compression/tension cyclic loading the compression fatigue life is significantly reduced at the low stress-high occurrence end of the range.

The usual way of assessing cumulative damage is by means of Miner's hypothesis. The hypothesis is empirical and has been investigated by Van Leeuwen and Siemes (1979). For concrete the Miner's sum was found to have average values generally below 1.0 showing that variable amplitude loading is more damaging than constant amplitude loading. Waagaard (1981) suggested values between 0.2 and 0.5 and the DnV rules stipulate a Miner's number of 0.2. This reduced Miner's number value does not include a safety factor, it is purely to take account of the inaccuracy of the method. Price et al (1987) recommend that further research work is required to arrive at a uniformly safe approach.

Tilly (1979) and British Steel Corporation (1987) have reviewed research on butt welded joints in reinforcing bars and conclude that the presence of butt welds causes considerable reduction in the fatigue strength of the reinforcement. The effect was not as severe for tests on reinforced concrete as for bars with butt welds tested axially.

Mechanical splices have been widely used in recent years and tests have shown satisfactory performance under fatigue loading. Tests by Bennett (1979) on laps have shown that cranked laps have very poor fatigue performance. Straight laps performed better, but there is some evidence that these laps also form a source of weakness. Bends in the reinforcement with an internal radius of less than 25 bar diameters exhibit a reduction in fatigue strength (Price et al 1987). For bends of radius equal to 10 bar diameters the reduction in fatigue strength becomes about 50% for deformed bars and 30% for plain bars.

Gerwick (1979) has addressed the question of the cyclic shear capacity of offshore concrete structures and concludes that the use of prestressing to limit principle tensile stresses in the concrete is an economical way of enhancing the fatigue life.

Generally, present design guides (e.g. FIP, DnV, ACI) are divided into preliminary checks to establish whether a particular member is likely to be prone to fatigue or not, and detailed analysis procedure for fatigue. The preliminary rules are quite conservative and if they indicate a possible fatigue problem then a detailed fatigue analysis is required.

In the light of the research evidence reviewed, certain conclusions can be drawn and these are:

- A design S-N curve for high yield reinforcement can be derived from the relatively large number of test results now available.

- Within the normal static compression stress limits, concrete subjected to cyclic loading does not appear susceptible to fatigue failure under wave type loading. Excursions into the tensile stress range are however damaging and must be kept to a minimum.
• Within the normal static stress limitations, cyclic loading will not produce failure below 10,000 cycles. It should be noted that in the case of members subject to wave loading the number of cycles will be greatly in excess of this figure and a fatigue check is appropriate.

• Miner's summation for calculating cumulative damage is rather unsatisfactory for concrete, but is the only convenient method available.

• Some detailing recommendations can be made concerning bends and laps.

On the basis of these conclusions a draft clause on fatigue was drafted for the Guidance Notes. A study was carried out (Offshore Certification Bureau 1986) to examine the applicability of the clause to existing platform designs and to formulate a view on the susceptibility of these designs to fatigue. The study drew on the design check calculations for one platform and reviewed selected fatigue calculations and the results of an instrumentation report for others.

It was concluded that for concrete continuously in compression at stress levels within the normal static design limits, fatigue life was considerably in excess of design life if not indefinite. The experimental work carried out has, for time reasons, not extended beyond about $10^7$ cycles. The shape of the S-N curve at higher numbers of cycles is a matter of extrapolation and whereas the design curve quoted by DnV follows a straight line down to zero stress range at $10^{10}$ cycles that used by TNO has an endurance limit at half the design static strength beyond $2 \times 10^6$ cycles. Both curves are reasonable based on the experimental data, but when calculating cumulative usage the DnV curve gives much higher values. For cases where predominantly compressive stresses acted in combination with repeated tensile stresses, causing cracks to open and close with water pumping in and out, the fatigue life could be seen from Wangaard's work to be markedly reduced. The compression fatigue strength of reinforced concrete might then become a critical design case. However, if under permanent loads the concrete is subject to substantial compression, although tensile stresses occur under extreme wave loading, only those loading bands in which the tension occurs need be assessed using a tension/compression design curve; all others should be assessed using a compression/compression design curve. If a section is under tension or very low compressive stress due to the permanent loads then the cumulative usage would be assessed entirely using the tension/compression design curve and critical results may be obtained.

The fatigue life of reinforcement was examined in the study using an S-N curve produced under the Concrete-in-the-Oceans programme. This curve is more onerous than other design curves used to date and has no endurance limit. However the study indicated that the stress ranges and number of occurrences anticipated by existing platforms would still lead to fatigue lives in excess of the design life.

The study found that the applicability of stress concentration factors to reinforced concrete was not a well researched area. In the study no stress concentration factors were used. However if stress concentration factors were adopted, then fatigue lives would drop to critical levels. In the analysis of concrete offshore platforms finite element shell analysis is invariably used and if the modelling is sufficiently fine, the stress concentrations will automatically be picked up. However if a coarse analysis only is used, there may be a case for using additional stress concentration factors.
During the study a computer program was developed to aid the assessment of fatigue cumulative usage. From the results of the finite element shell analysis and the reinforcement details the cumulative usage could be obtained relatively easily. Reinforcement stresses were evaluated using a modified sandwich method which took account of the central layers of steel. When examining concrete stresses, extreme fibre principal stresses were used.

In assessing the cumulative usage, Miner's hypothesis was used. For reinforcement a Miner's number of unity was used as the acceptance criterion. However in the case of concrete, the method has been shown by Van Leeuwen and Siemes to be more erratic and a Miner's number of 0.2 was used in accordance with the DnV recommendations.

In the calculations examined, as well as the independently assessed fatigue damage, factors of safety on a 30 year design life dropped no lower than 6. An exception was where tension failure of the concrete (as opposed to compression failure in compression/tension cycling) was given equal weight to compression failure. In this case the provision of additional prestressing eliminated the possibility of cracking under cyclic loading. The presence of tension or low compression stresses under permanent loads in an area subject to significant cyclic load would require particular attention to be given to compression fatigue life of submerged concrete.

3.1.7 Shear

During the period between 1950 and 1970 it had been recognised that the treatment of shear design in European practice was inconsistent and could lead to unsafe design. Hence, a number of research programmes were initiated to obtain a better understanding of the mechanism of shear failure. While committees were formed by the American Concrete Institute at various times to study the problem, the first major attempt in the U.K. to make a systematic examination of the subject was the formation of the Shear Study Group in the Institution of Structural Engineers in 1965 to study existing available research data and propose theories on the subject. This coincided with the drafting of CP110 and this Group became the Code Servicing Panel to make proposals for shear clauses in the new Code. The resultant shear clauses proposed, incorporated the latest understanding on the behaviour of concrete structures under shear at various stages, as a result of studying numerous research reports and test data. The shear clauses in CP110 were considered, at that time, a major breakthrough in rationalizing the understanding on shear design.

After the publication of CP110, research work on the subject continued. Both Gustafsson (1982) and Bazant et al (1983) explained shear failure in terms of fracture mechanics, while Braestrup et al (1979) utilized rigid plasticity. Regan et al (1981) proposed failure models that explained test results. All these approaches produced useful structural models or failure criteria to explain the behaviour of a member subjected to shear, but much more development is needed for practical application. By 1984 there was no available research work to justify amending the shear clauses in CP110. However, some improvement could be considered in the application of the depth factor in slabs. More detailed examination of experimental results indicated that thicknesses of up to 500mm could be affected.
For punching shear, more recent research has only confirmed that, in general, the existing approach, set out in BS 8110, is fundamentally adequate and practical. Certain refinement can be made in calculating the punching parameter as a matter of convenience rather than a change of principle.

In the calculation of shear reinforcement the use of truss analogy has always been considered a realistic approach. The choice of the inclination of the truss members has been arbitrary but on the conservative side. In BS 8110 a 45° inclination is considered appropriate and convenient. The CEB committee on shear recommends a flatter inclination to utilize reinforcement more efficiently. Collins et al (1980) whose approach takes into account compatibility as well as appropriate concrete failure criteria, also discussed this aspect.

The above comments apply to beam type structural elements in which the shear failure mechanism has been defined. In the case of offshore structures, the assumptions stipulated in BS 8110 or other normal codes of practice for building elements are not always valid. Even the more recent work by Regan et al (1981), specifically for offshore type structures, did not confirm or refute the validity of the BS 8110 approach to certain offshore type structural elements in the treatment of shear. Since modern techniques are available to determine the state of stress resultants in any complex structural element, the determination of the principal tensile stresses remains the only reasonable way to assess the effect of shear. The Guidance Notes for shear are based on this principle.

3.2 MATERIALS

Two of the major materials used for concrete production are cement and aggregates and these are discussed, in this section, in relation to their development and potential difficulties. Where appropriate, recent research is highlighted. In addition a commentary on cement replacement materials and admixtures is included as well as other materials, although the replacement materials are now covered in BS 8110.

Research has continued over the past years into the properties and behaviour of reinforced concrete and its constituent materials although the Concrete-in-the-Oceans programme has undertaken little work directly in relation to materials. The effects of differing concrete grades have, for example, been considered but only within a limited range of values.

A comprehensive literature search has been conducted as part of the revision process for the Guidance Notes. Several references have been identified as being of particular relevance and these are listed in Table 25, along with a summary of their contents. The full listing of all references is given in the Database listing as an Appendix to this report.

Since the 1950's there has been a great expansion of the use of concrete. Much of the development, notably where high strength concretes have been essential, would not have been possible if the cement industry had not been able to respond by producing higher strength cements (Corish et al 1982).

However, cement has chemically remained similar over the past 130 years. Cement fineness has also remained static over the years although fineness differences between
<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corish A et al&lt;br&gt;Portland cement properties - past and present. Concrete, July 1982</td>
<td>To explain developments from 1840 to date.</td>
<td>Reveals magnitude of cement strength increase this century.</td>
<td>Dispels the 'fineness' myth.</td>
</tr>
<tr>
<td>Nison PJ&lt;br&gt;Changes in Portland cement properties and their effects on concrete. BRE Information Paper IP3/86, 1986</td>
<td>To summarise significance of changes for benefit of practicing engineers.</td>
<td>Need for practising engineers to recognise and understand significance of changes for both specification and construction.</td>
<td>Brief but informative.</td>
</tr>
<tr>
<td>Anon&lt;br&gt;Changes in cement properties and their effects on concrete. Concrete Society Working Party Report 1984</td>
<td>To review the properties of cements and cementitious materials and their changes over the years. To identify key factors affecting durability. To produce recommendations to ensure long term durability.</td>
<td>Extends findings of Corish et al above and summarises changes.</td>
<td>Recommends need for further consideration of durability criteria.</td>
</tr>
<tr>
<td>Lea FM&lt;br&gt;The chemistry of cement and concrete. Third Edition 1970</td>
<td>Comprehensive treatise and a primary reference in its field.</td>
<td>Innumerable, including risks of sulphate attack by sea water, not as great as sulphate content would suggest.</td>
<td>Relevant to selection of cement type and durability generally.</td>
</tr>
<tr>
<td>Reference</td>
<td>Objective</td>
<td>Conclusion</td>
<td>Comments</td>
</tr>
<tr>
<td>----------------------------------------</td>
<td>---------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------</td>
</tr>
<tr>
<td>ASTM</td>
<td>Comprehensive treatise and one of the primary references in its field.</td>
<td>Numerous relevant articles on concrete properties and performance.</td>
<td>An essential reference to all aspects of concrete properties and performance.</td>
</tr>
<tr>
<td>ACI</td>
<td>As title.</td>
<td>Scawater less aggressive from sulphate attack aspect than its SO4 content would suggest.</td>
<td>Merits consideration by specifiers. See also ACI 318R-83 (Commentary on ACI 318).</td>
</tr>
<tr>
<td>Crowder JR</td>
<td>As title</td>
<td>SRPC may be undesirable where possibility exists of chloride contributing to the corrosion of reinforcement.</td>
<td>Merits consideration by specifiers.</td>
</tr>
<tr>
<td>Treadaway KWJ et al</td>
<td>Interim review report of joint studies by BRE and Aston University.</td>
<td>Cements of different chemical composition may have substantially different capacities for protecting reinforcement.</td>
<td>Merits consideration by specifiers.</td>
</tr>
<tr>
<td>Reference</td>
<td>Objective</td>
<td>Conclusion</td>
<td>Comments</td>
</tr>
<tr>
<td>-------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>Weibenga JG</td>
<td>Proceedings of International Conference, Performance of Concrete in Marine Environment. St Andrews-by-the-sea, Canada.</td>
<td>Relates to both in-service and laboratory investigation.</td>
<td>Useful compilation.</td>
</tr>
<tr>
<td>Idorn GM</td>
<td>Doctoral thesis.</td>
<td>Various, including significance of extreme alkalis versus ASR.</td>
<td>Relevant to limitations on external alkalis, as in marine situations.</td>
</tr>
<tr>
<td>Haynes III et al</td>
<td>To obtain data on time dependent failure, permeability and durability of concrete structures in deep ocean environments.</td>
<td>Compressive strength of concrete decreased due to becoming saturated with seawater and the rate of strength gain with time of concrete in deep-ocean slower than fog-cured concrete.</td>
<td>Raises a number of unexpected findings, in particular the drop of concrete strength after being placed in ocean.</td>
</tr>
</tbody>
</table>
Table 25  
Key references for materials (sheet 4 of 4)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Haynes HH et al  
Compressive strength of freshly mixed concrete placed, cured and tested in the deep-ocean. 
Technical Note N1609  
US Civil Eng. Lab.  
February 1981 | To investigate the behavior of concrete placed on the sea floor and allowed to set and cure. | Both low and high strength concretes had similar compressive strengths at different pressures. | Also found similar results to Haynes et al 1979. |
makes do exist. Setting times have shortened slightly over the years, largely as a result of pressure from users, and 28 day strengths have increased four or five-fold this century and about 25% in the last 20 years (Anon, Concrete Society Working Party Report, 1984; Nixon, Building Research Establishment Information Paper IP3/86, 1986).

Ordinary Portland cement, which constitutes about 90% of UK produced cement, has been produced continuously since about 1840 and BS 12 was first issued in 1904. In 1920 production of a Portland blastfurnace cement began at Gartsherrie in Scotland and this works remained the principal UK source of blastfurnace cement until recent date, but the amount produced was less than about 0.5% of the total cement UK production. Ordinary Portland cement was thus the only Portland cement in continuous national production for over a century. Sulphate-resisting Portland cement production commenced in the fifties and was developed primarily to combat sulphate attack from ground and groundwater, rather than for sulphate resistance in sea water.

Pulverised fuel ash has been used extensively in major civil engineering works, including offshore structures. The use of pfa in concrete, begun in the fifties primarily for reasons of heat reduction and cost, has continued and developed in more recent years following a processed quality controlled pfa (‘pozzolan’) becoming commercially available. The first standard for pfa, BS 3892, was issued in 1965. The material was recognised as an admixture for concrete, but not as a cement replacement material, in the first edition of CP 110 in 1972. In 1985, BS 6588 was published for Portland-pfa cement (15-35% pfa) and BS 6610 was also published for Pozzolanic cement (35-50% pfa).

The use of ground granulated blastfurnace slag (ggbs) as a cement replacement material began in the mid-sixties. Continuous, production of a ggbs, known as ‘Cemsave’, began in 1969 and the first Agreement Certificate for this product was issued in 1972. In the same year it was accorded recognition by the Department of Transport and the product was included in BS 5328 in 1976. The use of ggbs is extensive in both building and civil engineering work for reasons of cost, sulphate resistance and heat reduction. In 1984, a BSI Committee was preparing a standard for ggbs for use with cement and its publication (as BS 6699) was anticipated for 1986.

Tricalcium aluminate (C3A) is a compound in hydrated Portland cement which is susceptible to reaction with sulphates and, in extreme cases, this reaction can result in the disintegration of concrete. The reaction was identified in the early part of this century and ultimately led to the development of sulphate-resisting Portland cements where the C3A content was limited. In the case of BS 4027 cement, the maximum tricalcium aluminate content is 3.5% and for ASTM C150 Type II and Type V cements it is 8.0% and 5.0% respectively. Neither BS 4027 nor ATSM C150 Type V standard specifications for sulphate-resisting Portland cements impose a minimum C3A content.

The sulphate content of seawater (as SO3) is about 250 parts per 100,000 and this level, if in a ground water, would be classified by CP110 (Table 4) as a borderline but not covered in current British Standard Codes or Specifications but Class3/Class4 exposure. However, Lca (1970) suggests that this classification is unduly severe and that a lower classification is more appropriate. A lower classification for seawater is not presently covered in current British Standard Codes or Specifications but is covered, for example, in the ACI Building Code ACI 318-83. The high quality concretes used in the exposed or vulnerable parts of offshore structures are virtually certain to satisfy the lower classification requirements and thus be significantly more resistant to external sulphate attack because of their much lower permeability. Hence the need for a maximum C3A content of 12% is questionable for these offshore concrete
structures, but such a limit is not likely to present any significant supply problems. Evidence of deleterious sulphate reaction in high quality concrete marine structures is rare.

An increasing number of researchers, in various countries, have investigated relationships between the C3A content of Portland cement and both chloride ‘immobilisation’ in fresh concrete and chloride penetration resistance in hardened concrete. These investigations are mainly related to reinforcement corrosion and durability studies and suggest a balance against very low C3A cement concretes in chloride rich environments (eg offshore and other marine structures) and in favour of OPC-pfa and OPC-slag cements. Holden et al (1983) give a ranking order for chloride diffusion as reproduced in Table 26. Both Treadaway et al (1984) and Crowder (1983) have expressed tentative indications that corrosion risks in reinforced concrete could be influenced by cement type and, further, that low C3A content Portland cements may not be desirable in chloride contaminated concretes, or concretes in chloride rich environments such as seawater. However, there appears to be a paucity of long term UK evidence in support of this research indication except perhaps, that ordinary Portland cement was used successfully for nearly a century in marine works prior to the production of BS4027 cements in the fifties.

<table>
<thead>
<tr>
<th>Type of Cement</th>
<th>Diffusivity ( (X 10^8 \text{ cm s}^{-1}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC-A</td>
<td>3.14</td>
</tr>
<tr>
<td>OPC-B</td>
<td>4.47</td>
</tr>
<tr>
<td>OPC-B/30% PFA</td>
<td>1.47</td>
</tr>
<tr>
<td>OPC-B/65% BFS</td>
<td>0.41</td>
</tr>
<tr>
<td>SRPC</td>
<td>10.00</td>
</tr>
</tbody>
</table>

The use of high aluminum and super sulphated cements involves special considerations and the conditional exclusion of both is considered appropriate for the Guidance Notes.

The principal British Standard for aggregates for concrete, BS882 (Aggregates from Natural Sources for Concrete), has been substantially revised and was republished in 1983. Hence, it reflects the current general state-of-the-art regarding aggregate requirements, although some points may require further consideration for particular contract specifications. However, it does not cover the subject of alkali-aggregate reaction (AAR) except by reference to BS 8110.

Work carried out by Hawkins et al (1983) reflects the present state of knowledge regarding alkali aggregate reactivity and the means to avoid or minimize the risk of damage due to ASR. Further work by Hawkins et al is presently being undertaken and a revised and extended edition of the original report was in preparation at the time this Guidance was drafted. More than one type of reaction can occur between aggregates and alkalis, but the most common is when alkalis, normally those present in Portland cement in concrete, react with certain forms of silica in the aggregate to form a gel which
The passage of moisture into and through concrete can cause alkali metal salts, such as those present in seawaters, some groundwaters and other materials, to migrate and create temporary or permanent concentrations of these salts in some regions of the concrete. It is, however, a matter of opinion whether and by how much, alkali migration can cause or increase damage due to ASR. Hawkins et al acknowledges that alkalis 'may be absorbed by hardened concrete in contact with seawater', but explains that opinions vary about the effect that such external alkalis may have on any damage caused by alkali reactivity. Idorn (1967) puts forward some evidence to suggest that the alkalis in seawater may have a potentially detrimental influence on concrete that is otherwise vulnerable to alkali-reactivity.

Precautions against alkali reactivity which depend solely upon the control of alkalis within the concrete at the time of mixing may be less reliable when subsequently the hardened concrete becomes exposed to seawater. The use of cement replacement materials such as ggbs or pfa, two of the recommended alternative measures in the report by Hawkins et al, may therefore be the more relevant recommendation against alkali-reactivity in such circumstances. Research indicates an overall improvement in durability may be achieved with these materials due to a greater impermeability of the resultant concrete, but the influence of any curing inadequacies has to be considered.

The reported cases of ASR are fairly widespread geographically but there is no apparent history to indicate that the incidence or severity of damage in marine structures is any greater than in any other areas. It is therefore a question of balancing these risks against the additional costs, if any, of minimizing them.

There has been very limited research into concrete mixes used in offshore construction. Existing Codes of Practice such as FIP (4th Edition) require a minimum concrete grade of 40 but the practice to date had been to produce design mixes of grade 50 and over.

As part of the Concrete-in-the-Oceans programme, Taylor Woodrow (1987) studied the effect of concrete grade on chloride ingress. Nine different mixes were chosen but all were typical of the grades used in the industry. The low grade concrete, with a 28 day average strength of 50.3 N/sq.mm, gave the highest chloride ingress although not significant.
A limited amount of research has been undertaken to investigate the effects of immersion of concrete in seawater. One such programme sponsored by the US Naval Engineering Command began in 1971 to obtain data on time dependent failure, permeability and durability of concrete spheres actually placed at various depths in the ocean. A paper on the findings after 6 years by Haynes et al (1979), reported some significant and unexpected results.

In particular it was noted that the structures suffered a decrease in strength of about 10% on being placed in the sea (due to saturation). A further 1 to 2 years was then required to regain strength equal to the 28 day fog-cured strength. A graph showing these results is shown as Figure 11. After 5.6 years in the ocean the concrete shows a compressive strength that was 15% less than the concrete which was fog-cured.

More recent research by Stillwell (1987) as part of the Concrete-in-the-Oceans programme on corrosion also noted a reduction in compressive strength in deep immersed concrete. Stillwell's results are superimposed onto Figure 11 but it should be noted that the immersed (ocean cured) line (indicated by *) is interpolated from his results. This is due to the time lag which occurred between the removal of the specimens from the sea and their testing. Of the three results indicated this period was 2 months, 2 weeks and 7 months during which time the concrete was stored in laboratory conditions and thus able to regain strength.

Figure 11
Relative compressive strength of fog-cured and ocean-cured concrete
(after Haynes et al 1979 and Stillwell 1987)
At the time of writing this report, further work was being carried out at the Building Research Establishment. This measured strengths of concrete cubes in air but gave results which corresponded to those obtained by Haynes et al whilst measuring under pressure. However, the corresponding tensile strength loss, measured by a gas pressure tension test, was much more severe with results of over 80% loss. The initial results from the BRE work indicated that most of the loss occurred whilst under pressure and was not caused by depressurisation.

Haynes et al (1979) summarized by speculating that the cause of the decreased concrete strength was due to a combination of the temperature of the seawater, increased pressure at the depth of placing and the seawater itself. Previously, research by Lorman (1971) had noted that ocean cured concrete developed strength more slowly than fog cured concrete and indicated temperature had a significant role in this.

Haynes et al (1979) concluded by stating that the compressive behaviour of immersed concrete indicated that increases in strength with age, as permitted by CP110 (1972) may not be appropriate for offshore structures. The influencing parameters of low water temperature, pressure and seawater itself slow the strength gain behaviour of concrete after an initial decrease in strength due to saturation effects.

The research has identified a significant reduction in compressive strength on placing in the sea although this is based on limited available data. It is suggested that further research is needed to actually determine the changes in concrete strength that occur over the first 1 to 2 years. However, in the time scale of the construction of a concrete platform it seems unlikely that this strength reduction will have any significant influence in the design but it may be worth investigation.

The relevant British Standards or Codes of Practice for other constituent materials contain the state-of-the-art information and properly cover testing and other requirements. The requirements for some admixtures are already covered and others will be covered in British Standards being prepared at the time of writing this report.

3.3 CONSTRUCTION

3.3.1 Construction and planning

Limited research, as such, has been carried out into construction of offshore concrete structures. Methods and techniques have been developed as part of the overall design development and many papers written on construction of various structures. References have been identified for construction and planning topics and key references are summarised in Table 27.

The 3rd Edition of the Guidance Notes contains few requirements on construction aspects and yet the construction standards are an important element in the long term fitness of an installation. It is perhaps important to realise that experience acquired to date in the North Sea has indicated that no new technological barriers have been encountered in the construction work. All materials, techniques and plant employed have previously been proved in practice. The various papers published by Derrington (1976 and 1977), Doris (1977), Lindgren (1979) and Meleod (1979) all conclude that it has been the very scale of the operation which has been the main challenge.
### Table 27
**Key references for construction topics (sheet 1 of 2)**

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Derrington JA  
TP1: The construction of Gas Treatment Platform No 1 for the Frigg Field for Elf-Norge A/S.  
The Structural Engineer  
1977 | To describe the construction of large concrete gravity platforms by Sir Robert McAlpine and Co. Ltd. | Although large in size, no new technological barriers have been encountered. | Useful paper in that much interesting information has been presented. |
| Derrington JA  
Construction of McAlpine/Sea Tank gravity platforms at Ardyne Point, Argyll.  
ICE. Design and Construction of Offshore Structures  
1976 | As above. | As above. | As above. |
| Doris do Brazil CG  
Design, construction principles and setting of one type of concrete gravity platform installed on oil fields in the North Sea.  
Offshore Structures Conference  
1977 | Description of Doris type platform including perforated breakwater walls. | Minimised installation time once the platform had reached its final location as it could be towed virtually complete. | Interesting paper on the design/construction/installation of a further design type. |
| Lindgren J  
Condeep construction.  
FIANC Norwegian Section Meeting  
1977 | To describe the construction and quality control systems used in the construction of the Condeep type platform. | The paper does not draw particular conclusions. | Interesting descriptive paper on both construction and quality control. |
Table 27
Key references for construction topics (sheet 2 of 2)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>McLeod J I.</td>
<td>To describe the construction of one of the largest platforms built at that time. Both slip forming and precasting techniques used.</td>
<td>While a gravity platform may take longer and cost more before it is installed on location, the remaining installation work is significantly less than for a steel structure.</td>
<td>Interesting descriptive paper which includes comments on use of precast elements.</td>
</tr>
<tr>
<td>The Construction of the Ninian Central Platform for the British North Sea.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Offshore Technology Conference.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1979.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>In-Service Experience with Eleven Offshore Concrete Structures.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Offshore Technology Conference.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1982.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
By 1985, eighteen concrete platforms had been constructed and installed in the North Sea of which ten were in the UK sector. It is clear from various published papers that all of these have been constructed in much the same way. First the base slab cell walls were constructed up to an intermediate level in a dry dock. The structure was then ballasted with water to counteract any possible buoyancy once the dry dock had been flooded and become tidal. With the structure thus stabilized the dry dock was flooded and the sea wall removed. The structure was then deballasted and towed-out to a deep water site where the cells were completed. Slipforming of the towers then commenced and once completed the whole structure was ballasted down once again to allow deck hook-up. This procedure is of course generalized and it is important to realize that many other operations proceed concurrently.

A properly constructed structure has the advantage of cutting an operator's long term costs in the field of in-service inspection, maintenance and repair. It is thus essential that throughout the construction phase the contractor instigates a clear and workable Quality Assurance system in particular with regard to the keeping of records and drawings. As part of such a system, the contractor must pay particular attention to the maintenance of a quality control procedure for the materials he uses and for his workmanship. The working life of the structure can be said to depend on the consistent production of sound, dense concrete which has been accurately placed.

As part of the normal inspection requirements of construction with concrete, special attention is required for the following items:

- The maintenance of section thickness and location during slipforming, and the added problem of maintaining tolerances during construction afloat. Accurate surveying methods must be established to ensure tolerances are maintained and where problems are encountered these should be referred back to the design team.

- Research into cover requirements and its performance in resisting or limiting corrosion has raised considerable discussion especially within the Concrete-in-the-Oceans programme. Long term costs from in-service inspection, maintenance and repair can be reduced if, at the construction stage, the materials and workmanship are to a high standard. However, there has been no modern research into practical aspects of maintaining cover, of which there are two basic methods; these are the use of mortar or concrete blocks, and plastic spacers. Although there are advantages and disadvantages of both types, plastic spacers are probably better as they are more likely to be of a known consistency.

- Curing of the concrete is vital for the long term serviceability of the structure. Recent research by Taylor Woodrow (1987) has indicated the importance of curing on the corrosion of embedded reinforcement. The codes of practice such as BS6235 and FIP 4th Edition, recommend that concrete should be cured with fresh water whenever possible otherwise a heavy duty membrane curing compound should be used. Sea water should not be used for curing.

- The work carried out by Wilkins et al (1980 and 1987) has highlighted the need to prevent any electrical connection between embedded reinforcement and exposed steelwork. This bonding can be prevented by appropriate coatings and fixing methods.
- Any defect noted during construction should be repaired as soon as is practicable. However, prior to such work a detailed procedure should be established and, where required, a test procedure should be carried out. Epoxy resin systems suitable for use in a damp or wet environment may be used, although it is suspected that cement based systems may provide the most durable long term repair.

- Construction joints need careful preparation whenever the structure is to remain watertight or contain oil and particularly with reference to the demands of slipforming.

- Complex pumping and piping arrangements are required within the structure to enable ballasting/deballasting to be carried out at various stages in the construction. Whilst the specification and maintenance of access to the various pumps, valves and piping is primarily a design problem, the importance of accurate construction cannot be over emphasized in this essential part of the system.

For these construction topics, the extent of Guidance required is minimal. The majority of topics are covered, in accordance with the latest thinking in the industry, by existing codes and standards.
A key element in the successful construction and installation of a structure is planning. Matters which should be considered include:

- A detailed survey of the intended towage route, lay-by areas (for storm protection) and the final location are essential to the safe installation of the structure. Particular attention must be paid to areas where the sea way is restricted due to width or depth.

- Model testing may be necessary to determine both the handling characteristics of the structure under tow and the towing horsepower required.

- As part of the design process, calculations must be carried out to ensure proper stability of the structure during tow-out. This stability however, will be influenced by actual constructional tolerances achieved and should be reassessed as part of the normal weight control procedures at all phases of the installation procedure.

### 3.3.2 Inspection and testing during construction

The experience gained in the construction of concrete platforms for the North Sea has indicated the importance of Quality Assurance in achieving fitness for purpose. As part of this procedure it is essential that the contractor maintains a sophisticated Quality Control system to ensure that both materials and workmanship are of the highest order.

It is the magnitude of the structure that represents the challenge rather than the type of work involved, which is not technologically unusual. Existing standards of construction as outlined in BS 8110 are applicable although certain special considerations, particularly appropriate to offshore structures, need to be considered.

A few papers have been published on inspection and testing during construction and appropriate key references are given in Table 28. Moksnes (1982) has described the Quality Control system used by Norwegian Contractors in the construction of six platforms at Stavanger. He has indicated that the Quality Control organisation should at all stages, maintain liaison with the design and construction engineers and also:

- be independent from the production section
- scrutinise and interpret the drawings and specification
- evaluate materials and plant
- prepare construction procedures
- carry out materials testing
- inspect and approve formwork, reinforcement, geometry, etc.
- supervise concrete production, placing and curing
The various Certifying Authorities all require that detailed documentation for quality assurance and control should be submitted. The Quality Assurance programmes should include all materials, manufacturing and construction processes and the Quality Control details should include acceptance and rejection standards. These Certifying Authority requirements provide perhaps, the appropriate basis for the Guidance Notes.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Objective</th>
<th>Conclusion</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Lindgren J  
Condeep construction.  
PIANC Norwegian Section  
1977 | To describe the construction and quality control systems used in the construction of the Condeep type platform. | The paper does not draw particular conclusions except that quality control systems have to be rigorous. | Provides an insight into how one particular contractor set up a quality control system. |
| Mosknes J  
Quality assurance for concrete platforms in North Sea oil fields.  
Concrete International  
1982 | To describe the quality control system established in the construction of concrete platforms. | The quality control requirements of large offshore platforms far exceed those with which most contractors are normally accustomed. | Very significant comments and description of the type of quality control and assurance required. |
| British Petroleum  
Quality assurance in BP.  
April 1984 | To describe quality assurance procedures used in British petroleum. | The paper does not draw any particular conclusions but does describe the procedures expected within British Petroleum. | Useful document describing British Petroleum's quality assurance procedures. Also useful in explaining basics. |
| Hobson Lj  
Quality assurance in offshore projects.  
Solus Ocean Systems  
1977 | To define quality assurance requirements and procedures in relation to offshore projects. | Quality assurance in offshore projects requires special consideration as an offshore project is normally a one-off event. | Useful discussion of quality assurance requirements and procedures. |
4. BACKGROUND TO DRAFT GUIDANCE NOTES

4.1 OUTLINE CONCEPT AND FORM

The Guidance Notes are required to establish the procedures and technical standards required for offshore structures to be certified as being fit for their purpose. This has been taken, for the concrete sections, as being the minimum requirements for safety with clauses as brief as possible compatible with satisfying their purpose.

In order to fit into the overall framework of the Guidance Notes, all the necessary requirements for concrete structures have had to be included into three separate sections covering Design, Materials and Construction. This has led to some difficulties in appropriate location and arrangement of clauses.

As a general policy, guidance has been restricted to requirements for the design, and the owner has been left with discretion as to how this requirement can be met. This report provides suggestions of appropriate methods that could be used but these are put forward as design aids rather than Guidance requirements.

4.2 COMMENTARY ON CLAUSES

4.2.1 Section 12: Corrosion Protection

Clause 12.2.2 Corrosion zones (concrete structures) - This clause is similar to section 4.2.3.12.(4) of the existing Guidance Notes. Other codes such as FIP and ACI define exposure zones in the same manner, although not in the same detail. However, some if not all of these other documents define these zones for both external and internal areas. For the majority of structures it is felt no such distinction is appropriate.

Experience and research has indicated that corrosion is more likely to occur in the splash and atmospheric zones than in the submerged zones. Hence differing requirements for each zone are given in the Guidance Notes. However, the proportion of the structure in each zone is important as the splash and atmospheric zones may form a cathode to the submerged zone anode and, where this cathode is significant in relation to the anode, it may derive a corrosion cell giving rise to measurable corrosion of embedded reinforcement. Earlier Figure 15 indicates the differing proportions for two structures.

Fidjestol et al (1987), in presenting an overview of the criteria for crack and corrosion control in offshore structures, has defined a 'permanently wet' and 'interactive' zone and provides guidance on the effects of differing ratios of the two zones. Carney et al (1987), in their work on visual indications of corrosion, have also discussed these zones as well as the hollow leg effect. It should be noted that conclusions drawn for corrosion in the submerged zone assume a very large proportion of the structure is below water and that, therefore, they are not necessarily applicable to all of the zone, as defined in these Guidance Notes.

Clause 12.3.4 Cathodic protection (concrete structures) - This clause had been revised, by others, prior to the overall revision of the concrete clauses but nevertheless it was felt that further revision was justified.
As a result of concerns over dynamic cracking (see p 117) a recommendation has been made to cathodically protect the steel reinforcement and this is now incorporated in the Fourth Edition Guidance (Amend No 2). Research by Wilkins et al (1987) as part of the Concrete-in-the-Oceans programme has identified possible corrosion of reinforcement in the 'negative active' region, at potentials of -800mV or more negative. This supports an earlier report of pitting in this potential region (King et al 1977) and further evidence has recently been provided by Hildebrand et al (1983) of pitting on cathodically protected steel in cement mortar in seawater. In view of this evidence, a possible need to polarise systems containing steel embedded in concrete to a rather more negative potential than that normally accepted for bare steel in seawater should be considered.

4.2.2 Section 23: Concrete

Clause 23.1 Fixed concrete installations - This provides a definition of the type of structure covered by this section of the Guidance Notes. The requirements for mobile concrete installations, covered in Clause 30.2.3, will for many requirements be substantially the same as for fixed installations but nevertheless this clause is drafted specifically only for fixed installations.

As an integral part of their construction process, fixed structures, as defined, will be afloat for periods. The requirements for their temporary condition should be as put forward in Clause 23.2.5. The existing Guidance Notes require that, under such temporary conditions, the installations should not be exposed to conditions, loads or stresses more severe than those envisaged for the permanent condition. However, for many structures, the most severe loading, particularly hydrostatic, is during deck mating and installation. Nevertheless, the owner should exercise caution in allowing temporary stress levels higher than those that will be induced in the permanent conditions.

Clause 23.2: Design and detailing

Clause 23.2.1 Standards - The design sections for concrete structures are based on the use of BS 8110 : 1985 The Structural use of Concrete’ for the reasons discussed in Section 2.4. Under the general clauses of the Guidance Notes, owners are free to propose alternative recognised codes of practice and this clause is not intended to remove that option.

Clause 23.2.2 Limit state design - All codes relating to concrete offshore structures recommend that the Limit State Design approach should be used. This requirement is continued in the Guidance Notes in line with present practice and developments.

Clause 23.2.3 Limit states - In the Limit State Method the owner's object is to design the structure to give acceptable probabilities that it will not reach either of the two principal Limit States: the Ultimate Limit State and the Serviceability Limit State. In designing to the Ultimate Limit State, the owner is looking to ensure that the structure does not become unfit for its purpose by collapse, overturning, or buckling (i.e. it does not suffer failure) and to the Serviceability Limit State by deformation, cracking, vibration etc. (i.e. it does not become unserviceable).

The FIP Recommendations define further Limit States, partly as separate States and partly as sub-divisions of the two main States. This, however, does not affect the basic concept of Ultimate and Serviceability Limit States but perhaps adds a degree of refinement.
Throughout the revised Guidance Notes, guidance is put forward suggesting against which Limit States any particular load or stress condition should be considered. These are determined subjectively and for any particular structure may require revision particularly where drawdown is utilised.

Clause 23.2.4 Loads - In using the Limit State Method for the design of concrete offshore structures all loads whether dead, imposed, hydrostatic or environmental should be the characteristic value of the load. To take into account loading effects on the structure, a partial safety factor, the Load Factor, is introduced as defined in BS 8110 Clause 2.4.1.3.

The loads and load combinations must be considered in conjunction with the load factors to be adopted. The load factors quoted herein have been taken from BS 6235 (now withdrawn); Table 29 gives a comparison of the loading requirements for that Code and the Guidance Notes.

It has been argued that, for present designs of concrete platforms there is no merit in retaining the concept of normal operating conditions. The maximum stresses on the structure may be due to extreme environmental loads and, in accordance with guidance, the operating loads are defined by the operator. Such an arrangement has been retained as it is believed to provide a proper basis for design whereby, in selecting his operating criteria, the operator is potentially incurring a penalty.

Various standards and codes of practice give recommended values for Load Factors for particular load categories. In certain circumstances, it may be appropriate to adjust these factors because specific data is available for the structure under consideration which will allow (or require) such adjustments to be made, for example, during deck mating when the hydrostatic load on immersed cells may be accurately controlled, or because an as-built survey has provided more accurate structural dimensions.

The justification for any changes to the factors should ideally be based on reliability principles but, for all such circumstances, it may be unrealistic to carry out the necessary analysis. However, the owner should be careful in making value judgements on adjustments to factors and should make every effort to ensure that necessary levels of safety are maintained.

The use of Reliability Theory to determine Load Factors has been shown theoretically to be feasible for offshore structures. Particular caution though should be taken before applying this theory as there is little experience in the use of reliability indices in the design of concrete structures generally. Flint & Baker (1976) determined a Reliability Index value of $10^{-4}$ per annum for offshore structures.

Clause 23.2.5 Load factors for the ultimate limit state - As discussed in Section 3.1.1 of this report, all present codes give similar load factors derived by industry consensus and experience. A comparison of the Factors from the various codes is given in Table 30. This clause is taken from BS6235 (now withdrawn), as this is felt to provide the most appropriate model.

For extreme environmental conditions, two separate load cases for maximum and minimum imposed loads are given. These are to identify the maximum (ie. maximum compression) and minimum (ie maximum tension) stresses induced in the structure.
<table>
<thead>
<tr>
<th>Load Condition</th>
<th>BS 6235 (now withdrawn)</th>
<th>Guidance Notes (3rd edition 1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead loads</td>
<td>Weight of structure in air</td>
<td>as BS 6235</td>
</tr>
<tr>
<td>Imposed loads</td>
<td>All loads except dead, hydrostatic and environmental loads</td>
<td>as BS 6235</td>
</tr>
<tr>
<td>Environmental Loads:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• combination</td>
<td>Not less severe than individual loads specified</td>
<td>Not defined</td>
</tr>
<tr>
<td>• normal environmental conditions</td>
<td>Wave climate corresponding to densest part of the wave scatter diagram</td>
<td>Operator to define</td>
</tr>
<tr>
<td>• max annual tides</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• storm surge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• 1 minute mean wind speed with 1 month recurrence</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• currents</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• lowest air and sea temperatures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• marine growth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• snow and ice</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• temporary environmental condition</td>
<td>10 year wave</td>
<td>Not specified</td>
</tr>
<tr>
<td>• 1 minute mean wind speed of 10 year recurrence period</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• highest current anticipated during towout</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• lowest air and sea temperatures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• snow and ice</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• extreme conditions</td>
<td>50 year wave</td>
<td></td>
</tr>
<tr>
<td>• highest annual tides plus surges</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• 1 minute mean wind speed of 50 year wind</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• maximum current</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• lowest air and sea temperature</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• marine growth</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 30
Comparison of ultimate limit state loading factors

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Normal operating</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dead Load</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>or</td>
<td></td>
<td>or</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td></td>
<td>1.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Imposed Load</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
<td>1.6</td>
<td>1.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Hydrostatic Load</td>
<td>0.9</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
</tr>
<tr>
<td></td>
<td>or</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Environmental Load</td>
<td>1.4</td>
<td>1.4</td>
<td>1.0</td>
<td>1.3</td>
<td>0.7</td>
<td>1.4</td>
</tr>
<tr>
<td>2. Extreme environmental</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dead Load</td>
<td>1.2</td>
<td>1.1</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>or</td>
<td></td>
<td>or</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td></td>
<td>1.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Imposed Load</td>
<td>1.2</td>
<td>1.3</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td>1.3</td>
</tr>
<tr>
<td>Hydrostatic Load</td>
<td>0.9</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
</tr>
<tr>
<td></td>
<td>or</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Environmental Load</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
</tbody>
</table>

*1 Hydrostatic loading effect included within dead load category
*2 Values for Lloyds taken from FIP
*3 Now withdrawn

**Clause 23.2.6 Load factors for the serviceability limit state** - A comparison of load factors for the Serviceability Limit State is given in Table 31 where it can be seen that all codes agree on appropriate values. This clause has thus been written recommending these values from BS6235 (now withdrawn). The accompanying text from BS6235 (now withdrawn) has also been included to clarify the design loads required in using the Serviceability Limit State, except that the last paragraph of BS 6235 (now withdrawn), concerned with special considerations for deflection, has been omitted since the necessary technical points are covered elsewhere in Guidance.
Table 31
Comparison of serviceability limit state loading factors

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Imposed Load</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Hydrostatic Load</td>
<td>1.0</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
<td>*1</td>
</tr>
<tr>
<td>Environmental Load</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

*1 Hydrostatic loading effect included within dead load category.
*2 Values for Lloyds taken from FIP
*3 New withdrawn

Clause 23.2.7 Strength of materials - This clause is taken from BS 8110 Clauses 2.4.2, 2.4.4 and 2.4.6. All other codes define characteristic strength, design strength and materials factors in the same manner as BS 8110 and give identical recommended values for the material factors.

The level of Quality Assurance used for the present generation of concrete platforms has led to better site control of concrete mix than is traditional in the construction industry. In these circumstances it seems appropriate to allow for reduced values providing the owner can demonstrate that with the cements, aggregates and facilities available for the project the necessary consistency of mix can be achieved. Such a reduction implies a rigorous sampling and testing regime for the concrete.

Clause 23.2.8 Shear - The shear clauses in BS 8110, for both reinforced and prestressed concrete, are based on a shear failure mechanism such as for beams, utilising arching action and truss action with the effect of dowel resistance, aggregate interlock, etc. taken into account. In complex structural elements which occur in offshore structures the shear failure mechanism is usually not clear. This is often the case in thick shell type structures with diaphragms where the resultant redistribution of stress after cracking has occurred is even more complex and difficult to assess, unless a sophisticated non-linear method of analysis is attempted. This, however, is not readily feasible. Hence, for such cases it is only realistic to ensure that the principal tensile stress is within the admissible limits under Service and Ultimate Limit States. It is thus recommended that in calculating the principal tensile stress the Ultimate Limit State factors are used.

However, if such recommendations were applied to less complex structural elements, although they may not strictly comply with BS 8110 beam type structural elements, it may be unnecessarily conservative and result in an uneconomical design. For such cases, if the resultant structural system carrying the applied load after cracking can be identified, it will be more reasonable to calculate principal tensile stresses using the load factors of the Serviceability Limit State to ensure no cracking under working load and to design the resultant structural elements for the Ultimate Limit State. Thus, both the Serviceability and Ultimate Limit States are catered for. The prestressing force, if needed, will be a minimum.
Clause 23.2.9 Fatigue - The clause draws attention to the need to check fatigue resistance at sudden changes in cross-section and at openings. The stresses obtained from a coarse finite element analysis around such areas might be considered to be subject to additional stress concentration factors. However as reinforced concrete is a relatively "notch insensitive" material it seems that very localised concrete distress could be accommodated without leading to any general failure in the area. Little research information is currently available on this. Some further discussion is included in the Concrete-in-the-Oceans report by Gifford and Partners (1987).

The use of load factors of unity means that when fatigue life is evaluated on the basis of cumulative damage, the only factor of safety included is the partial factor for materials incorporated in the design S-N curves. In comparing the calculated fatigue life with the design life the owner must judge whether or not the available margin of safety is adequate.

The conditions (a) - (c) allowing fatigue to be discounted at a particular location relate to reinforcement stress ranges, concrete stresses and detailing aspects. They apply to structures in which the cyclic loading is primarily caused by wave action.

The limits on tensile stress range in reinforcement apply to the greatest loading and further to a loading which can be exceeded one million times. In calculating the tensile stress ranges, excursions into the compressive range are excluded. The Concrete-in-the-Oceans programme demonstrated that fatigue failures can occur at actual stress ranges as low as 70 N/sq.mm.

Condition (b) relating to concrete stresses applies essentially to concrete subject to small tensile stresses not causing cracking and to compressive stresses no greater than the normal design limits of the concrete. If the concrete is cracked and is subject to repeated crack opening when submerged then the compression fatigue strength is likely to be reduced. The compression limit of 0.33f_{cu} is the service stress corresponding to a maximum ultimate stress of 0.44f_{cu} divided by a load factor of 1.3.

If cumulative damage assessment is carried out it should include both high stress ranges of only isolated occurrence as well as low stress ranges with a very high number of occurrences.

The cumulative damage calculations are normally based on Miner's hypothesis which is of the form:

\[ \sum \frac{n_i}{N_i} = M \]

where: \( n_i \) = number of occurrences of a particular stress range i.
\( N_i \) = number of occurrences of stress range i required to produce failure (obtained from the design S-N curve).
M = Miner's Number.

For steel it is usual practice to adopt a Miner's number of unity. For concrete however, experimental work by Van Leeuwen and Siemnes (1979) shows that Miner's hypothesis gives widely scattered results with failure occurring at mean values of calculated cumulative damage generally well below unity. They concluded that variable amplitude loadings are more damaging than the constant amplitude loading tests used to plot S-N
curves. As a lower bound for use in design calculations it is suggested that a value of Miner's number for concrete of 0.2 be used.

In calculating cumulative damage in reinforcement it is recommended that the design S-N curve from the Concrete-in-the-Oceans programme for reinforcement embedded in partly submerged concrete be used. This curve is given in Figure 13 and is expressed by the following formulae:

Stress ranges between 400N/sq.mm and 235N/sq.mm

\[
\log S = 3.27 - \frac{\log N}{6}
\]

Stress ranges between 235N/sq.mm and 65N/sq.mm

\[
\log S = 4.30 - \frac{\log N}{2.8}
\]

Stress ranges below 65N/sq.mm

\[
\log S = 3.26 - \frac{\log N}{4.8}
\]

In calculating the above tensile stress ranges excursions into the compression region are discounted. The effect of mean stress at normal design stress levels is secondary and it is suggested that it may be ignored in cumulative damage calculations for the reinforcement.

![Figure 13](image)

Figure 13
Design S-N curve for reinforcement

The curve was derived from tests on 32mm diameter Torbar. Its use for smaller diameter bars would be slightly conservative. For fully submerged conditions the curve may also be conservative.
For concrete design S-N curves for compression/compression cycling have been put forward by Waagaard of DnV (1986) and by TNO. They are shown for a typical case in Figure 14. The DnV design curve is governed by the equation:

\[ \log_{10} N = 10 - \frac{1 - S_{\text{max}} / (\alpha \cdot f'_c / \gamma_m)}{1 - S_{\text{min}} / (\alpha \cdot f'_c / \gamma_m)} \]

where: 
- \( N \) = number of cycles to failure under constant amplitude loading from \( S_{\text{min}} \) to \( S_{\text{max}} \).
- \( S_{\text{max}} \) = maximum stress
- \( S_{\text{min}} \) = minimum stress
- \( f'_c \) = concrete cylinder strength
- \( \gamma_m \) = partial safety factor for concrete = 1.25
- \( \alpha \) = flexural gradient coefficient

\( (\alpha = 1.19 - \frac{\varepsilon_{\text{min}}}{\varepsilon_{\text{max}}} \times 0.19 \) where \( \varepsilon_{\text{min}} \) and \( \varepsilon_{\text{max}} \) are simultaneous extreme fibre strains).

It should be noted that different values of \( \alpha \) have been proposed in different codes and therefore care needs to be taken in the selection of \( \alpha \) for design purposes.

Cumulative damage calculations based on the two curves will vary widely. Normal reinforced concrete structures operate at concrete stresses rarely reaching one third of the static compressive strength. At these stress levels virtually no cumulative damage at all is predicted by the TNO curve which has an endurance limit.

The DnV curve on the other hand predicts finite, though long, lives even at very low stress range levels. It is at these low amplitude high cycle conditions that the greatest contributions to cumulative damage are calculated. The high cycle end of the S-N curve is determined by extrapolation and results obtained from its use must be treated with caution. Indeed experimental work under the Concrete-in-the-Oceans programme indicated that during tension/compression cycling with compressive stresses below 6N/sq.mm crack blocking may occur, improving the fatigue life substantially. The two design curves lead to cumulative damage results which can differ by up to three orders of magnitude depending on the assumptions made. The DnV curve is backed by recent research work and its use in design will give safe, though probably conservative, results.

In the Fatigue Calibration Study (Offshore Certification Bureau 1986) the use of the DnV curve was not found to predict unreasonably short fatigue lives for existing structures. By extending the Goodman diagram into the tension range it is possible to include tension failure usage, in the cumulative damage calculations. The purpose of this is to eliminate any cracking in the concrete under fatigue and the results obtained are quite different to those based only on compression usage where cracking under tension is assumed. It noted that this method was different from that proposed by Waagaard as identified in the following paragraph. The TNO curve has been used previously relative to a static strength or cube strength x 0.67 divided by a partial safety factor on concrete of 1.5 which leads to a lower design static strength than used in the DnV equation. In order to use the TNO curve for minimum stresses other than zero, use is made of a Goodman diagram. Both the DnV and the TNO curve apply to submerged concrete.
Design S-N curves for submerged concrete with characteristic strength = 50N/mm².

Design stress strength:

\[ \text{Design strength} = \frac{\text{Ultimate strength}}{k} \times \text{Subm. strength} \times 0.67 \times \frac{1}{k_e} = 1.50 \]

\[ = \frac{60 - 6}{1.25} \times 26.04/\text{mm}^2 \times 0.67 \times 0.6 \times 1.50 = 22.30/\text{mm}^2. \]

---

**Figure 14**

Typical design S-N curves for submerged concrete

(for \( f_{cu} = 50\text{N/sq.mm} \))
The above design S-N curves are for compression/compression cycling. However the preliminary checks on concrete in the fatigue clause are more likely to be exceeded on the grounds of high tensile stresses than high compression stresses as the compression limit is essentially the same as the static design compression limit. Waagaard (1986) has proposed a design S-N curve for compression tension cycling of pre-cracked reinforced concrete submerged in water. This curve is also shown in Figure 14. It is governed by the equation:

$$\log_{10} N = 8.0 \left( 1 - \frac{S_{\text{max}}}{\alpha (f_c' / \gamma_m)} \right)$$

where symbols are as before.

Due to pumping of water in cracks opening and closing, Waagaard has observed a marked reduction in the compression fatigue life of concrete used together with reinforcement.

The clause contains some recommendations for detailing in areas of significant cyclic loading and reference should be made to the technical appraisal paper for the source of these recommendations.

When considering transverse shear the American Concrete Institute states that where the cyclic range is more than half the design ultimate shear resistance of the concrete alone, then all transverse shear should be taken by reinforcement.

Further background information is contained in 'Fatigue Calibration Study', Offshore Certification Bureau (1986).

**It is concluded that unless a body of data obtained under realistic conditions is available, DNV S/N curves for concrete should be used for design purposes.**

**Clause 23.2.10 Deflection** - This clause requires the owner to ensure that deflections will be acceptable and provides guidance on the limits to be adopted by reference to BS 8110. Sections 3 and 4 of BS 8110 give limits for individual member types whether reinforced or prestressed. However, these requirements must also be supplemented by any more specific requirements for offshore structures. The FIP Recommendations give additional requirements in connection with well heads and conductor tube assemblies. The effects of motion or vibration may make it appropriate to isolate parts of the structure (e.g. dormitories or radio room.)

**Clause 23.2.11 Cracking** - (See also section 3.1.2 Cracking/cover/durability). This clause considers methods of assessment and limits to structural cracking, mainly under static loads. Cracking for non-structural reasons, including plastic settlement which may be of more importance in determining durability, should be limited by selection of appropriate mix properties and proper construction techniques. Remedial action may be appropriate where such cracking exceeds the limits suggested for structural cracking. Various aspects of non-structural cracking are discussed in Concrete Society Technical Report No. 22.

Previous codes and standards, including previous Guidance Notes, have required that checking of crack widths be carried out and have laid down limits for such cracking. As discussed in section 3.1.2 of this report, recent research (Beeby 1978, Wilkins et al 1987, Stillwell 1987, Fidjestol 1987) suggests that, for all zones, crack widths have little if any
effect on long term corrosion of the reinforcement although they will influence the short term initiation of corrosion. However, this research has considered only crack widths, cover and concrete mixes broadly within the range of values presently associated with 'good practice' and has not extended test specimens to extreme values.

![Diagram of typical concrete gravity platform and shallow water structure]

**Figure 15**
Definition of exposure zones for typical structures

It is felt therefore that, whilst more relaxed criteria may be appropriate, owners should show caution in abandoning crack width limitations and corresponding calculations.

Crack width calculations are not required for areas of structures where the embedded steel is covered by an effective cathodic protection system. However see the recommendation for part of the submerged zone on page 117.

The research has demonstrated that cracks longitudinal to the reinforcement are more critical than transverse cracks. The exposure tests carried out by Stillwell (1987) indicated corrosion after 5 years at crack positions in the splash zone with crack widths of 1mm to 0.1mm. The findings for the submerged zone (Wilkins et al 1987, Stillwell 1987, Fidjestol 1987) demonstrate that without external anodic polarisation, corrosion will not occur at cracks even up to 2mm wide. The required polarisation, though, may be caused by the development of a large cathode in the splash and atmospheric zones possibly leading to localised corrosion.
Lenschow (1979) reports the results of tests by others, most notably Schliessl (1976), indicating an increasing probability for serious corrosion where crack widths approach 0.5mm. Significant corrosion is defined as more than 4% loss of section and this conclusion is suggested for all zones other than submerged. The test results quoted indicate very little change in corrosion with cracks of less than 0.5mm and particularly in the typically adopted range of 0.1 to 0.3mm.

For the submerged zone, there may be benefit in having a minimum amount of cracking to predispose that there is a general distribution of cracks of significant size. The total amount of any corrosion is governed by the oxygen supply which, in the submerged zone, is limited to the dissolved oxygen in the seawater. A general distributed crack pattern will make it likely that any corrosion that takes place will be evenly distributed and, conversely, a concentration of cracks will lead to a concentration of corrosion in one area.

There are several methods or formulae for prediction of crack widths and, of these, the CEB-FIP and BS 8110 are most widely used for offshore structures. Both Bceby (1978) and Fidjestol et al (1987) have investigated these methods but draw differing conclusions. The latter recommends a modified CEB-FIP formula whereas the former regards this as 'irrelevant' and recommends the BS 8110 formula. These two formulae are derived from the same basic equations for cracking but have been simplified to calculate different parameters. These may be summarized as:

- **CEB-FIP Formula** - This calculates the characteristic crack over the main reinforcement at the level of the reinforcement. The characteristic crack is taken as that width with a 5% probability of being exceeded and the calculation is carried out at a loading less than the maximum, characteristic load.

- **BS 8110 Formula** - This calculates crack widths, with a 20% probability of being exceeded, at any point on the surface of the member. The calculation is carried out at the characteristic Serviceability Limit State Loading.

There appears to be considerable evidence to indicate that each formula provides a reasonably accurate estimate of what it sets out to calculate. However, they set out to calculate different quantities and the decision as to which formula to use must be based on a decision as to which parameter is required. For offshore structures the primary reason for limiting crack width is corrosion control and since the surface crack problem over the whole member will determine this corrosion the BS 8110 formula is included in Guidance.

Crack widths can be amplified when reinforcement is not parallel to the principal stress. Lenschow (1979) reports tests on reinforced concrete discs with reinforcement at angles of 0°, 22.5° and 45° to the applied tension which indicate an amplification of up to 6 times. Both Lenschow (1979) and Fidjestol et al (1987) put forward formulae for calculating such cracking but in each case they are derivatives based on the CEB-FIP formula. BS 5400: Part 4 suggests that in calculating the effect of the tension stiffening,
$A_s$ should be taken as:

$$A_s = \sum (A_i \cos^4 \alpha_i)$$

where:
- $A_s$ = area of tension reinforcement
- $A_i$ = area of reinforcement in particular direction
- $\alpha_i$ = angle between axis of the applied load or moment and direction of tensile reinforcement, $At$, resulting from that load or moment.

The BS 8110 formula for calculation of crack widths is a simplification and some variables have been ignored as being of minor significance in flexural members. For pure tension, the approximations are likely to be most in error and will lead to an underestimate of crack width. Rowe et al (1972) suggest the following more accurate formula:

$$w = (1.6 + \frac{1.4c}{\phi \sqrt{A/B}}) \cdot a_{cr} \cdot \sigma_m$$

where:
- $w$ = calculated crack width
- $c$ = cover
- $\phi$ = bar diameter
- $A$ = longer side of a prism of concrete which may be assumed to be surrounding a particular bar
- $B$ = as $A$ but shorter side
- $a_{cr}$ = distance from the point considered to the surface of the nearest longitudinal bar
- $\sigma_m$ = average strain at a level where cracking is being considered and calculated allowing for the stiffening effect of the concrete in the tension zone.

The suggested maximum static crack widths are a relaxation generally on those previously put forward particularly for the submerged zone where the recommendation put forward by Fidjestol et al (1987) has been incorporated.

**Dynamic Cracks** - Research carried out by Hodgkiess et al (1987) as part of the Concrete-in-the-Oceans programme has indicated the possible significance of corrosion at dynamic cracks due to cyclic loads in the upper part of the submerged zone. The conclusions reached by several workers, that crack width has little effect on corrosion, relate only to static cracks. However the previous design limitations applied to the static crack widths will also have limited the widths of dynamic cracks.

In view of the conflicting evidence from research and the different conclusions that have been drawn from the results, it has not been possible to draw up design rules for the case of cracks under dynamic loading. It is therefore suggested that sections in the upper region of the submerged zone should be designed to remain in compression under loads which are frequently applied. This requirement can be waived for those members where localised corrosion of the reinforcement is likely to have minimal effect on the overall integrity of the structure or where the embedded steel is covered by an effective cathodic protection system. Subsequent to the publication of the Fourth edition Guidance Notes, the Department of energy, in a letter dated 25 January 1990, has circulated a recommendation that all concrete structures should be protected by a cathodic protection system in order to avoid localised corrosion.

All other codes and standards put forward separate criteria for pre-stressed and
reinforced concrete although there appears to be no basis for anticipating greater corrosion on prestressing tendons than reinforcing bar nor necessarily for supposing that the consequences of any corrosion will be more detrimental to the overall structural integrity. For post-tensioned members the methods and adequacy of the duct grouting will be of far greater consequence than crack widths, cover or concrete quality and for these reasons, separate criteria have not been given. Where the owner believes that proper grouting of the ducts cannot be carried out, consideration should be given to the use of reinforced rather than prestressed concrete.

Clause 23.2.12 Cover - Cover is required to provide corrosion protection to the reinforcement, and concrete around the bars (or tendons) for bond and the overall structural integrity of the member. Investigations in the Concrete-in-the-Oceans programme (Wilkins et al 1987, Stillwell 1987, Fidjestol et al 1987) suggest that the amount of corrosion may be independent of cover thickness; concrete surface area in relation to bar size may be more important. The quality of the concrete forming the cover will be of more significance than the amount of cover.

The parameters put forward of 1.5 times the nominal maximum aggregate size or 1.5 times the maximum bar size are intended to ensure that

- sufficient space is allowed around the bars for proper placing and compaction of the concrete forming the cover.

- proper bond of the reinforcement is achieved.

BS 8110 suggests that cover equivalent to the maximum bar or aggregate size is required rather than the 1.5 times required by the previous Guidance Notes. This latter figure is used by other offshore codes and has, hence, been retained in the Guidance Notes but the owner may consider in certain circumstances that it is inappropriate.

For crack width parameters, as discussed in connection with clause 23.2.11 of the Guidance Notes, separate requirements for reinforced and prestressed concrete have not been retained. Similarly for cover, only one set of requirements is given which applies equally to both prestressed and reinforced concrete.

Table 32 gives a comparison of the cover requirements for other codes and standards. The nominal covers put forward in Table 1 of the Guidance Notes were derived, by inspection, from this comparison bearing in mind the following concepts:

- greatest cover is required in the splash zone and least cover in the submerged zone.

- similar cover is required for both the internal and external zones.

- allowance for construction tolerances to achieve an acceptable minimum cover.
Research has been undertaken to allow the cover thickness to be designed to prevent sufficient chloride ingress to achieve activation of the reinforcement within the design life of the structure. Taylor Woodrow (1985) has proposed a root time law for chloride ingress but this is based upon data for uncracked specimens.

**Table 32**  
**Comparison of cover requirements**

<table>
<thead>
<tr>
<th>Nominal cover to reinforcement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FIP DoE DnV Lloyds 3rd Ed. 4th Ed. CP 100 BS 6235 BS 8110 ACI 8110</td>
</tr>
</tbody>
</table>

**Reinforcing bars**

(a) Submerged: 60 50 60 60 50 60 60 50 60
(b) Splash: 75 50 75 75 75 60 75 65 60
(c) Atmospheric: 75 40 75 75 75 60 75 50 60

**Prestressing tendons**

(a) Submerged: 75 100 75 75 65 60 75 75 60
(b) Splash: 100 100 100 100 90 60 100 90 60
(c) Atmospheric: 100 80 100 100 90 60 100 75 60

*1 Now withdrawn

Figure 16 indicates calculated estimates of activation times for chlorides, and calibration for differing concrete grades and cover thicknesses as proposed by Browne (1986). The proposed nominal covers will be, for the majority of structures in the North Sea, greater than suggested by Browne (1986).
Figure 16
Times for corrosion activation (after Browne 1986)

Data on actual cover specified for existing structures in the UK sector of the North Sea have been obtained as part of the Guidance Notes revision assignment. These are given in Table 33.

In assessing cover requirements the owner should consider the requirements for any surface treatments to be applied, and fire resistance. Where a member due to its particular situation is required to resist the action of fire, the nominal cover may need to be increased or, alternatively, the concrete cover to the main bars may need to be reinforced to prevent premature spalling.

The owner may consider reductions in cover requirements and perhaps increased crack width requirements in conjunction with the use of coated reinforcement. Recent research data, regrettably not published at the time of writing this report, indicates, it is understood, that the use of non-inhibiting barrier coatings may lead to accelerated surface corrosion or under coating corrosion. The use of any reinforcement coating should be approached with caution.
### Table 33
**Specified cover for existing installations**

<table>
<thead>
<tr>
<th>Platform</th>
<th>Nominal cover (mm)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Submerged zone</td>
<td>Splash zone</td>
</tr>
<tr>
<td>A</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>B</td>
<td>reinforcement 60</td>
<td>100 or 75 depending upon member</td>
</tr>
<tr>
<td></td>
<td>prestress 75</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>generally 70 in all zones</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>60</td>
<td>100</td>
</tr>
<tr>
<td>E</td>
<td>70</td>
<td>70</td>
</tr>
<tr>
<td>F</td>
<td>reinforcement 50</td>
<td>data not available</td>
</tr>
<tr>
<td></td>
<td>prestress</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>generally 70 in all zones</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>65</td>
<td>65</td>
</tr>
</tbody>
</table>

**Clause 23.2.13 Reinforcement detailing** - This clause was included for completeness but no special requirements were intended.

**Clause 23.2.14 Minimum reinforcement** - A minimum reinforcement quantity is required to ensure an even distribution of small cracks rather than the formation of single larger cracks. Leeming (1987), based upon a compilation of research data, proposes that a total minimum steel area of 0.6% of the section area should be adopted. The formula put forward for the minimum area of reinforcement does not include for any additional requirements due to water pressure in cracks. DNV (1977) recommends that such an allowance be included and, for structures in 350 meter water depths, this may prove to be a controlling factor.

**Clause 23.2.15 Drawdown** - Experience with existing structures in the North Sea has indicated problems with the use of drawdown. Instances have occurred where damage due to both ship impact and dropped objects has caused loss of drawdown sufficient to mean that the structure, temporarily, would theoretically be unable to withstand the full environmental loading.

Drawdown could also be lost due to leakage (e.g. at construction joints) into the structure and the presence of drawdown may make maintenance and/or repair difficult or impossible. Particular attention should be paid to pumps and pipework for which maintenance facilities must be available.
The operation of this clause may well prevent the use of drawdown in designing the structure to meet the Ultimate Limit State requirements. However, Serviceability Limit State requirements associated with temperature effects may be assisted by drawdown but, in this case, loss of drawdown may prevent oil storage and may necessitate a shutdown of the platform.

**Clause 23.2.16 Impact** - The treatment of impact loads is based upon only the analysis of damage due to extreme accidental loads. DnV (1981) in Technical Note TNA 101 specify two levels of loadings; one corresponds to a 'no damage' situation and the other to a 'local failure' case but with no overall failure. Ideally the magnitude of each of these two load cases should be determined following appropriate statistical studies to establish the level of reliability for which DnV suggests $10^{-4}$ as an acceptable basis for design. The difficulties in defining the design loads for each of the two load cases precluded the adoption of this philosophy into the Guidance Notes. However, the owner should consider the use of a lower level of loading than the maximum proposed, as an extreme accident, and ensure that this can be accommodated without permanent damage or deformation.

In checking for global failure, consideration must be given to progressive collapse as a possible failure mode. For present designs of concrete platforms, this mode takes the form of local damage leading to a loss of drawdown leading to instability under extreme environmental loads.

**Impact (dropped objects)** - This sub-section of the Guidance Clause is written by selecting the two types of categories identified in work undertaken by Wimpey (1987). Table 2 is given to show typical parameters for the most onerous case of each of these two categories.

The recommended impact velocities given in Table 2 are based on 35m free fall through air, impact with water and fall 'end-on' in 120m water depth and were calculated by Wimpey (1987). Impact velocities may alternatively be calculated independently using the principles and formulae provided by Wimpey (1987).

Definitions of impact damage are not sufficiently precise and are subject to different interpretation, (Brown et al 1987). In particular, there is no formal classification of damage occurring before seabbing occurs, such as cracking or shear plug formation which may be particularly relevant to leakage. Figure 17 shows the main descriptive terms for impact damage which are intended by the terms used in Guidance.
Figure 17
Forms of impact damage (after Brown and Perry 1987)
Assessment of impact damage in relation to perforation and scabbing may be made using the equations derived by Brown et al (1987):

- **For Penetration** - An estimate of the penetration by solid or pipe dropped objects is given by:

\[
\frac{x}{d_e} = 10N_{pen}
\]

where:

\[
N_{pen} = \frac{m^{0.5} \cdot V}{E^{0.5} \cdot d_e^{1.5}}
\]

\[
x = \text{penetration depth}
\]

\[
d_e = \text{equivalent diameter of the dropped object given by } \frac{4A}{\pi}
\]

\[
A = \text{cross-sectional area of the dropped object.}
\]

\[
m = \text{mass of the dropped object.}
\]

\[
V = \text{impact velocity}
\]

\[
E = \text{Modulus of longitudinal deformation for the concrete.}
\]

For solid cylindrical objects \(d_e = d_o\), the outside diameter. This formula should not be used if \(x/d_e\) is greater than 2.

- **For scabbing and shear plug formation** - An indication of scabbing and shear plug formation for solid and pipe dropped objects is given by \(N_{scab}\), defined as:

\[
N_{scab} = \frac{m^{0.5} \cdot V \cdot d_e^{0.5} \cdot E^{0.5}}{d_o \cdot t \cdot (1 + \frac{t}{d_e}) \cdot f_r}
\]

where:

\[
N_{scab} = \text{dimensionless number}
\]

\[
t = \text{slab thickness}
\]

\[
f_r = \text{nominal shear strength}
\]

In General \(f_r = f_{ctm}\) where \(f_{ctm}\) is mean tensile strength of the concrete.

If \(N_{scab}\) is less than 50, scabbing is very unlikely. Some inclined cracking may have occurred but there should be no gross movement of the shear plug. For solid dropped objects, if \(N_{scab}\) is greater than 70 then scabbing is almost certain to occur. For pipe dropped objects, if \(N_{scab}\) is greater than 60 there is more than an even chance that scabbing will occur.

Rather than design the slab directly to resist the impact, it may be appropriate to incorporate an energy absorbing protective layer to vulnerable parts of the structure such as cell roofs.

**Impact (ship collision)** - The study completed by the Concrete-in-the-Oceans programme (Wimpey 1987), specified several component aspects which should be considered in the appraisal of ship collision. Assessment of collision risk and design impact loading is provided in Section 15 of the Guidance Notes which refers to both steel and concrete structures.
The study by Wimpey (1987) defined several aspects which should be considered in the appraisal of ship collision. Lloyd's (1987), have studied ship impact on steel structures and concluded that the energy level for supply vessel impact should be increased, together with more stringent fendering requirements.

Impact damage arising from forces generated during the collision may give rise to failure by local punching shear or local bending. The following formula derived by Caldwell et al (1981) was proposed to calculate the punching shear strength of a hollow concrete cylinder. This is based on a wide range of tests (Olesen et al 1978, Brakel et al 1979, Regan et al 1981). However the formula should be used with caution as the tests from which it was derived used unrepresentative D/T ratios:

\[ P_{br} = 2.92 ks \cdot kt \cdot kp \cdot hn (a + hn) \sqrt{f_c'} \]

where: 
- \( P_{br} \) = punching shear strength (MN)
- \( hn \) = effective depth of cylinder wall (m)
- \( a \) = 0.25 x loaded perimeter (m)
- \( f_c' \) = compressive strength of concrete from cylinder tests (N/sq.mm)
- \( ks \) = 0.65 + 0.15/(0.12 + hn)
- \( kt \) = 0.5 + 5.0 (t/d)^2 where \((0.05 < (t/d) < 0.10)\)
- \( kp \) = 1.0 + 0.02 \( \sigma_c \)
- \( t \) = cylinder wall thickness
- \( d \) = outside diameter of cylinder
- \( \sigma_c \) = compressive prestress in cylinder walls

Local bending failure may be analysed by suitable non-linear computer programs. Failure due to overall collapse may be considered using conventional structural analysis techniques with respect to Ultimate Limit State.

The design impact loading implies that the structure is not designed for higher levels of risk corresponding to higher levels of loading, for example, tankers and commercial traffic. It follows that lower levels of loading sufficient to cause cracking, but at lower levels of risk, are acceptable. Additional checks of stability resulting from loss of drawdown should also be made. This will correspond to an Ultimate Limit State and will occur when cracking is sufficiently extensive to cause flooding. Stability in this instance should also therefore be linked to an acceptable level of risk.

Clause 23.2.17 Implosion - The maximum hydrostatic pressure to which a structure will be subjected may not arise in the permanent condition, but rather during the temporary construction and installation phases, particularly deck mating. Hence, all phases of the life of the structure must be considered.

The hydrostatic head used for assessment of implosion in the permanent condition should be based on greatest still water depth in conjunction with the highest astronomical tide and storm surge. Differential pressure changes due to wave and current action may be considered as secondary, but, particularly the former, should be assessed. Temporary hydrostatic loading should be based on an assessment of the worst combination of conditions that may occur.
The influence of wave height on the pressure distribution is due to the effect of a combination of drag, inertia and diffraction forces. Where diffraction dominates, the variation of pressure due to wave disturbances may be determined by the equation:

\[ P(x,s,t) = \gamma \frac{H}{2} \frac{\cosh (k s)}{\cosh (k d)} \cos (k x - \sigma t) \]

where: \( P(x,s,t) \) = wave induced pressure
\( \gamma \) = unit weight of fluid
\( H \) = wave height
\( s \) = \((z + d)\) = elevation above bottom
\( z \) = height of crest above still water level
\( d \) = depth from still water level to sea bed
\( k \) = \(2\pi/L\)
\( L \) = wave length
\( \sigma \) = \(2\pi/T\)
\( T \) = wave period
\( t \) = time.

Chrapowicki et al (1987), in reviewing suitable methods of analysis of the buckling strength of cellular structures subjected to hydrostatic pressure, concluded that the proposed method by Haynes (1979) was the most suitable of the desk calculation type methods. This is a semi-empirical approach which is based on three equations which describe the failure of concrete cylindrical structures under hydrostatic loading. These equations apply to thick wall cylinders, moderately long thin wall cylinders (Donnell), and long thin wall cylinders (Bresse). An empirical plasticity reduction factor is used which accounts for inelastic behaviour and out-of-roundness.

The method referred to above has been further developed to provide a graphical solution for complete and partial cylindrical shells and is applicable when a preliminary sizing is required or when the cylindrical shell has simple and well defined design conditions (Chrapowicki et al 1987). A particular feature of this procedure is that provision is made for partial cylinders. The parameters in this instance have been obtained using the BOSOR 4 computer program which was developed at the Lockheed Palo Alto Research Laboratory.

The recommended calculation procedure, based on Figures 18, 19 and 20, is as follows:

- For a full cylinder, a slenderness number is obtained directly from the chart in Figure 18 for the selected parametric ratios.

- For partial cylinders, also with simply supported ends, Figures 18 and 19 are used in conjunction to obtain the slenderness number of shell elements with free, partial or full edge restraints.

- On finding the slenderness number, and with the knowledge of factors limiting the design compressive strength, the buckling strength is then obtained from Figure 20. The upper curve in this figure (short term duration) was based on Haynes formula (Haynes 1979) and on mean experimental results curve in the group \( n = 3 \) but the left hand end has been dropped to encompass all the results in the group. The lower curve (long term duration) was obtained from the upper curve by allowing for creep.
Figure 18
Slenderness number $\beta$ for a complete Cylinder
(after Chrapowicki et al 1987)
• Among the factors reducing compressive strength the following are normally considered: saturation/age factor, materials factor, cylinder characteristic strength reduction factor which is used for members in flexure, cube/cylinder conversion factor (where applicable), and creep factor. For example, the following typical values may be applicable:

  • saturation/age factor = 0.98 (Haynes et al 1976)
  • cylinder characteristic strength reduction factor = 0.85 (ACI), 0.67 (BS 8110)
  • materials factor = 1.5
  • creep factor = 0.2

• The resulting design buckling stress is then compared to the calculated circumferential stress from the applied factored loads. The buckling stress for cylinders with other than simply supported ends may be obtained from published values on a pro-rata basis. (Leick et al 1978, Haynes 1976, Haynes 1979).

Computer programs may be considered for analysis of complex details such as branching, multi-lateral composition, asymmetrical loadings, temperature loadings, etc. Chrapowicki et al (1987) provides a listing of suitable programs but these should, nevertheless, be verified before use since programs have generally been developed for specific tasks with limited verification.

Imperfections may cause significant variations in buckling strength (Borseth et al 1976). However, the Haynes method, on which the graphical method described above is based, allows for imperfections. The cylinders used in the experimental program included imperfections in which the out-of-roundness ranged from $\frac{R}{208}$ to $\frac{R}{265}$. Imperfections greater than $\frac{R}{200}$ should therefore be checked either with more sophisticated computer methods or by using the graphs provided by Borseth et al (1976).

The influence of load duration corresponding to the variation in elastic modulus may be significant. The graphical method provides data for a range between 1 day and several years. Further graphical data may be obtained from Zaleski-Zamenhof (1976).

The effect of reinforcement is difficult to assess. The graphical method is based on original work which did not include reinforcement but it is, nevertheless, recommended that reinforcement may contribute in compression members for ultimate conditions provided that the reinforcement is tied against lateral movement. Further data on variation in buckling strength with reinforcement may be obtained from Zaleski-Zamenhof (1976) and Borseth (1976).

Assessment of the stability of the structure regarding Limit States has been based on the recommendations of Zaleski-Zamenhof (1976) with respect to both implosive buckling and stress arising from bending moments, compression and other secondary effects.
Clause 23.2.18 Oil control systems and storage - The requirements for oil control systems and storage should be read in conjunction with those for temperature effects. The clause essentially provides the Serviceability Limit State requirements when storage systems are subjected to temperature differentials and has been taken from BS6235 (now withdrawn). No distinction for crack width parameters between reinforced and pre-stressed concrete has been made, the purpose of these crack width requirements are to prevent leakage and hence it does not seem appropriate to have separate requirements.

![Diagram](image)

**Figure 19**
Partial cylinder slenderness number design envelopes
(after Chrapowicki et al 1987)

Clause 23.2.19 Temperature effects - Temperature effects may occur when oil in conductors transmits heat through the conductor casing to the base or tower walls, or when oil, gas, liquified gas or similar fluids are stored. The magnitude of the design temperature range will be dependent on the product temperature and cooling applied before storage.

The guidance on temperature effects is based on tests carried out by Clarke et al (1987) which concluded that provision of normal minimum reinforcement areas was adequate for the temperature ranges tested. The tests also indicated that cracking was not evident at temperature differentials below 20°C. The Lloyds formula for minimum reinforcement has therefore been adopted in the Guidance Notes and a note included that cracking is unlikely at temperature gradients below 20°C.

It should be noted that the series of tests carried out by Clarke et al (1987) was not intended to form the basis of a comprehensive design method for cyclic thermal loading of oil storage vessels, as it considers only local effects. These results should therefore be considered in conjunction with analysis that considers the global effects of thermal loading.
Figure 20
Implosion design curves for imperfections $< R/200$
(from Chrapowicki et al 1987)

Where it is considered that the global effects of thermal loading may be significant, additional analysis may be performed on the basis of the recommendations by Richmond et al (1980) which describes methods for calculating the stresses and strains produced by differential temperature in concrete caissons and other elements in the platform. The following may require consideration:

- **Thermal gradients in walls.** A difference in temperature between the inside and outside face of a wall under steady state conditions produces a uniform gradient through the wall. The cross-section of a cylinder corresponding to a ring therefore comprises a series of elements which are fully restrained against rotation but free to move in a translation sense. Flexural restraint is also provided in the vertical direction, that is along the longitudinal axis of the cylinder.

- **Overall thermal differentials.** Structures normally comprise several cells leading to complex interaction between cells due to variations in wall thickness as well as temperature levels between cells and the surrounding sea water.
• **Influence of temperature on creep of concrete.** The strain in components of concrete is influenced by temperature in various ways. The elastic modulus reduces with increasing temperature, giving rise to higher elastic strains at elevated temperatures. In contrast, the delayed elastic (recoverable) creep component of the total creep strain responds little to temperature, and may be considered temperature-constant in most situations. The non-recoverable creep, however, is highly sensitive and increases with rise in temperature. In thermal problems, the non-recoverable creep dominates the stress and strain behaviour of concrete structures. The extent to which stresses are redistributed as a result of differential thermal creep depends on the nature of the spatial temperature distribution and the internal constraints of the structural system. In oil storage structures, complete restraint to thermal curvature is commonplace by reason of their geometry. When such constraints exist, stress redistribution can be severe, to the extent that moments can be caused to change sign with passage of time as the result of temperature-dependent creep. The interaction of major elements (e.g., walls and domes), which are at differing temperatures, also results in a high degree of membrane stress redistribution.

Since creep strains are proportional to temperature change as well as stress, any long term non-uniform temperature distribution results in creep strains which cause a redistribution of the stress system. Also, where there are temperature differentials, not only thermal stress systems are affected by creep but also those produced by long term applied loads. Changes in temperature are produced by either the storage of oil or the discharging of oil to tankers. Each case is subject to different characteristics with respect to the temperature cycle.

• **Change in stiffness due to cracking.** Internal forces produced by imposed strains and deformations are generally reduced by the decreasing stiffness of concrete as it cracks. The curvature after cracking may therefore be approximated by the formula (Beaby et al 1969):

\[
\frac{1}{R} = \frac{M - M_{cr}}{0.85EI_{cr}} + \frac{M_{cr}}{EI_0}
\]

where: 
- \( M \) = bending moment 
- \( E \) = modulus of elasticity (concrete) 
- \( I_o \) = 2nd moment of area of uncracked section 
- \( I_{cr} \) = 2nd moment of area of cracked section 
- \( M_{cr} \) = cracking moment = \( f_t I_o / y_o \) 
- \( f_t \) = tensile strength of concrete 
- \( y_o \) = distance from neutral axis to tension face before cracking

Checks of both Ultimate and Serviceability Limit States are required on the recommendations of Richmond et al (1980). The influence of the above stress fields in the concrete structure at the Serviceability and Ultimate Limit States must be considered. One approach is to ensure that the ultimate load is not affected by temperature effects, since they are imposed deformations rather than applied loads. It is only possible to justify such an approach for a particular structure and load system by a detailed investigation, including appropriate experimental data or tests. The complexity of offshore structures suggests that no conclusion of such generality can be justified.
**Clause 23.2.20 Fire resistance** - There is limited specific information on fire resistance outside the various Codes and Guidance Notes. All Codes, which cover the subject, indicate that the minimum period of fire resistance should be stated in the design brief so that adequate protective measures may be taken by the selection of appropriate aggregates, reinforcement and concrete cover.

**Clause 23.2.21 Lightning protection** - Brief notes on lightning protection are made in various Codes and the Clause is written based on BS6235 (now withdrawn). This and BS 8110 require that any lightning protection system should be designed in accordance with CP326. Where the reinforcement in a concrete structure can be shown to have sufficiently low inherent resistance to earth, as required by CP326, it may be used as part of the lightning protection system. Otherwise an electrical ground conductor should be provided to protect prestressing tendons and reinforcing steel from accidently acting as a ground.

The lightning path will be determined by impedance rather than resistance and large amounts of prestressing bars in a (relatively) thin circular wall or shaft can form a low impedance path. Although the end anchorages may be concreted in, the tendons may still act as the lightning conductor with possible consequential damage.

**Clause 23.2.22 Monitoring** - The provision of instruments to monitor the behaviour of the structure and foundations is useful in confirming the predicted performance. This is particularly useful in the tow-out and installation phase. The availability of monitoring data may simplify re-certification (see section 2.3.6 of the Guidance), satisfying the Certifying Authority of the continuing performance of the installation.

**Clause 23.3 Specification and workmanship**

**Clause 23.3.1 Materials**

**Clause 23.3.1(a) Standard materials** - This clause repeats the requirement of Clause 23.2.1 in that BS 8110 Parts 1 and 2 apply equally to the material aspects.

**Clause 23.3.1(b) Non-standard materials** - This clause has been introduced to allow recognition of materials in regular use, but for which a British Standard is not available. For these materials appropriate investigations are recommended to assess their fitness for purpose, but due recognition should be given to any Agreement Certificate which may be available.

**Clause 23.3.1(c) Materials investigations** - All materials must be investigated to ensure not only adequate and timely supply but also that they satisfy the design requirements. These requirements must include the safety, structural performance and durability of the finished structure taking full account of both the construction, installation and operating environments to which it will be subjected. The timing and nature of these investigations should be such as to enable the characteristics of the materials and their likely effects to be fully evaluated in the design and reflected in the specification.
Where non-standard or unfamiliar materials or combinations of materials, are used in concrete, account should be taken of any possible interactions between the materials, the effect of environmental conditions, and any other construction and/or in-service requirements for the finished concrete. The properties of such concretes may differ, in various respects, from those made with standard materials and their suitability for the purpose required should be established by previous data, experience or tests.

**Clause 23.3.2 Cement**

**Clause 23.3.2(a) Cement type** - Research indicates that the selection of cement type perhaps merits more detailed technical consideration by specifiers than may have been general practice in the past. These considerations should encompass the concrete performance requirements at the time of construction and the in-service conditions. They should also take into account any additional measures or precautions which may be necessary to achieve these requirements such as rate of strength development, and longer or more effective curing.

In this clause a distinction has been introduced between active hydraulic binders and blended hydraulic binders (as defined in BS6100: Section 6.1:1984, British Standard Glossary of Building and Civil Engineering Terms). Reference to Sulphate Resisting Portland cement has been omitted, as its use does not appear to be essential for an adequate defence against the sulphates present in sea water, due to the use of high quality concretes in these structures, and because its use may not be advantageous in relation to its tolerance of and resistance to chlorides. Low heat Portland cement has also been omitted for simplification; it has always been a special order cement whose actual use since the 1970’s has been limited by the availability of lower cost alternatives.

However, Portland blastfurnace cements have been retained. Portland pulvlsed-fuel ash and pozzolanic cements have been introduced following acknowledgement of their sulphate resisting properties in BRE Digest 250, and research indications of their relative merits regarding tolerance to original chlorides and to chloride ingress.

**Clause 23.3.2(b) Latent hydraulic binders** - Research by Taylor Woodrow (1987) and Stillwell (1987) as part of the Concrete-in-the-Oceans programme has indicated possible advantages of including pfa into the concrete mix. The use of latent hydraulic binders, including pfa and ggbs, may be advantageous to reduce heat of hydration, permeability and cost, and to provide increased resistance to sulphate attack.

The relative proportions of ordinary Portland cement and latent hydraulic binder should be suitable for the appropriate construction, installation and operating conditions involved. Appropriate testing for the combined mix should be carried out as part of the design process.

**Clause 23.3.2(c) Tricalcium aluminate** - A tricalcium aluminate (C₃A) content limit of 12% maximum has been retained, with a lower limit of 5% introduced. This reflects research indications that low C₃A content cements have lesser resistance to original chlorides and subsequent chloride penetration. These proposed limits do not apply to blended hydraulic binders.

Where it is wished to demonstrate that different limits should be used, particular reference should be made to any chlorides originally present at the time of mixing and to any subsequent contact, and to sulphate resisting properties.
Clause 23.3.3 Aggregates

Clause 23.3.3(a) Standard Specification - Appropriate investigations should be carried out where it is anticipated that any aggregate is likely to have any unusual or adverse effect upon the physical and chemical properties of the concrete, or upon the protection of reinforcement and prestressing tendons, or upon the overall performance and durability in-service.

The choice of aggregates may, at some construction sites, be limited for reasons of availability rather than any technical requirements. In such cases a particular property may not meet a specified requirement and the question of its acceptability arises notwithstanding non-compliance.

Clause 23.3.3(b) Dimensional changes - Any aggregates which may have higher than normal drying shrinkage characteristics should be tested and assessed in accordance with the guidance given in BS 8110: Part 1, Clause 6.1.3.7 and in Part 2, Section 7.

Clause 23.3.3(c) Alkali-silica reaction - In the UK the incidence of problems due to, Alkali Silica Reaction (ASR) is small. However, it should be appreciated that there is presently no proven effective remedy once ASR has begun in a concrete and its continuance could, therefore, ultimately result in structural distress and the need for demolition.

Guidance on the choice of materials and other limitations to avoid or minimize damage due to ASR are given in the original report by Hawkins et al. (1983) as extended in the later edition (at time of writing, still to be published). This guidance should be considered by the owner and note taken of the uncertain significance of the alkalis present in the sea water.

Clause 23.3.3(d) Lightweight aggregates - Guidance on lightweight aggregates has been introduced by reference to BS 8110: Part 2 Section 5, 'Additional considerations in the use of lightweight aggregate concrete'.

Clause 23.3.4 Water

BS3148 (1980) includes, as Appendix A, guidance on the suitability of water for making concrete. This guidance is encompassed through reference to BS 8110: Part 1.

Clause 23.3.5 Admixtures

Clause 23.3.5(a) Standard Specification - Admixtures have been used extensively for many years in high quality concretes. Hence, it is considered that the benefits they can provide and the need for strict dosage control are now well understood. For this reason, only limited guidance on their selection and use has been incorporated into the Guidance Notes.

The requirements for certain common admixtures are covered in BS5075: Parts 1, 2 and 3.
Clause 23.3.5(b) Approval and performance - This clause is intended to ensure that the suitability of any admixture, or any combination of admixtures, is fully established before use in permanent works. The behaviour of admixtures with blended hydraulic binders may differ from their behaviour with ordinary Portland cements and, when appropriate, the trial mixes should be so designed to assess this possibility.

Clause 23.3.5(c) Air-entraining admixtures - An air-entraining admixture may be used to mitigate the effects of harsh or coarsely graded fine aggregates, or to minimize any tendency of a concrete to bleed or suffer cracking due to plastic settlement. This application would be in addition to the primary reason for use of providing improved resistance to freezing and thawing.

The performance of air-entraining admixtures should, in addition to the normally specified requirements for total air content, measured at the placing point, be assessed by direct measurements of the total and entrained air content of the hardened concrete. These measurements should be made in accordance with ASTM C457 and should include determination of the 'spacing factor'. Thus, the entrained air spacing factor with a limit of 0.2 maximum has been introduced following experience of failure investigations of air-entrained concretes.

Clause 23.3.6 Concrete mixes

Clause 23.3.6(a) Concrete grade - Subject to any limitations in the Guidance Notes themselves all concrete mixes should be 'designed mixes', which should be specified, produced and tested for compliance in accordance with BS 8110 : Part 1.

The primary requirement for any concrete, which is to provide effective long-term protection to reinforcing systems and other embedded metal and, equally to resist aggressive environments, is to have very low diffusion characteristics to water, chlorides, sulphates, alkali metals (sodium & potassium), carbon dioxide, sulphur dioxide and oxygen. In reality these seemingly formidable requirements apply mainly to that concrete which forms the cover to the reinforcement and which, inevitably, is most vulnerable to inadequate curing and protection during construction; potentially more so for slipformed constructions and where, for other reasons, cements having slow rates of strength development (i.e. lower heat cements) are used.

The achievement of low diffusion rates must, at least in specification terms, rely upon the traditional requirements for minimum strength and minimum cement contents coupled with maximum water/cement ratios. Less well recognised, but of importance, is the need to ensure that the free-water content of the concrete is as low as can be achieved with given materials and workability requirements. However, these requirements cannot do more than create the required potential qualities, the extent to which they are realised being largely governed by mix design, workmanship and supervision factors.

There has been very limited research into the minimum grade of concrete required for offshore concrete structures. This clause has been written by reference to BS6235 (now withdrawn) and FIP (4th Edition) except for the concrete grade for severe exposure condition where the two codes differ. In this case the recommended grade is taken from BS6235 (now withdrawn), which recommends grade 50 and not grade 45 as recommended by FIP.
On an existing platform investigations were carried out on an area of leg which had been subjected to hydrogen sulphide attack. This had penetrated some 1 to 2mm into the concrete before ceasing and no significant damage occurred. In another structure stagnant water was found but, as above, no degradation of the concrete was discovered. At the time these two discoveries raised some concern and it is important the owner is aware of these potential hazards.

As discussed in Section 3.2 of this report, research by Haynes et al (1979) and Stillwell (1987) has highlighted a reduction in strength of concrete due to immersion. Haynes et al have suggested that the owner should decrease the 28 day fog-cured strength by 10% for saturation effects, then increase the strength for age effects by 10% at 6 months and 15% after 1 year. One existing platform took into account a 25% reduction in 28 day cube strength for immersion effects in its design. Initial findings from work at BRE indicate the very substantial tensile strength loss that may occur.

Clause 23.3.6(b) Chloride content - The chloride content limits in BS 8110: Part 1 apply to the 'total' chloride content of the concrete as per cent chloride ion by weight of cement (inclusive of pfa or slag when used) and these limits should not be exceeded.

It is, however, normal practice for compliance to be assessed indirectly, on the basis of computed total chloride contents using those determined for the individual constituents, and which commonly do not allow for any chloride present in the water and the cement. This practice is deprecated for the purposes of determining compliance. However, it may be accepted for control purposes, provided that the chloride contents of all the constituents are known and that there is no significant difference between the 'total' acid soluble chloride content of the aggregates and those determined by the water-soluble method prescribed in BS812.

Whenever there is chloride in concrete there is an increased risk of corrosion of reinforcing systems and other embedded metals. The higher the chloride content, the curing and operating temperatures, or if subsequently exposed to warm moist conditions, the greater the risk of corrosion. Chloride may also adversely affect the sulphate resistance of concrete. Aggregates and cement may contain chlorides and concrete may suffer penetration by chlorides from de-icing salts, by air-borne salt spray, and direct contact with the sea. Calcium chloride and chloride based admixtures should never be used in reinforced concrete, prestressed concrete and concrete containing embedded metal.

Previous British Standards and Codes of Practice, and therefore Guidance Notes, have tended to apply different parameters for the control of durability to prestressed and reinforced concrete. These parameters are cover, crack width, concrete grade and chloride content; Clauses 4.2.3.13 and 4.2.3.12 which put forward previous guidance for cover and cracking did differentiate between the requirements for prestressed and reinforced concrete. The BS 8110 requirements for chloride content, called up in this clause, retain the separate limits but, for post-tensioned offshore structures the owner may consider it appropriate to work to a single standard equivalent to that for reinforced concrete. Corrosion protection of post-tensioned tendons is more dependent upon complete grouting of the ducts, than external parameters applied to the parent concrete.
Clause 23.3.6(e) Sulphate content - Guidance on the maximum permissible sulphate content of concrete is now included by reference to BS 8110: Part 1. However, it should be noted that, at time of writing, a BSI amendment is expected to revise the limit to apply to total sulphate content, as opposed to total water-soluble sulphate content, as at present.

Clause 23.3.6(d) Design assumptions - The concrete properties which may require verification will include coefficient of thermal expansion/contraction, drying shrinkage, wetting expansion, modulus of elasticity, compressive and tensile strength, strength development with age (i.e. age factors), and creep. They should be verified by British Standard or other recognised test procedures, preferably using samples of concrete produced under full scale conditions. Guidance on a number of these factors is given in BS 8110: Part 2, Section 7.

Clause 23.3.7 Reinforcement - There is no reason to modify or amend the BS 8110 requirements.

Clause 23.4 Construction

Clause 23.4.1 Standard Specification

Clause 23.4.2 Concrete mix and grade - Guidance on concrete mixes is given in the materials section.

Clause 23.4.3 Construction joints - The section in BS 8110 on construction joints, clause 6.12, adequately covers all requirements on this subject in considerable detail. Because of the nature of offshore concrete structures particular attention should be paid to those sections which are to remain watertight or oiltight. It should also be noted that BS 8110 defines movement joints in Clause 6.13, in addition to construction joints in Clause 6.12.

The number of joints should be kept to the minimum possible, consistent with reasonable precautions against shrinkage. Experience with existing structures in the North Sea has indicated the vulnerability to leakage of joints, leading to difficulties in maintaining drawdown or preventing corrosion. The use of 'letter box' shutters, even in conjunction with epoxy grouting, has proved in certain cases to be unsuccessful and whenever possible boxouts for subsequent concreting should be avoided.

Clause 23.4.4 Curing - Guidance on curing is given in BS 8110 Clause 6.6 which states that curing is the process of preventing the loss of moisture from the concrete whilst maintaining a satisfactory temperature regime. Curing and protection should start immediately after the compaction of the concrete to protect it from:

- premature drying out, particularly by solar radiation and wind
- leaching out by rain and flowing water
- rapid cooling during the first few days after placing
- high internal thermal gradients
- low temperature or frost
\begin{itemize}
\item vibration and impact which may disrupt the concrete and interfere with its bond to the reinforcement.
\end{itemize}

The minimum requirements for curing are given in Table 6.5 of the BS 8110 Clause 6.6 but special attention should be paid to offshore concrete structures in order to ensure maximum durability and to minimize cracking.

Both BS6235 1982 (now withdrawn) and FIP fourth edition require that the concrete should be cured with fresh water whenever it is possible. Seawater should not be used although if demanded by the construction programme, concrete may be submerged in sea water provided that it has gained sufficient strength to avoid physical damage from waves etc. When there is doubt about the ability to keep concrete surfaces permanently wet for the whole of the curing period or where there is a danger of thermal shock, a heavy duty curing membrane should be used.

Leeming (1987) reports that, for testing carried out in the Concrete-in-the-Oceans programme, "the proper curing of concrete has been found to have a much greater influence in producing the qualities in concrete required for enhanced durability than any improvements in strength or changes in type of cement". Early exposure to seawater, however, as part of the curing regime may cause increased chloride ingress outweighing any advantages of the better curing.

**Clause 23.4.5 Concreting in cold weather** - Guidance on concreting in cold weather is given in detail in BS 8110 Clause 6.7. FIP gives brief notes on this subject, but in insufficient detail to warrant particular reference.

**Clause 23.4.6 Concreting in hot weather** - Guidance on concreting in hot weather is given in detail in BS 8110 Clause 6.8. FIP does not deal with this subject, while ACI refers to the Code 305R 'Hot Weather Concreting'.

**Clause 23.4.7 Cover** - The nominal cover to all reinforcement is given in Clause 23.2.12 of these Guidance Notes and particular care must be made in construction to achieve these values, taking into account acceptable tolerance for which suggested values are given in Clause 23.4.8 of these Guidance Notes.

**Clause 23.4.8 Tolerances** - Absolute accuracy exists only in theory and dimensional variability is inevitable in practice. The permissible deviation specified should be as large as possible, without rendering the structure unacceptable for the purpose for which it was intended.

Data from existing platforms has been provided by Chevron and C.G. Doris. A table of suggested values has been compiled from this data and is shown as Table 5.

**Clause 23.4.9 Reinforcement** - There are no specific requirements for reinforcement in the construction of offshore concrete structures. Guidance on any aspects of reinforcement should be taken from BS 8110 Section 7.

**Clause 23.4.12 Planning** - Only BS6235 of the offshore-related Codes of Practice gives any guidance on planning requirements in the design and construction of offshore concrete structures. This clause has therefore been taken from sections 7.4.2, and 7.5 of BS6235.
Clause 23.5 Tow-out - Section R6.5 of FIP (fourth edition) gives considerable guidance on tow-out of concrete offshore structures and it is recommended that an owner should at least consider the requirements detailed in this Code of Practice.

In the design and planning of tow-out it is important to ensure that the structure, or any part of the structure, is not exposed to loadings greater than those for which it was designed.

Particular attention needs to be given to the response to motion of the structure and its stability during tow-out.

The choice of towing route should take into account the depth of water, the strength and direction of tidal streams and currents and any navigational hazard that could affect the safe conduct of the tow.

Consideration should also be given to identifying possible holding areas on the towing route, to allow for weather and tidal factors.

In restricted waters towing should receive special attention, in particular it may be necessary to consider using pusher tugs as well or instead of pulling tugs.

At all times navigation lights and aircraft warning lights should be displayed and the structure should conform to the relevant marine or air regulations.

Clause 23.6 Placing the Structure - Section R6.6 of FIP (fourth edition) gives considerable guidance on Installation aspects and it is recommended that an owner should refer to this Code of Practice for further details.

All aspects of the installation of the structure, including its immersion and placing on the seabed, should be planned and carried out with the greatest care. The structure should be placed in position to tolerances given in Clause 23.4.8 of these Guidance Notes.

Environmental factors such as wind, wave and the variation in the unit weight of seawater should be taken into account when planning the immersion operation.

The condition of the seabed and its topography should be determined beforehand so that any pre-installation seabed preparation or strengthening can be finished before the arrival of the structure.

If scour protection is to be provided care must be taken to ensure that damage to the structure is avoided.

Before ballasting or pulling down is begun, the environmental conditions should be checked to make sure they are not worse than those assumed during design and planning of the operation. In addition, all systems should be checked to ensure that all aspects of the plan can be carried out as scheduled and with the required margin of safety.

Once immersion has begun, it should proceed decisively. The stability of the structure at the various stages of immersion should be analysed and immersion should be planned so as to shorten the duration of critical phases. Where the structure has a sudden reduction of the water plane, immersion should be planned to continue to a depth producing an acceptably increased stability before commencing any operation which
might influence trim or heel. Variations in buoyancy with depth and with time should be taken into account during immersion of the structure. During immersion the structure may be kept on location by means of tugs and/or by using moorings.

The method used to maintain location should possess adequate capacity to resist horizontal forces which may arise. Seabed anchors to be used for pulling down should be designed for a working load at least equal to the pull required and in excess of the breaking load of the pulling lines. The flooding or pumping system for controlling water ballast should possess adequate redundancy and excess capacity to deal with the possibility of leaks or inoperative valves.

All sea intake lines should be protected by spring-operated valves which close automatically in the event of loss of control air or fluid pressure.

Where compressed air or gases are used to resist hydrostatic loads, potential operational risks associated with such means shall be considered, taking into account realistic variations in the pressure of the compressed air.

The level and location of the structure should be monitored with extra care and accuracy as it nears the seabed. The effects of impacting collision with seabed, suction and break-out should be taken into account and the structure should be equipped with means to control these effects.

Clause 23.7 Ballasting/de-ballasting - Concrete gravity platforms possess complex pumping and piping arrangements to enable ballasting/de-ballasting to be carried out. In the design of these arrangements the system should contain adequate redundancy and excess capacity to deal with the possibility of leaks or inoperative valves.

At the time of construction special attention should be paid to maintenance of access to the various pumps for maintenance purposes. All sea intake lines should be protected by spring operated valves which close automatically in the event of loss of control of air or fluid pressure. These closures should have a manual over-ride or other means of redundancy.

In cases where drawdown is used in the structure, in order to prevent accidental future leakage occurring, particular attention must be paid to removing and/or sealing pipework or fittings.

Clause 23.8 Inspection and testing during construction - Concrete is a very durable structural material when concrete structures have been well designed and then constructed to a high standard of workmanship. With good materials, little repair work is required. Nevertheless, repairs sometimes have to be carried out for a variety of reasons. Faulty construction or poor materials may lead to deterioration of the concrete or corrosion of the reinforcement and the structure may be damaged by overload, impact, abrasion or fire.

Experience in the North Sea to date has shown an almost complete absence of the need for maintenance and repair of in-service concrete platforms. This is due to conservative design criteria and good Quality Control procedures during construction. The cost of in-service inspection, maintenance and repairs, particularly in the underwater section of the structures is high. It is therefore essential that a high priority is given to achieving a
high quality product at the time of construction. To achieve this objective a comprehensive Quality Assurance system is desirable, of which sophisticated Quality Control procedures are an essential element. It should be noted perhaps that Quality Assurance is a system whose aim is to achieve fitness for purpose at the lowest cost. BS 8110 gives detailed requirements for the inspection and testing of concrete, reinforcement and other structural components and it is suggested that these should be the minimum requirements of the Quality Control procedure.

The design and equipping of the structure should include provision for periodic in-service inspections. Such provisions should include, as appropriate, diving bells and diving bell recovery systems, diver support equipment, underwater television systems, clamping systems for submersibles, etc. Permanent reference points or marks for the accurate location of areas to be inspected are important and should be provided.

4.2.3 Section 30: Floating Installations

Clause 30.2 Design and Construction requirements

Clause 30.2.3 Concrete - The period during construction and transit when fixed concrete Offshore Installations may be afloat should be taken into consideration during the design. In most cases it will be necessary to demonstrate that the conditions during this period will not be more severe than those envisaged during design.

Concrete floating installations may be designed according to the latest rules for mobile installations where relevant.

The requirements of Section 23 of the Guidance should be followed as far as is practical.
APPENDIX 1
DATABASE FOR PROJECT

This Appendix contains the listing of the relevant literature collated for the revision of the draft Guidance Notes for Concrete Structures. The references are listed under the main topic headings of Design (D), Materials (M), Construction (C) and Standards (S) in alphabetical author order.

Details of the data collection and handling methods used for the project are given in Section 1.3 of this Report and the field list of keywords used is given in the Table below. Because many papers or publications refer to more than one topic, they may be listed under more than one keyword.

A designation A in the keywords signifies that only an abstract of the paper is held in the database and a designation R that the entry is a reference only and is not physically held in the database.

**Keyword list for database management**

<table>
<thead>
<tr>
<th>Keyword</th>
<th>Subject</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design</td>
<td></td>
</tr>
<tr>
<td>D1</td>
<td>Durability (cover, cracking and corrosion of reinforcement)</td>
</tr>
<tr>
<td>D2</td>
<td>Shells (caissons, implosion, buckling)</td>
</tr>
<tr>
<td>D3</td>
<td>Shear (impact, dropped objects and ship impact)</td>
</tr>
<tr>
<td>D4</td>
<td>Temperature Effects</td>
</tr>
<tr>
<td>D5</td>
<td>Fatigue</td>
</tr>
<tr>
<td>D6</td>
<td>Deflection</td>
</tr>
<tr>
<td>D7</td>
<td>General</td>
</tr>
<tr>
<td>Materials</td>
<td></td>
</tr>
<tr>
<td>M1</td>
<td>Cement</td>
</tr>
<tr>
<td>M2</td>
<td>Aggregates</td>
</tr>
<tr>
<td>M3</td>
<td>Water</td>
</tr>
<tr>
<td>M4</td>
<td>Admixtures</td>
</tr>
<tr>
<td>M5</td>
<td>Grout</td>
</tr>
<tr>
<td>M6</td>
<td>Reinforcement</td>
</tr>
<tr>
<td>M7</td>
<td>General</td>
</tr>
<tr>
<td>Construction</td>
<td></td>
</tr>
<tr>
<td>C1</td>
<td>General</td>
</tr>
<tr>
<td>C2</td>
<td>Inspection and Maintenance</td>
</tr>
<tr>
<td>Standards</td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>Standards, Guides and Codes of Practice.</td>
</tr>
</tbody>
</table>
OFFSHORE INSTALLATIONS
REVISION TO GUIDANCE ON DESIGN AND CONSTRUCTION
DATABASE

ACI
Guide for the design and construction of fixed offshore concrete structures ACI 357R-78
American Concrete Institute 1978
S1.1

ACI
The use of fly ash - silica fume - slag and other material by-products in concrete Volumes 1 and 2
Proc of the CANMET/ACI 1st Int Conf Quebec Canada
American Concrete Institute Publication Sp 79 July/Aug 1979
R M721

ACI
Guide for the design and construction of fixed offshore concrete structures ACI 357R-84 Draft copy
American Concrete Institute Feb. 1984
S1.34

ACI
State-of-the-art report on concrete structures for the arctic
American Concrete Institute 1984
S1.46

ACI
Building Code requirements for reinforced concrete
American Concrete Institute 318-83
M1.27

ACI Committee 349
Reinforced concrete design for thermal effects on nuclear power structures.
Journal of the American Concrete Institute November/December 1980
R D4.22

ASTM
Corrosion of reinforcing steel in concrete
American Society for Testing and Materials
December 1978
R D1.55
ASTM
Significance of tests and properites of concrete and concrete making materials
American Society for Testing and Materials 1978
M7.40R

Albertson HD
Influence of compressive strength and wall thickness on behaviour of concrete cylindrical hulls under hydrostatic loading.
Technical Report R790 CEL June 1973 Port Huenenua
California
D7.67

Allen AH
Reinforced concrete design to CP110 - Simply explained
Cement and Concrete Association 1974
D7.73

Allen RTL
Concrete in marine works
D7.62 C1.27

Almazar VO Kopaigorodski EM
Bearing capacity of a reinforced concrete shell in arctic environment
Moscow Institute of Civil Engineering January 1982
D7.92

Anderson GH
Cathodic protection of a reinforced concrete bridge deck
Concrete International
A D1.92

Anderson KH et al
Review of foundation design principles for offshore gravity platforms
Norwegian Geotechnical Institute
C2.25

Anon
Examples of damage assessment
D3.35 C2.23
Anon
Eurocode No.2: Concrete structures
Commission of the European Communities
1984
S1.43

Anon
Concrete for deep ocean construction
Concrete Construction May 1983
D7.78

Anon
Structural codes - the rationalisation of safety and
serviceability factors
CIRIA - Proceedings of the Seminar in London 1976
September 1977
D7.75

Anon
Eurocode No.1: Common unified rules for different types of
construction material
Commission of the European Communities
S1.55

Anon
Changes in cement properties and the effects on concrete
Concrete Society Working Party Report 1984
M1.26

Anon
Maintenance inspection and repair - State-of-the-art for
Arctic structures
1984
C2.35

Armitage JS
The anatomy and exercise of engineering judgment
The Structural Engineer/Volume 59A/No. 5/May 1981
M7.23

Arthur PD Earl JC Hodgkiss T
Fatigue of reinforced concrete in seawater
Concrete May 1979
D5.17
Arup H
Galvanised steel in concrete
National Association of Corrosion Engineers April 1979
D1.24

Arup H
Recent progress concerning electrochemistry and corrosion of steel in concrete
ARBEM Symposium October 1982
D1.98

Arup H
Corrosion related maintenance problems especially offshore
Sixth European Maintenance Congress June 1982
D1.99 C2.32

Askheim NE  Roland B
The relevance of present criteria for corrosion resistance of marine reinforced concrete structures
Det Norske Veritas 1983
D1.19

BRE
The durability of steel in concrete : Part 1 Mechanism of protection and corrosion
Building Research Establishment July 1982
R D1.58

BRE
Research programme 1981/82
For Department of Environment
Building Research Establishment July 1982
A M1.15

BS 12 1978
Ordinary and rapid hardening Portland cement
British Standards Institute
S1.10

BS CP 110 Part 1 1972
The structural use of concrete
British Standards Institute
S1.23
BS 146 Part 2  1973
Portland blastfurnace cement
British Standards Institute
$1.11

BS 915  Part 2  1972
High alumina cement
British Standards Institute
$1.12

BS 3148  1980
Water for making concrete
British Standards Institute
$1.13

BS 4027  1980
Sulphate-resisting Portland cement
British Standards Institute
$1.14

BS 4449  1978
Hot rolled steel bars for the reinforcement of concrete
British Standards Institute
$1.15

BS 4461  1978
Cold worked steel bars for the reinforcement of concrete
British Standards Institute
$1.16

BS 4482  1969
Hard drawn mild steel wire for the reinforcement of concrete
British Standards Institute
$1.17

BS 4483  1969
Steel fabric for the reinforcement of concrete
British Standards Institute
$1.18
BS 4486 1980
Hot rolled and hot rolled and processed high tensile alloy
steel bars for the prestressing of concrete
British Standards Institute
$1.19

BS 4757 1971
Nineteen wire steel strand for prestressed concrete
British Standards Institute
$1.20

BS 4891 1972
A guide to quality assurance
British Standards Institute
$1.48

BS 5075 : PART 3
Draft British Standard for concrete admixtures BS 5075 Part 3
Specification for superplasticizing admixtures
British Standards Institute November 1983
$1.35

BS5400
Steel, concrete and composite bridges
British Standards Institute
$1.52

BS 6235 1982
Code of practice for fixed offshore structures
British Standards Institute
$1.5

BS 6349 : Part 1 : 1984
Code of practice for maritime structures Part 1 : General
criteria
British Standards Institute
$1.56

BS 6349 : Part 2
Code of practice for maritime structures Part 2 : Design of
quay walls jetties and dolphins
British Standards Institute
$1.57
BS Handbook 22 1983
Quality assurance
British Standards Institute
$1.49

Backx E  Rammant JP
Behaviour of fibre-reinforced concrete structures in sea
environments
Offshore Structures Conference October 1979
D7.5

Baker ALL
Limit state design
Cement and Concrete Association
D7.68

Baker MJ
Report and summary of recommendations on reliability of
offshore structures
Marine Technology Directorate April 1984
D7.72

Bakker R
On the cause of increased resistance of concrete made from blast
furnace cement to the alkali-silica reaction and to sulphate
Corrosion
June 1980
R M7.18

Bate SCC  Lewsley CS
Environmental changes - temperature creep and shrinkage in
concrete structures
Building Research Station
R D4.15 D7.4

Bazant ZP
Physical model for steel corrosion in concrete sea structures -
Application
ASCE Journal Structures Division V105 N6 June 1979
D1.22

Bazant ZP Kim JK
Size effect in shear failure of reinforced concrete beams
Northwestern University Illinois USA Report 83-5/4285
D3.36R
Beeby AW
Cracking and corrosion
Concrete in the Oceans Technical Report 1 1978
ISBN 072 1011276
D1.1

Beeby AW
Corrosion of reinforcement and crack widths
Offshore Structures Conference Penteck Press Oct 1979
D1.5

Beeby AW
Cracking cover and corrosion of reinforcement
Concrete International February 1983
D1.35

Beeby AW Miles JR
Proposals for the control of deflection in the new unified code
Concrete March 1969 3 101 to 110
D4.18R

Bennett EW
Report on fatigue tests of tensile bar splices in reinforced concrete beams
University of Leeds Dept of Civil Engineering Oct 1979
D5.23

Bergan PG Fiskvatan A Sorensen SI
Non-linear analysis and design of offshore structures
Norwegian Institute of Technology August 1979
A D7.26

Beslac J Bjegovic D Hranilovic M
Durability of reinforced concrete elements and structures placed by elements in the sea and with slipforms by the sea
Performance of Concrete in Marine Environments
St Andrews by-the-sea Canada August 1980 ACI Pub SP-65
A D1.29

Blok JJ Brozius LH Dekker JN
The impact loads of ships colliding with fixed structures
Offshore Technology Conference May 1983 Houston Texas
D3.7
Bobrowski J
Outstanding applications of lightweight concrete and an appreciation of likely future developments
Lightweight Concrete London Eng. April 1980
M7.13

Borseth I Groner F
Influence of imperfection and reinforcement on collapse pressure of concrete cylinders
BOSS 1976 WIT Trondheim Norway
D2.8

Braestrup MW Nielsen MP Bach S
Plastic analysis of shear in concrete
Technical University of Denmark Report 120 May 1979
D3.37R

Brakel J Oostlander LJ
Concentrated loading on a thick walled concrete cylinder
2nd Inter. Conf. on the Behaviour of Offshore Structures Aug 1979
D3.9

Brakel J Reinhardt HW Oostlander LJ
Concentrated loading on a thick walled concrete cylinder.
Offshore Structures Conference October 1979
D3.3

Brakel J Schults AA
Temperature stresses in the walls of an undersea oil tank
Cement (Netherlands) Vol XXXIII No 3 1981
R D4.26

Brewer RA Wendenburg GA Vache IC
Retrievable offshore complex for marginal fields
Offshore Petroleum Conference October 1980
A D7.33

British Petroleum
Quality assurance in BP
BP Group Engineering & Technical Centre April 1984
C2.27
British Steel Corporation Research Services
Fatigue and corrosion fatigue performance of 16mm diameter reinforcing steel
Concrete in the Oceans Technical Report No 18 1987
OTH 87 241
ISBN 011 4128960
D5.48 M6.20

British Steel Corporation Research Services
Toughness of reinforcing bars and its influence on the resistance of reinforced concrete beams to impact
BSC for Department of Transport Contract Report Nr. BE/22/2/0148
D3.51 M6.21

Brown IC Perry SM
The assessment of impact damage caused by dropped objects on concrete offshore structures
Concrete in the Oceans Technical Report No 17B 1987
OTH 87 240
ISBN 011 412398
D3.47

Browne RD
Design prediction of the life for reinforced concrete in marine and other chloride environments
Taylor Woodrow Research Lab Southall July 1982
A D1.13

Browne RD
Mechanisms of corrosion of steel in concrete in relation to design inspection and repair of offshore and coastal structures
Performance of Concrete in Marine Environments St Andrews by-the-sea Canada August 1980 ACI PUB SP-65
A D1.33

Browne RD
Low maintenance concrete - specification versus practice
Taylor Woodrow Construction Ltd - ICE 1986
D7.98 C2.37

Browne RD Domone PLJ
The long-term performance of concrete in the marine environment Offshore Structures Conference October 1974 ICE
D1.46
Browne RD Doyle VJ Papworth F
Inspection and repair of offshore concrete structures
European Offshore Petroleum Conference
London Oct 1980
A C2.4

Browne RD Doyle VJ Papworth F
Inspection of concrete offshore structures
Journal Petroleum Technology V33 N11 November 1981
A C2.3

Burdekin FM Rothwell GP
Survey of corrosion and stress corrosion in prestressing
components used in concrete structures ref to offshore structures
Cement and Concrete Association 1981
A M6.11

Bureau Veritas
Rules and regulations for offshore structures - Concrete section
S1.7

Bureau Veritas
Amendments and additions to the rules and regulations for the
construction and classification of offshore platforms
No. 1 - August 1982
S1.45

Burks Green and Partners
Fire damage to offshore concrete installations
Report to Department of Energy January 1982
D7.89

Bury MRC Fielder NR O'Flynn MC West FS Pannett RJ
Fixed offshore structures : the new code of practice BS6235
Symposium of the ICE Volume 74 August 83
S1.36

Busby RF
Underwater inspection/testing/monitoring of offshore structures
Ocean Eng V6 n4 1979
A C2.5

154
Butt HG Salewski J Wagner P Grallert M Kokkinowrachos K
Test conditions at a large scale test in the vicinity of the research
platform "Nordsee"
Offshore Petroleum Conference October 1980
A D7.31

CEB-FIP
CEB-FIP Model code for concrete structures 3rd edition 1978
S1.25

Caldwell D Billington CJ
Major ship collision damage to the prestressed concrete towers of
offshore gravity structures
Integrity of Offshore Structures
2nd Symposium July 1981 Glasgow
D3.26

Cammack GF Patterson L
Problems of maintenance for offshore structures
ICE Proceedings Volume 74 August 1983
C1.26

Carlin B Morley C Nattreit A
Collision and fendering offshore
DNV Report 77-156 Nov. 1977
D3.10R

Carneiro FLLB
Codes for offshore structures design criteria and safety
requirements
Offshore Structures Engineering Conference September 1977
S1.6

Carneiro FLLB Sphaier SH
Comments on codes for design and construction of fixed steel and
concrete offshore structures
Offshore Structures Conference October 1979
S1.4

Carneiro FLLB Ferante AJ Brebbia CA
Offshore structures engineering
Proceedings of International Conference at Rio de Janeiro 77
D4.17 C1.23
Carneiro FLLB Ferrante AJ Batista RC
Offshore Engineering Volume 4
Proceedings of 4th International Symposium
Rio de Janeiro September 1983
D5.49

Carneiro FLLB Ferrante AJ Sphaier SH Brebbia CA
Offshore structures
Proceedings of International Symposium at Rio de Janeiro 79
D3.33D1.66D4.18M7.27

Carney RFA Lawrence PF Wilkins NJM
Detection of corrosion in submerged reinforced concrete
Concrete in the Oceans Technical Report No 16 1987
OTH 87 239
ISBN 011 412 8979
C2.29 D1.103

Cembureau
Cements standards of the world
May 1980
M1.23R

Cement and Concrete Association
Alkali-aggregate reaction. Working party on specification.
Minimising the risk of alkali-silica reaction guidance notes
Draft 1982/83
M2.1

Charlton RM
North sea platform operations
Proceeding Institution of Civil Engineers 1981
D7.57

Chen WF Mehta MC Chang T-YP
Experiments on axially loaded concrete shells
ASCE Journal Structures Division V105 N8 August 1979
A M7.8 D2.12
Chrapowicki KA  Boon RD
A review of methods for design against implosion of concrete cylinders in offshore structures
Concrete in the Oceans Technical Report No 13 1987
OTH 87 236
ISBN 011 4128987
D2.9

Christensen T
Reliability theory and its application in structural and soil mechanics
NATO Advanced Study Institute
D7.82

CiO
Concrete in the Oceans - coordinating report on the whole programme
Concrete in the Oceans Technical Report No. 25 1987
OTH 87 248
ISBN 011 412 9495
D7.55

Clark LA
Flexural cracking in slabs and beams
C&CA Report 42.479
D7.100

Clarke JL
Thermal stresses in concrete gravity platforms
Offshore Structures Symposium Rio de Janeiro October 1979
D4.1

Clarke JL
Thermal stresses in hollow concrete cylinders
Offshore Structures - The use of physical models in their Design Construction Press 1981
D4.8

Clarke JL  Symmons RM
Effects of temperature gradients on walls of oil storage structures
Concrete in the Oceans Technical Report No. 3. 1977
D4.5
Clarke JL Williams A
The effects of temperature gradients on the walls of concrete oil storage structures
Concrete in the Oceans Technical Report No. 11 1987
OTH 87 234
ISBN 011 4128820
1987
D4.20

Collins MP Mitchell D
Shear and torsion design of prestressed and non-prestressed concrete beams
PCI Journal Vol 25 No. 5 Oct 1980
D3.38R

Conjeaud ML
Mechanism of sea water attack on cement mortar
American Concrete Institute Publication SP 65-3
M1.11

Cookson PJ
On the effects of ship collision with concrete gravity platforms
Design Research Department - Cement and Concrete Association
April 1977
D3.29

Corish A Jackson P
Portland cement properties - past and present
Concrete Journal July 1982
M1.24

Cornelissen HAW
Fatigue failure of concrete in tension
Heron Volume 29 No 4 1984
D5.54

Crane AP
Corrosion of reinforcement in concrete construction
Ellis Horwood 1983
D1.71

Crowder JR
United Arab Emirates :: Building conditions and materials
BRE News 1983
M7.32R
Davies IL et al
Scaling laws applied to impact testing and computer assessments
made to compare tests at two scales
Proceeding 5th Conference of International Assessment for
Structural Mechanics in Reactor Technology (Berlin) 1979
D3.28R

Davies LL Mavrides A
Assessment of the damage arising from collisions between ships
and offshore structures
2nd Symposium July 1981 Glasgow
Integrity of Offshore Structures
D3.25

Department of Energy
Guidance on the design and construction of offshore
S1.27

Derrington JA
TP1 : The construction of gas treatment platform No. 1 for the
Frigg Field for Elf-Norge A/S
The Structural Engineer February 1977
C1.1

Derrington JA
Principles of design and construction for marine structures for
wave/tidal/ocean thermal energy
International Symposium on Wave and Tidal Energy University of
Kent England September 1978
A D7.39 C1.11

Derrington JA
Construction of McAlpine/Sea Tank gravity platforms at
Ardyne Point Argyll
ICE Design and Construction of Offshore Structures 1976
C1.15

Derrington JA Collard MJ Skillman JM
Seabed containment structures for hydrocarbon production
2nd International Conference on the Behaviour of Offshore
Structures August 1979
D7.10
Det Norske Veritas
Rules for the design construction and inspection of offshore structures
Det Norske Veritas 1977
S1.3

Det Norske Veritas
Impact loads from boats
Det Norske Veritas Technical Note May 1981
D3.48

Det Norske Veritas
Guidelines for the design construction and classification of floating concrete structures
Det Norske Veritas March 1979
S1.44

Det Norske Veritas
Design against accidental loads
Det Norske Veritas Technical Note TNA 101 October 1981
D3.50

Donegan E
Designing offshore structures to overcome collisions
Offshore Engineer August 1982
D3.20

Doris do Brazil C.G.
Design construction principles and setting of one type of concrete gravity platform installed on oil Fields in the North Sea Offshore Structures Conference 1979
C1.3

Dowrick DJ
Modes of failure of concrete platforms
Concrete in the Oceans Technical Report 2: 1979
ISBN 072 101142X
D7.1

Dubois F Dawance G Noel G
Determination du module de rigidite thermique sur trois betons de composition differente, soumis a des sollicitations thermiques et mecaniques Annales de l'Institut Technique du Batiment et des Travaux Publics N0 266 February 1970
R D4.25
Dupuch GW  
Micro - concrete data sheet  
Offshore Structures : The Use of Physical Models in their Design  
England 1979  
A M7.6  

Eddy CA  
Atlantic region cements : past present and future  
Performance of Concrete in Marine Environment  
St Andrews by-the-sea Canada August 1980  
ACI Pub SP 65-18  
A M1.10  

Efes Y  
Effect of cement with varying content of granulated blastfurnance slag on chloride diffusion in concrete  
Betonwerk Fertigteil - Technik Heft  
A M1.18  

Eibl IJ  
Reinforced concrete slabs and beams under impact loading  
D3.31  

Eide O Andersen KH Lunne T  
Observed foundation behaviour of concrete gravity platforms installed in the North Sea 1973 - 78  
Norwegian Geotechnical Institute Publication No. 127  
Oslo 1979  
C1.19  

Eide O Kjekstad O Brylawski E  
Installation of concrete gravity structures in the North Sea  
Norwegian Geotechnical Institute 1978  
Paper for the state-of-the-art volume of Marine Geotechnology  
C1.18  

England GL Andrews KRF Mohram A Macleod JS  
The influence of creep and temperature on the working stresses in concrete oil storage structures  
2nd International Conference on the Behaviour of Offshore Structures August 1979  
D4.4 D7.40
England GL, Moharram A
The influence of cyclic temperatures and creep on the stress prediction for concrete offshore structures
Offshore Structures Engineering Conference September 1979
D4.2

Escalante E, Cohen M, Kahn AH
Measuring the corrosion rate of reinforcing steel in corrosion
U.S. Department of Commerce NBSIR 84-2853 April 1984
D1.85

FIP
The inspection - maintenance and repair of concrete sea structures
FIP Commission on Concrete Sea Structures, August 1982
C2.20

FIP
Recommendations for the design and construction of concrete sea structures
FIP Fourth Edition 1984
S1.37

FIP
Foundations of concrete gravity structures in the North Sea
FIP State of the art report August 1979
C2.24

FIP Commission on Concrete Sea Structures
State of art report - Sea operations
Federation Internationale De La Precontrainte FIP
C1.22

FIP Working Group on Floating Concrete Vessels
Cover to steel reinforcement for floating concrete structures
Federation Internationale De La Precontrainte FIP
March 1982
D1.61

Faulds EC
Structural inspection and maintenance in a North Sea environment
Offshore Technology Conference
Houston Texas May 1982
A C2.6
Fidjestol P  Askheim NE  Roland B
Factors affecting the design for corrosion protection in concrete offshore and marine structures
Part A: Design and materials criteria for durability and corrosion resistance
Concrete in the Oceans Technical Report No. 14A 1987
OTH 87 237
ISBN 011 4128839
D.1.41

Fidjestol P  Ronning B  Roland BT
Factors affecting the design for corrosion protection in concrete offshore and marine structures
Part B: Criteria for cover and crack control in the permanently wet part of such structures
Concrete in the Oceans Technical Report No 14B 1987
OTH 87 237
ISBN 011 412 8839
D.1.77

Fidjestol P  Nilsen N
Field test of reinforcement corrosion in concrete
American Concrete Institute Publication SP 65-12
M.6.6

Fidjestol P  Ronning B  Roland BT
Revised criteria for prevention of corrosion in offshore concrete structures
Corrosion 85 Conference March 1985
D.1.97

Fidjestol P  Ronning B  Roland BT
Criteria for cover and crack control in the fully submerged part of marine concrete structures
Concrete-in-the-Oceans Final Report PA2 1985
D.1.87

Figg JW  Marsden AF
Development of inspection techniques for reinforced concrete: a state of the art survey of electrical potential and resistivity measurements for use above water level
Concrete in the Oceans Technical Report No. 10 1984
OTH 84 205
ISBN 011 4119120
D.1.42
Fjeld S
Code calibration
Det Norske Veritas Industrial and Offshore Division
D7.76

Fjeld S Roland B
In-service experience with eleven offshore concrete structures
Offshore Technology Conference May 1982 Houston Texas
D1.26

Fjeld S Furnes O Hansvold C Roland B Blaker B Morley C
Special problems in structural analysis and design of offshore concrete platforms
Offshore Technology Conference May 1978 Volume 1
D7.8

Flint AR
Rationalisation of safety and serviceability factors in structural codes
CIRIA Report No 63 July 1977
D7.70

Flint AR Baker MJ
Rationalisation of safety and serviceability factors in structural codes - Supplementary report on offshore structures
CIRIA October 1976
D7.69

Frankel EG
Design and operation of offshore gravity and displacement terminals
Annual Combined Conference of IEEE & Marine Technology September 1979
A D7.38

Furnes O
Buckling of reinforced and prestressed concrete members - simplified calculation methods
Journal Prestressed Concrete Institute V26 No. 4
July-August 1981
A D7.21
Furnes O
Model tests of concrete capped cylinders
Offshore Structures : The Use of Physical Models in their Design
England 1979
A D2.10

Furnes O
Instability of plane and curved concrete walls in the design of
gravity offshore platforms
Offshore Petroleum Conference London October 1978
A D7.42

Furnes O
Concrete and other alternative platform designs
Internation Meeting on Petroleum Engineering Benjing China
March 1982
A D7.45

Furnes O
Instability of plane and curved concrete walls in the design of
gravity offshore platforms
Det Norske Veritas 1978
D7.81

Furnes O  Amdahl J
Computer simulation study of offshore collisions and analysis of
ship-platform impacts
Offshore Structures Conference October 1979
D3.4

Furnes O  Loset O
Shell structures in offshore platforms application and design
World Congress on Shell & Spatial Structures Madrid
Sept 1979
A D2.14

Furnes O  Oystein L
Shell structures in offshore platforms : Design and application
(In English)
Engineering Structures V3 No. 3 July 1981
A D7.14
Garas FK
Research and development in support of the design of a prestressed concrete pressure vessel for a working pressure of 10000 psi
International Journal of Pressure Vessels Piping V8 No. 3
May-June 1980
A D7.37

Garrison CJ  Chow PY
Wave forces on submerged bodies
ASCE Journal of Waterways Harbours and Coastal Engineering Division Aug. 1972
D3.1

Gautesfall O  Vennesland O
Effects of cracks on the corrosion of embedded steel in silica-concrete
Cement and Concrete Research Institute
D1.96

George CM
Long term and accelerated tests of the resistance of cements to sea water with special reference to aluminous cements
American Concrete Institute Publication Sp 65-19
M1.13

Gerwick BC
Cyclic shear capacity of offshore concrete structures in sea environments
Offshore Structure Conference October 1979
D5.2

Gerwick BC
Design and construction of concrete sea structures
International Ocean Development Conference Tokyo Japan September 1978
C1.8 D7.35

Gerwick BC
Research requirements for concrete in marine environments
Performance of Concrete in Marine Environment St Andrews by-the-sea Canada August 1980 ACI Pub SP-65
A D7.49
Gerwick BC
Lessons from an exciting decade of concrete sea structures
Concrete Internation August 1985
D7.95 M7.47

Gerwick BC  Venuti WJ
High and low cycle fatigue behaviour of prestressed concrete in offshore structures
Offshore Technology Conference May 1979 Houston Texas
D5.11

Gerwick BS
Lessons from an Existing Decade of Concrete Sea Structures
Concrete Internation August 1985
D7.95 M7.47

Gibson JE
Interaction of cylindrical container units
The use of physical models proceedings Garston 1979
A D7.22

Gifford and Partners
Review of fatigue in concrete marine structures
Concrete in the Oceans Technical Report No. 12 1987
OTH 87 235
ISBN 011 4129266
D5.4

Gjørv OE
Durability of concrete structures in the ocean environment
FIP Symposium Concrete Sea Structures Sept 1972
D1.8

Gladkov VS  Goncharov AA
Freeze-thaw creep of concrete in compression in sea water
FIP Symposium - Concrete Sea Structures Sept 1972
D4.3

Graff WJ  Chen WF
Bottom supported concrete platforms : Overview
ASCE Journal Structures Division VI07 N6 June 1981
A D7.30
Gustafsson PJ
Analysis of shear strength of reinforced concrete beams
Lund Institute of Technology 1982
D3.35

Gutt WM Harrison WM
Chemical resistance of concrete
(Reprinted from) 'Concrete' 1977
R D1.53

Gylltoft K
A fracture mechanics model for fatigue in concrete
Materials and Structures Research and Testing
January 1984 No. 97
A D5.32

Hale KF Hockenhull BS Christodoulou G
Application of optical fibres as witness devices for the
detection of plastic strain and cracking
Strain V16 N4 Oct 1980
A D1.17 D7.29

Hallingstad OK Olsen TO Stove ØJ
Practical design of reinforcement in plates & shells including
effects of reinforcement directions crack widths
2nd International Conference on the Behaviour of Offshore
Structures August 1979
D2.6

Hamadi RP
Behaviour in shear of beams with flexural cracks
Magazine of Concrete Research Vol 32 No 111 June 1980
D3.41R

Harris AJ Fox BM
Adjustment of structural concrete techniques to offshore
conditions
2nd International Conference on the Behaviour of Offshore
Structures. August 1979
D7.9
Harrison JD  Pisarski HG
Background to Guidance Notes sections dealing with
structural steels and construction
The Welding Institute July 1985
D7.93 C1.30

Hawkins M
Fatigue considerations for offshore concrete structures.
Reinforcements
Materials and Structures Research and Testing
January 1984 No. 97
A D5.30

Hawkins MR et al
Minimising the risks of alkali-silica reaction - Guidance Notes
Report of a Working Party
Cement and Concrete Association 1983
M7.30R

Haynes HH
Permeability of concrete in sea water
American Concrete Institute Publication Sp 65-2
M7.12

Haynes HH
Design for implosion of concrete cylinder structures under
hydrostatic loading
Civil Engineering Lab (Navy) Port Hueneme CA Aug 1979
D2.17R

Haynes HH
Handbook for designer of undersea pressure resistant concrete
structures
CEL NCBC Port Huenema California Sept 1979
D2.20R

Haynes HH  Highberg RS
Long term deep ocean test of concrete spherical structures
Civil Engineering Laboratory January 1976
D7.65R

169
Haynes HH Highberg RS
Concrete properties at ocean depths
ASCE Jnl. Waterways Harbours and Coastal Eng
November 1976 Vol 102
M7.34

Haynes HH Highberg RS
Deep ocean study of concrete spheres
Proceedings 8th Congress FIP London May 1978
M7.36

Haynes HH Highberg RS
Long-term deep ocean test of concrete spherical structures -
Results after 6 years
Civil Engineering Laboratory Technical Report R869
January 1979
M7.37

Haynes HH Underbakke LD
Compressive strength of freshly mixed concrete placed cured
and tested in the deep ocean
Civil Engineering Laboratory February 1981
M7.35

Haywood JH
Ship collision with fixed offshore structures
ATMS Report 78282 January 1978
D3.42R

Heiman JL
Corrosion problems in reinforced concrete structures in
marine environments
Symposium On Concrete 1981 Adelaide Australia June 1981
D1.34

Highberg RS Haynes HH
Predicting the maximum ocean depths for submerged concrete
structures
European Offshore Petroleum Conference October 1978 Eur 30
D2.22

Hildebrand H Schulte M Schwenk W
Corrosion characteristics of steel in cement mortar under
cathodic polarization in seawater and M NaCl solution
1983
C2.33 D1.100
Hjelde E Thebault J
Use of instrumentation results as a basis to extend
the platform service life.
Elf Aquitaine Norge A/S
D7.91

Hobbs C
Eurocode
Concrete Journal April 1983
S1.42R

Hobson LJ
Quality assurance in offshore projects
Solu Ocean Systems
M7.28 C1.24

Hodgkiess T Arthur P D
Fatigue and corrosion effects in reinforced concrete
beams partially submerged in seawater and subjected to
reverse bending
Concrete in the Oceans Technical Report No. 19 1987
OTH 87 242
ISBN 011 412910X
D5.26

Hoff GC
The Challenge of offshore concrete structures
Concrete International August 1985
D7.94 C1.31 M7.46

Hoff GC
The service record of concrete offshore platforms in
the North Sea
Mobil Research and Development Corporation - October 1985
C2.35

Hogan FJ Meusel JW
Evaluation for durability and strength development of ground
granulated blast furnace slag
American Society for Testing and Research 1981
A M7.16
Holden WR et al
The influence of chlorides and sulphates on durability
Corrosion of reinforcement in concrete construction Editor Alan P Crane
Ellis Horwood 1983
D1.71

Holm TA
Performance of structural lightweight concrete in a marine environment
Performance of Concrete in Marine Environment
St Andrews by-the-sea Canada August 1980 ACI Pub SP-65
A D7.48

Holmen JO
Fatigue design evaluation of offshore concrete structures
Materials and Structures Research and Testing
January 1984 No. 97
A D5.35

Horii O Ueda S
Study on fatigue behavior of offshore concrete structure
Offshore Technology Conference 9th Annual Houston
May 1977
D5.37

Hsu Thomas TC
Fatigue and microcracking of concrete
Materials and Structures Research and Testing
January 1984 No. 97
A D5.33

Hughes BP
Control of thermal and shrinkage cracking in restrained reinforced concrete walls
CIRIA Technical Note 21 June 1971
R D4.27

ICE
Offshore structures
Proceedings of the Conference London October 1974
D7.60
ICE
Maintenance of marine structures
R C1.17

Idorn GM
Durability of concrete structures in Denmark
M7.31R

Jensen JJ et al
Investigations of offshore concrete structures with respect
to static strength - Summary report
COSMAR Report No. PP1-4-1
May 1981
D7.101

John Laing Research & Development Ltd
Fatigue strength of reinforced concrete in seawater
Concrete in the Oceans Technical Report No. 7 1981
ISBN 072 1012671
D5.21

Kavrychine M
Fissuration du beton sous gradient thermique
Chapter 2 of Recherche sur les structures en beton
Annales de l'Institut Technique du Batiment et des Travaux Publics
Paper 177 No 300 April 1978
R D4.24

Kennedy RP
A review of the procedures for the analysis and design of
concrete structures to resist missile impact effects
ELCALAP Seminar Berlin 1975
D3.43R

King RA Nabizadeh H Ross TK
Cathodic protection of steel in concrete saturated with seawater
Corrosion Prevention and Control April 1977
C2.34 D1.101

Kowalski TG
Ferrocrete marine mixes in a warm and humid environment
FIP Symposium - Concrete Sea Structures Sept 1972
M1.2
Kudzis A
Prestressed polymer-cement concrete for sea structures
FIP Symposium - Concrete Sea Structures Sept 1972
M1.3

Kumar Metha P
Durability of concrete in marine environment - A review
ACI Publication SP-65
D1.36

Larsen C M
Ship collision and fendering of offshore concrete structures
European Offshore Petroleum Conference London October 1978
A D3.17

Lea FM
The chemistry of cement and concrete
Third Edition 1970
M1.21 M7.33R

Ledoigt B  Angello G
Dynamic behaviour of an offshore concrete platform
Offshore Petroleum Conference October 1980
A D7.32

Leeming MB
Surveys of existing concrete marine structures
Part C: Surveys of existing structures containing pulverised fuel ash
Concrete in the Oceans Technical Report No. 21 Part C 1987
OTH 87 244
ISBN 011 412941X
M1.20

Leeming MB
Concrete in the Oceans - coordinating report on the whole programme
Concrete in the Oceans Technical Report No 25 1987
OTH 87 249
ISBN 011 4129193
S1.47
Lefevre AA, Vinckier AG
Testing of steel reinforcing bars used in the concrete containment structure for LNG storage tanks
Symposium on Engineering in Marine Environment: Brugges Belgium May 1982
A M6.1

Leick RD, Bode JH
Implosion strength of cylindrical concrete shells. A comparison of theoretical and experimental results
Offshore Technology Conference May 1978 Volume I D2.1

Lenschow R
Serviceability state of marine structures with emphasis on cracking
Offshore Structures Engr Rio de Janeiro October 1979 D1.27

Leonard TE
Design fabrication and installation of the North Cormorant Platform
Petroleum Conference London October 1982 A D7.44 C1.13

Lin CY
Bond deterioration due to corrosion of reinforcing steel
ACI American Concrete Institute Publication SP 65-15 M6.7 D1.104

Lindgren J
Condeep construction
PIANC Norwegian Section Meeting 1977 C1.2

Lloyd JP, Heidersbach RH
Use of the scanning electron microscope to study cracking and corrosion in concrete
Concrete International May 1985 D1.102
Lloyds Register
A review of methods for design against implosion of concrete cylinders in offshore structures
Concrete in the Oceans Technical Report No 13 1987
OTH 87 236
ISBN 011 4128987
D2.7

Lloyds Register
Draft rules for concrete structures
Lloyds Register of Shipping
S1.31

Lloyds Register
Rules and regulations for the classification of mobile offshore units
Lloyds Register of Shipping June 1984
S1.50

Lloyds Register
Boat impact study
Department of Energy - Offshore Technology Report June 1985
OTH 85 224
ISBN 011 41228545
D3.52

Long JE
Experience in the prestressing grouting concrete offshore structures
ICE Design and Construction of Offshore Structures
M5.3

Lunne T Kvalstad TJ et al
Analysis of full scale measurements on gravity platforms
Norwegian Geotechnical Institute/Det Norske Veritas 1982
D7.90

Makita Moriy Katawaki K
Marine corrosion behaviour of reinforced concrete exposed at Tokyo Bay
American Concrete Institute Publication SP-65
A D7.52
Makita M  Mori Y  Katawaki K
Performance of typical protection methods of reinforced concrete in marine environment
American Concrete Institute Publication SP65
A D7.53

Malhotra VM  Carette GG  Bremner TW
Durability of concrete containing granulated blast furnace slag or fly ash or both in marine environment
Minerals Research Program - Mineral Sciences Laboratories
Energy Mines and Resources Canada June 1980
M1.19

Malhotra VM  Carette GG  Bremner TW
Durability of concrete in marine environment containing granulated blast furnace slag - fly ash or both
American Concrete Institute Publication SP-65
D1.38

Mather B
Concrete in sea water
Concrete International March 1982
M7.20  A

Maxson OG  Achenbach GD
Properties of concrete in contact with pressurized hydrocarbons and sea water
Journal of Petroleum Technology April 1974
M7.39

McAlpine Sea Services Ltd
Classification and identification of typical blemishes visible on the surface of concrete underwater
Concrete in the Oceans Technical Report No. 9 1985
OTH 84 206
ISBN 011 4119031
C2.36

McLeod IL
Construction of the Ninian Central platform for the British North Sea
Offshore Technology Conference April/May 1979 Houston Texas
C1.12
Meyer A
Investigations on the carbonation of concrete
A M7.22

Miranda de Camargo W
Research of a rupture envelope for concrete strength to be
adopted on deep water concrete structures design
Offshore Structures Conference October 1979
D7.6

Moksnes J
Condeep platforms for the North Sea - some aspects of concrete
technology
Offshore Technology Conference May 1975
M1.1

Moksnes J
Quality assurance for concrete platforms in North Sea oil fields
Concrete International
A C1.6 M7.1

Moksnes J
Admixtures in offshore structures
Admixtures London April 1980
A M4.1

Moksnes J
Offshore concrete - Recent developments in concrete mix design
Reprint Nordisk Betong 2-4 : 1982
M7.45

Moksnes J Jakobsen B
High strength concrete development and potentials for platform
design
Offshore Technology Conference May 1985
M7.44

Montcrieff MLA Waggott JG
Time, temperature, creep and shrinkage in concrete
Conference on Prestressed Concrete Pressure Vessels London 1969
R D4.20
Muir Wood AM
Foundation engineering for offshore structures
Sir William Halcrow and Partners BOSS79
D7.64

Murata J Kawasaki M Minami M
Mechanical characteristics of ferrocement and application to
offshore structures
International Symposium on Ferrocement Bergamo Italy 1981
A M1.9

NDRC
Effects of impact and explosion
U.S. National Defense Research Committee Vol 1
Summary Report 1946
D3.44R

NGI/DnV
Analysis of full scale measurements on gravity platforms
NGI/DnV 1982
D7.77

NPD
Regulations for the structural design of fixed structures on
the Norwegian continental shelves 1976
Norwegian Petroleum Directorate 1976
D7.18 S1.9

Nagataki S Kodama K Okumura T
Application of high-strength prepacked concrete to offshore
structures
Offshore Structures Conference October 1979
D7.7

Neville AM
Properties of concrete
M7.41R

Nichols JH Westby KA
Innovative engineering makes Maureen development a reality
Offshore Petroleum Conference October 1980
A D7.34
Nilsen N Espelid B
Corrosion behaviour of reinforced concrete under dynamic loading
Corrosion 85 Conference March 1985
D1.95

Nixon PJ
Changes in Portland cement properties and their effect on concrete
BRE Information Paper IP3/P6 1986
M1.26

Northcott GDS et al
Changes in cement Properties and their effects on concrete - Working Party report
Concrete Society September 1984
M1.25

Norwegian Contractors
Gullfaks A platform - Construction tolerances GBS general 1984
C1.29

Nowak AS Lind NC
Practical bridge code calibration
ASCE December 1979
S1.54 D7.85

O'Neil EF
Study of reinforced concrete beams exposed to marine environment
American Concrete Institute Publication SP 65-8
D1.37 M6.5

Offshore Certification Bureau
Certification manual
Offshore Certification Bureau January 1981
S1.24

Offshore Certification Bureau
Fatigue calibration study
Offshore Certification Bureau April 1986
D5.55
Offshore Engineer
Scandinavia review
April 1983
D7.56

Offshore Engineer
Concrete platforms in the North Sea
Offshore Engineer April 1984
S1.41

Offshore Research
Repairing concrete structures
Offshore Research Focus No. 4 December 1983
C1.25 C2.21

Okada K Miyagawa T
Chloride corrosion of reinforcing steel in cracked concrete
Performance of Concrete in Marine Environment
St Andrews by-the-sea Canada August 1980 ACI Pub SP 65-14
A D1.31 M6.2

Okada K Miyagawa T Koyanagi JP
Chloride corrosion of reinforcing steel in cracked concrete
Offshore Structures Conference October 1979
D1.6

Olesen SO Sorensen KA
Marine betonkonstruktioners stodbaereevne
BKF Centra len Report 070 1978
D3.45R

Osami Horn and Sigeru Veda
Study of fatigue behaviour of offshore concrete structure
9th Offshore Technology conference 1977 OTC 3005
D5.25

Ove Arup and Partners
Development of inspection techniques for reinforced concrete:
A state of the art survey of electrical potential and resistivity measurements
Concrete in the Oceans Technical Report No. 10 1984
OTH 84 205
ISBN 011 4119120
C2.17
PI Group  
Shell Brent B - Instrumentation project final report -
Load/response system
October 1978  
C1.28A

Page CL  Short NR  Tarras AE  
Diffusion of chloride ions in hardened cement pastes  
Cement & Concrete Research Volume II 1981  
A M1.17

Page CL  Cunningham PJ  
Electrochemical methods of corrosion monitoring for
marine concrete structures  
Concrete in the Oceans Technical Report No 22 1987  
OTH 87 245  
ISBN 011 4129061  
D1.84 C2.26

Parrott LJ  Symmons RM  
Deformation properties of an oil storage vessel concrete subject
to fluctuating stresses and temperatures  Report P3/14  
CIRIA Underwater Engineering Group  December 1977  
Concrete in the Oceans Technical Report  
D4.16

Paterson WS  Dill MJ  Newby R  
Fatigue strength of reinforced concrete in seawater  
Concrete in the Oceans Technical Report 7 : 1982  
D5.1

Paterson WS  Dill MJ  
Fatigue strength of reinforced concrete in seawater - results from Phase II  
Concrete in the Oceans Technical Report No 20 1987  
OTH 87 243  
ISBN 011 4129029  
D5.52

Plank G  
General report and review of contributions  
Materials and Structures Research and Testing  
January 1984  No. 97  
D7.13R
Pliskin L  
Use of physical models in the design of offshore structures  
The Use of Physical Models. Proceedings Garston 1979  
A D7.24

Price WJ  Hambly EC  Tricklebank AH  
Review of fatigue in concrete marine structures  
Concrete in the Oceans Technical Report No. 12 1987  
OTH 87 235  
ISBN 011 4129266  
D5.19

Priedeman JS  Anderson TR  
The first ten years: Floating concrete structures  
Concrete International August 1985  
D7.96 M7.48

Pullar-Strecker P  Sharp JV  
Concrete in the Oceans Phase II prospectus  
CIRIA Underwater Engineering Group  May 1979  
S1.39

Pyman MAF  Austin JS  Lyon PT  
The risks of collision between merchant shipping and offshore platforms in the UK sector of the North Sea  
PIANC Bulletin No. 44 1984  
D3.49

Rabbit BG  Corley WG  
Long time fatigue properties of high yield reinforcing bars  
Materials and Structures Research and Testing  
January 1984  No. 97  
A D5.36

Raithby K D  
Flexural fatigue behaviour of plain concrete  
Fatigue of Engineering materials and structures Vol 2 1979  
D5.22

Regan PE  Hamadi YD  
Behaviour of concrete caisson and tower members  
Concrete in the Oceans Technical Report 4 : 1981  
ISBN 072 1012027  
D7.2
Regourd M
Physico-chemical studies of cement pastes mortars and concretes exposed to sea water
American Concrete Institute Publication SP-65
A M1.12

Reinhardt HW
Unaxial impact tensile strength of concrete
Offshore Structures Conference University of Rio de Janeiro
October 1979
D3.2

Reynolds GC
Bond strength of deformed bars in tension
Cement and Concrete Association Technical Report 548
May 1982
D7.71

Richmond B
The time-temperature dependence of stresses in offshore concrete structures
ICE Design and Construction of Offshore Structures 1976
D4.10

Richmond B England GL Bell TA
Designing for temperature effects in concrete offshore oil containing structures
D4.14

Rider RG Heidersbach RH
The effects of sea water on the structural properties of metal-fibre reinforced concrete
Offshore Technology Conference 1978
D1.74

RILEM Committee 36 - RDL
Long term random dynamic loading of concrete structures
Materials and Structures Research and Testing
Jan/Feb 1984 No. 97
D5.38
Roland B Olsen TO Shkaare E
Ship impact on concrete shafts
European Petroleum Conference London October 1980
A D3.18

Roper H
Durability of concrete marine structures
6th International Congress on Marine Corrosion and Fouling
Athens September 1984
D1.63

Roren EMQ Hove K Foss I Olsen O
Concrete gravity structures for petroleum production offshore -
merits and problems
Oceanology International March 1975
D7.3

Ross AD England GL Suan RH
Prestressed concrete beams under a sustained temperature crossfall
Magazine of Concrete Research Vol 17 No 52 September 1965
R D4.23

Rowe RE et al
Handbook on the unified code for structural concrete
Cement and Concrete Association 1974
D7.80 S1.51

Roy DM Idorn GM
Hydration - structure - and properties of blast furnace slag -
cements - mortars and concrete
American Concrete Institute Journal November-December 1982
A M7.17

Runge KH
Review of ACI Committee 357 Report : Guide for the design and
construction of fixed offshore concrete structures
Offshore Structural Engineering Rio de Janeiro October 1979
A D7.47 C1.14

Runge KH Haynes HH
Experimental implosion study of concrete structures
FIP Eight Congress London May 72
D2.21R
Saetre O, Jensen F
Developments in cathodic protection of offshore concrete structures
Corrosion/81 International Corrosion Forum Toronto
Ont. Canada April 1981
A D1.89

Sandberg A, Collis L
Toil and trouble on concrete bubbles
Consulting Engineer August 1982
M7.25

Sandberg A, Collis L
Current concrete conundrums
Consulting Engineer August 1982
M7.24

Schiesel I
Corrosion of the reinforcement and its effect on concrete durability
November 1981
R D1.52 M6.12

Shah SP
Predictions of cumulative damage for concrete and reinforced concrete
Materials and Structures Research and Testing
January 1984 No. 97
A D5.31

Sharp JV, Pullar-Strecker P
A description of the 'Concrete-in-the-Oceans' research programme
Concrete Ships and Floating Structures Convention
S1.40

Shimada H, Okada H, Nishi H
Development of sea water corrosion resistant steel bars for offshore concrete structures
Offshore Technology Conference 3194 1787
D1.76 M6.17

Shirley DE
Impurities in concreting aggregates
Construction Guide
A M2.3
Shreir LL
Electrochemical principles of corrosion - A guide for engineers
Department of Industry Publication 1982
D1.65

Shreir LL et al
Report of the cathodic protection study group
Department of Energy Report No CPSG/P60
December 1983
D1.94

Sides GR  Mueller HR
Installing prestressing tendons in arduous conditions
11th Congress - IABSE Vienna August 1980
A M6.4

Sighjørnsen R  Smith EK
Wave induced vibrations of gravity platforms - A stochastic theory
Applied Math Modelling V4 N3 June 1980
A D7.36

Sjursen A  Lenschow R
Model tests of cylindrical storage cell for Statfjord A Platform
Offshore Structures : The use of Phys Models in their Design
England 1979
A D3.13

Slæken T
Quality control of the concrete platform "Statfjord B"
Controle de Qual des Struct. en Beton. V2 Stockholm
Sweden June 1979
A C1.9

Slater JE
Corrosion of metals in association with concrete
ASTM Special Technical Publication 818
M6.15

Society of Chemical Industry
Corrosion of steel reinforcement in concrete construction
R D1.57
Somerville G  
The design life of concrete structures  
The Structural Engineer February 1986  
D7.99

Sonoda K  Korikawa T  
Fatigue strength of reinforced concrete states under moving loads  
D5.51

Stillwell JA  
Exposure tests on concrete for offshore structures  
Concrete in the Oceans Technical Report No 8 : 1983  
ISBN 072 1012744  
D1.4

Stillwell JA  
Exposure tests on reinforced concrete in seawater  
Concrete in the Oceans Technical Report No 23 1987  
OTH 87 246  
ISBN 011 412924X  
D7.74 D1.105

Straube P  
evaluation of hydrocrete for use as reinforced concrete in offshore structures  
DnV for Underwater Concrete Ltd March 1983  
M1.4

Subedi NK  Gares FK  Armer GST  
Model behaviour of square cell cellular concrete elements  
Offshore Structures : The use of Physical Models in their design : England 1979  
A D7.23

Suhara T  Nishimaki K  Matsuishi M  Iwata S  
On the strength of composite steel concrete structures of Sandwich system  
Naval Architecture Ocean Engineering V18 1980  
A M7.5 D7.20
Sullivan PJE  Newman JB  McLeish A
Durability of concrete in marine applications - A factorial
approach to experimentation
Conference on the behaviour of Offshore Structure
August 1979
D1.9

Taylor HPJ  Sharp JV
Fatigue in offshore concrete structures
The Structural Engineer  March 1978 Volume 56A Number 3
D5.42

Taylor Woodrow
Marine durability survey of the Tongue Sands Tower
Concrete in the Oceans Technical Report 5 : 1980
ISBN 072 1012078
D1.2

Taylor Woodrow
Surveys of existing concrete marine structures
Concrete in the Oceans Technical Report No 21
Parts A, B and C 1987
OTH 87 244
ISBN 011 412941X
D1.40

Taylor Woodrow
Research proposal Project 2 - Implosion
Concrete in the Oceans phase II April 1980
D2.19

Taylor Woodrow
Effectiveness of concrete to protect steel from
corrosion in marine structures
Concrete in the Oceans Technical Report No 24 1987
OTH 87 247
ISBN 011 4129401
D1.80

Taylor Woodrow
The development of methods for structural reinstatement of
concrete structures - Tests on underwater trial model
Taylor Woodrow Research Labs April 1983
C2.31 M7.43

189
Taylor Woodrow
The development of methods for structural reinstatement of concrete structures - Laboratory repair trial
Taylor Woodrow Research Labs March 1983
C2.30 M7.42

Tepfers R Hedberg B Szezekocki G
Absorption of energy in fatigue loading of plain concrete
Materials and Structures Research and Testing
January 1984 No. 97
A D5.29

Tilly GP
Fatigue of steel reinforcement bars in concrete: A review
Fatigue of Engineering Materials and Structures Vol 2 1979
D5.20 M6.10

Tilly GP
Fatigue testing and performance of steel reinforcement bars
Materials and Structures Research and Testing
January 1984 No. 97
A D5.34

Tilly GP
Dynamic behaviour of concrete bridges
RILEM Symposium
D7.82

Tilly GP et al
Dynamic behaviour of concrete structures - Recommendations of good practice for methods of testing and design
RILEM 65M6B Committee 1984
D7.83

Timoshenko S Goodier JN
Theory of Elasticity
McGraw Hill 1970
R D4.28

Treadaway KWJ Page CL
The durability of steel in concrete
BRE News No. 61 1984
R M6.19
Tuthill LH
Performance failures of concrete materials and of concrete as a material
Concrete International January 1980
A M7.19

Tuutti K
Service life of structures with regard to corrosion of embedded steel
Performance of Concrete in Marine Environments
St Andrews by-the-sea Canada August 1980 ACI Pub SP 65-13
A D1.32 M6.3

Tuutti K
Corrosion of steel in concrete
Swedish Cement and Concrete Research Institute
R D1.54

UEG
Concrete as an offshore material
Marine Engineering Review November 1982
A D7.16 M1.5

UEG
Applications for concrete offshore
Report UR20
M7.15

Van Leeuwen J Siemes AJM
Miner's rule with respect to plain concrete
2nd International Conference on the behaviour of Offshore Structures August 1979
D5.3

Vanden Bosch VD
Performance of mortar specimens in chemical and accelerated marine exposure
Performance of Concrete in Marine Environment
A M5.2
Vassie PR
A survey of site tests for the assessment of corrosion in reinforced concrete
Transport and Road Research Laboratory
Department of the Environment Department of Transport
D1.62

Waagaard K
Design recommendations for offshore concrete structures
IABSE Colloq Lausanne 1982 Fatigue of Steel & Concrete Structures
D5.13

Waagaard K
Fatigue of offshore concrete structures - Design and experimental investigations
Offshore Technology Conference 1977
D5.53

Waagaard K
Experimental investigation on the fatigue strength of offshore concrete structures
Offshore Operations Symposium - ASME 1986
D5.56

Waagaard K et al
Fatigue strength of offshore concrete structures
COSMAR Report No. PP2-I
April 1981
D5.57

Watson L Machemehl J Barnes B
Deterioration of asbestos cement sheet material in the marine environment
Coastal Structures 1979
A M7.9

Watt BJ et al
Earthquake survivability of concrete platforms
Journal petroleum Technology V32 N6 June 1980
A D7.41
Weirne SH
A review of reports of failure
Mechanical Engineering 1979
Proceeding Institution of Mechanical Engineers
D7.54

Weibenga JG
Durability of concrete structures along the North Sea coast of the Netherlands
Performance of Concrete in Marine Environment
St Andrews by-the-sea Canada August 1980 ACI Pub SP-65
ISBN 072 1012086
A D1.30

Wilkins NJM Lawrence PF
Fundamental mechanisms of corrosion of steel reinforcement in concrete immersed in sea water
Concrete in the Oceans Technical Report 6 : 1980
ISBN 072 1012086
D1.3

Wilkins NJM Lawrence PF
The corrosion of steel reinforcements in concrete immersed in seawater
Corrosion of reinforcement in concrete construction Editor Alan P Crane
Ellis Horwood 1983
D1.72

Wilkins NJM Lawrence PF Carney RFA Warburton JB
Fundamental mechanisms of corrosion of steel reinforcement in concrete immersed in seawater - Results from Phase II
Concrete in the Oceans Technical Report No 15 1987
OTH 87 238
ISBN 011 4128995
D1.81

Wills J
Epoxy coated reinforcement in bridge decks
Transport and Road Research Laboratory
R M6.13
Wimpey Laboratories Ltd
Exposure tests on concrete for offshore structures
Concrete in the Oceans Technical Report No. 8
ISBN 072 1012744
1983
D1.11

Wimpey Laboratories Ltd
The assessment of impact damage caused by dropped objects
on concrete offshore structures
Concrete in the Oceans Technical Report No 17A 1987
OTH 87 240
ISBN 011 412398
D3.12

Wimpey Laboratories Ltd
Repair of major damage to the prestressed concrete towers
of offshore structures
Concrete in the Oceans Technical Report No. 27 1987
OTH 87 250
ISBN 011 4129231
D3.19 C2.18

Woisin G  Gerlach W
On the estimation of forces developed in collision between
ships and offshore lighthouses
D3.30

Zaleski-Zamenhof LC Doris CG
Limit state approach of buckling capacity of
concrete sea structures
Boss 1976
D7.84