FOREWORD

DET NORSKE VERITAS (DNV) is an autonomous and independent foundation with the objectives of safeguarding life, property and the environment, at sea and onshore. DNV undertakes classification, certification, and other verification and consultancy services relating to quality of ships, offshore units and installations, and onshore industries worldwide, and carries out research in relation to these functions.

DNV service documents consist of amongst other the following types of documents:

— Service Specifications. Procedual requirements.
— Standards. Technical requirements.

The Standards and Recommended Practices are offered within the following areas:

A) Qualification, Quality and Safety Methodology
B) Materials Technology
C) Structures
D) Systems
E) Special Facilities
F) Pipelines and Risers
G) Asset Operation
H) Marine Operations
J) Cleaner Energy
O) Subsea Systems
CHANGES

• General
As of October 2010 all DNV service documents are primarily published electronically.

In order to ensure a practical transition from the “print” scheme to the “electronic” scheme, all documents having incorporated amendments and corrections more recent than the date of the latest printed issue, have been given the date October 2010.

An overview of DNV service documents, their update status and historical “amendments and corrections” may be found through http://www.dnv.com/resources/rules_standards/.

• Main changes
Since the previous edition (April 2007), this document has been amended, most recently in April 2009. All changes have been incorporated and a new date (October 2010) has been given as explained under “General”.

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A. General

101 This offshore standard provides principles, technical requirements and guidelines for the design, construction and in-service inspection of Offshore Concrete Structures. The Concrete Structures may be a floating or gravity based structure.

102 This standard shall be used together with the general offshore design standards for steel structures DNV-OS-C101, DNV-OS-C102, DNV-OS-C103, DNV-OS-C105 and DNV-OS-C106. These standards cover a wide range of different structures.

103 The standard covers design, fabrication/construction, installation and inspection of offshore concrete structures.

104 Design, construction, inspection and maintenance of Offshore Concrete LNG Terminals are covered by DNV-OS-C503.

105 For design and construction of offshore concrete wind turbines, reference is made to DNV-OS-J101 “Design of Offshore Wind Turbines Structures”.

106 This standard cover design of platforms/structures for oil production, oil storage, harbours and deep water foundation of bridges where reinforced and prestressed concrete is used as structural material.

For structures like deep water foundation of bridges and harbours special considerations will be required on load specifications.

200 Objective

The objectives of this standard are to:

— provide an international standard for the design, construction and in-service inspection of Offshore Concrete Structures with an acceptable level of safety by defining minimum requirements for design, construction control and in-service inspection.

— serve as a contractual reference document between supplier and purchasers related to design, construction and in-service inspection.

— serve as a guideline for designer, supplier, purchasers and regulators.

A 300 Scope and applications

The standard is applicable to Design, Construction, Inspection and Maintenance of Offshore Concrete Structures, using concrete as the structural material in the support structure as defined in 302 below.

The standard can be used in the structural design of the following types of support structures:

— GBS (Gravity Based Structures) offshore concrete structures for oil/gas production

— GBS structures for oil/gas production with oil storage facility

— Floating concrete structures for production of oil/gas. The structure may be of any type floating structure, i.e. tension leg platform (TLP), column stabilised units and Barge type units

— Concrete harbours

— Artificial concrete export/import harbours, either floating or fixed and with storage facilities, allowing transport of articles and goods with small boats to the artificial harbour for reloading on large sea going vessels. Cargo may be different types or ore or oil/gas

303 Appendices A to F are appended to the standard. These appendices contain guidelines for the design of Offshore Concrete Structures.

304 Floating Offshore Concrete structures shall be designed with freeboard and intact stability in accordance with DNV-OS-C301. For temporary phases the stability shall be in accordance with DNV Rules for Planning and Execution of Marine Operations.

305 The development and design of new concepts for Offshore Concrete Structures requires a systematic hazard identification process in order to mitigate the risk to an acceptable risk level. Hazard identification is therefore a central tool in this standard in order to identify hazards and mitigate these to an acceptable risk level.

A 400 Non DNV codes and standards

In case of conflict between the requirements of this standard and a reference document other than DNV standard, the requirement of this standard shall prevail.

The provision for using non-DNV codes or standards is that the same safety level as provided by this DNV standard, is obtained.

Where reference is made to non-DNV codes, the valid revision shall be taken as the revision which is current at the date of issue of this standard, unless otherwise noted.

In addition to the requirements mentioned in this standard, it is also the responsibility of the designer, owner and operator to comply with additional requirements that may be imposed by the flag state or the coastal state or any other jurisdictions in the intended area of deployment and operation.

A 500 Classification

Classification principles, procedures and application class notations related to classification services of offshore units are specified in the DNV Offshore Service Specifications given in Table A1.

B 100 General

The DNV documents in Tables B1 and B2 and recognized codes and standards in Table B3 are referred to in this standard.

The latest valid revision of the DNV reference documents in Tables B1 and B2 applies. These include acceptable methods for fulfilling the requirements in this standard. See
also current DNV List of Publications.

103 Other recognised codes or standards may be applied provided it is shown that they meet or exceed the level of safety of the actual DNV Offshore Standard.

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Table B2 DNV Offshore Object Standards for Structural Design

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C. Definitions

C 100 Verbal forms

101 Shall: Indicates a mandatory requirement to be followed for fulfilment or compliance with the present standard. Deviations are not permitted unless formally and rigorously justified, and accepted by all relevant contracting parties.

102 Should: Indicates a recommendation that a certain course of action is preferred or particularly suitable. Alternative courses of action are allowable under the standard where agreed between contracting parties but shall be justified and documented.

103 May: Indicates a permission, or an option, which is permitted as part of conformance with the standard.

C 200 Terms

201 Accidental Limit States (ALS): Ensures that the structure resists accidental loads and maintain integrity and performance of the structure due to local damage or flooding.

202 Accidental Loads (A): Rare occurrences of extreme environmental loads, fire, flooding, explosions, dropped objects, collisions, unintended pressure differences, leakage of LNG etc.

203 Aggregates are the main ingredient both with respect to volume and weight in a structural concrete mix.

204 Air Gap: Free distance between the 100 year design wave and the underside of a topside structure supported on column supports allowing the wave to pass under the topside structure. When air gap is sufficiently large, then no wave pressure is applied to the topside structure.

205 AS-BUILT Documentation: Documentation of the offshore Structure as finally constructed. Includes design basis/design brief documents, updated designed calculations, updated construction drawings, construction records and approved deviations reports.

206 Atmospheric zone: The external surfaces of the unit above the splash zone.

207 Cathodic protection: A technique to prevent corrosion of a steel surface by making the surface to be the cathode of an electrochemical cell.

208 Cement is the binder component in a structural concrete mix.

209 Characteristic load: The reference value of a load to be used in the determination of load effects. The characteristic load is normally based upon a defined fractile in the upper end of the distribution function for load.

210 Characteristic resistance: The reference value of structural strength to be used in the determination of the design strength. The characteristic resistance is normally based upon a 5% fractile in the lower end of the distribution function for resistance.

211 Characteristic material strength: The nominal value of material strength to be used in the determination of the design resistance. The characteristic material strength is normally based upon a 5% fractile in the lower end of the distribution function for material strength.

212 Characteristic value: The representative value associated with a prescribed probability of not being unfavourably exceeded during some reference period.

213 Classification Note: The Classification Notes cover proven technology and solutions which is found to represent good practice by DNV, and which represent one alternative for satisfying the requirements stipulated in the DNV Rules or other codes and standards cited by DNV. The classification notes will in the same manner be applicable for fulfilling the requirements in the DNV offshore standards.

214 Coating: Metallic, inorganic or organic material applied to steel surfaces for prevention of corrosion.

215 Concrete Grade: A parameter used to define the concrete strength. Concrete Grade for different characteristic val-
ues of concrete strength is provided in Sec.6 Table C1.

216 Corrosion allowance: Extra wall thickness added during design to compensate for any anticipated reduction in thickness during the operation.

217 Cryogenic Temperature: The temperature of the stored LNG.

218 Deck mating: The operation when the deck floated on barges are mated with the concrete support structure.

219 Deformation loads (D): Loads effects on the Terminal caused by thermal effects, prestressing effects, creep/shrinkage effects, differential settlements/deformations etc.

220 Design Hazards: Hazards, which based on risk assessment is likely to occur. The Design Hazards are mitigated into the structural design of the Terminal.

221 Design brief: An agreed document where owners requirements in excess of this standard should be given.

222 Design temperature: The design temperature for a unit is the reference temperature for assessing areas where the unit can be transported, installed and operated. The design temperature is to be lower or equal to the lowest daily mean temperature in air for the relevant areas. For seasonal restricted operations the lowest daily mean temperature in air for the season may be applied. The cargo temperature shall be taken into account in the determination of the cargo temperature.

223 Design value: The value to be used in the deterministic design procedure, i.e. characteristic value modified by the resistance factor or load factor.

224 Driving voltage: The difference between closed circuit anode potential and the protection potential.

225 Ductility: The property of a steel or concrete member to sustain large deformations without failure.

226 Ductility level Earthquake (DLE): The ductility level earthquake is defined probabilistically as an earthquake producing ground motion with a mean recurrence as a minimum of 10,000 years.

227 Environmental Loads (E): Loads from wind, wave, tide, current, snow, ice and earthquake.

228 Expected loads and response history: Expected load and response history for a specified time period, taking into account the number of load cycles and the resulting load levels and response for each cycle.

229 Expected value: The most probable value of a load during a specified time period.

230 Fatigue: Degradation of the material caused by cyclic loading.

231 Fatigue critical: Structure with calculated fatigue life near the design fatigue life.

232 Fatigue Limit States (FLS): Related to the possibility of failure due to the effect of cyclic loading.

233 Functional Loads: Permanent (G) and variable loads (Q), except environmental loads (E), to which the structure can be exposed.

234 Grout is a cementitious material and includes the constituent materials; cement, water and admixture. Appropriate aggregates may be included.

235 Guidance note: Information in the standards in order to increase the understanding of the requirements.

236 Hazards Identification: A list of critical elements, if failed, will have the potential to cause, or contribute substantially to, a major accident. The list is based on consequence of failure only, not on likelihood for failure of the individual hazards.

237 High Strength Concrete: A concrete of Grade in excess of C65.
254 **Offshore Standard:** The DNV offshore standards are documents which presents the principles and technical requirements for design of offshore structures. The standards are offered as DNV’s interpretation of engineering practice for general use by the offshore industry for achieving safe structures.

255 **Offshore installation:** A general term for mobile and fixed structures, including facilities, which are intended for exploration, drilling, production, processing or storage of hydrocarbons or other related activities or fluids. The term includes installations intended for accommodation of personnel engaged in these activities. Offshore installation covers subsea installations and pipelines. The term does not cover traditional shuttle tankers, supply boats and other support vessels which are not directly engaged in the activities described above.

256 **Operating conditions:** Conditions wherein a unit is on location for purposes of production, drilling or other similar operations, and combined environmental and operational loadings are within the appropriate design limits established for such operations (including normal operations, survival, accidental).

257 **Partial Load Factor:** The specified characteristic permanent, variable, deformation, environmental or accidental loads are modified with a load factor. This load factor is part of the safety approach and varies in magnitude for the different load categories dependent on the individual uncertainties in the characteristic loads.

258 **Permanent Functional Loads (G):** Self-weight, ballast weight, weight of permanent installed part of mechanical outfitting, external hydrostatic pressure, prestressing force etc.

259 **Potential:** The voltage between a submerged metal surface and a reference electrode.

260 **Prestressing systems:** Tendons (wires, strands, bars), anchorage devices, couplers and ducts or sheaths are part of a prestressing system.

261 **Quality Plan:** A plan implemented to ensure quality in the design, construction and in-service inspection/maintenance. An interface manual shall be developed defining all interfaces between the various parties and disciplines involved to ensure that the responsibilities, reporting routines and information routines are established.

262 **Recommended Practice (RP):** The recommended practice publications cover proven technology and solutions which have been found by DNV to represent good practice, and which represent one alternative for satisfy the requirements stipulated in the DNV offshore standards or other codes and standards cited by DNV.

263 **Reinforcement** is defined as the constituents of structural concrete providing the tensile strength that will give the concrete its ductile characteristics. In these Rules reinforcement is categorised as:

- ordinary reinforcement
- prestressing reinforcement
- special reinforcement.

264 **Robustness:** A robust structure is a structure with low sensitivity to local changes in geometry and loads.

265 **Redundancy:** The ability of a component or system to maintain or restore its function when a failure of a member or connection has occurred. Redundancy may be achieved for instance by strengthening or introducing alternative load paths

266 **Reference electrode:** Electrode with stable open-circuit potential used as reference for potential measurements.

267 **Reliability:** The ability of a component or a system to perform its required function without failure during a specified time interval.

268 **Repair Materials:** Material used to structurally repair the Offshore Concrete Structure.

269 **Risk:** The qualitative or quantitative likelihood of an accidental or unplanned event occurring considered in conjunction with the potential consequences of such a failure. In quantitative terms, risk is the quantified probability of a defined failure mode times its quantified consequence.

270 **Service temperature:** Service temperature is a reference temperature on various structural parts of the unit used as a criterion for the selection of steel grades or design for crackwidth etc. in SLS.

271 **Serviceability Limit States (SLS):** Corresponding to the criteria applicable to normal use or durability.

272 **Sheaths:** Ducts for post-tensioning tendons. Sheaths shall in general be of a semi rigid or rigid type, water tight and with adequate stiffness to prevent damages and deformations.

273 **Slamming:** Impact load on an approximately horizontal member from a rising water surface as a wave passes. The direction of the impact load is mainly vertical. Slamming can also occur within tanks due to stored liquids.

274 **Specified Minimum Yield Strength (SMYS):** The minimum yield strength prescribed by the specification or standard under which the material is purchased.

275 **Specially aggressive environment (SA):** Structures exposed to strong chemical attack which will require additional protective measures. This may require specially mixed concrete, membranes or similar.

276 **Specified value:** Minimum or maximum value during the period considered. This value may take into account operational requirements, limitations and measures taken such that the required safety level is obtained.

277 **Severely aggressive environment (MA):** Structures in saline water, in the splash zone or exposed to sea spray, structures exposed to aggressive gases, salt or other chemical substances, and structures exposed to repeated freezing and thawing in a wet condition

278 **Splash zone:** The external surfaces of the unit that are periodically in and out of the water. The determination of the splash zone includes evaluation of all relevant effects including influence of waves, tidal variations, settlements, subsidence and vertical motions.

279 **Stability:** The ability of the floating structure to remain upright and floating when exposed to small changes in applied loads.

The ability of a structural member to carry small additional loads without buckling.

280 **Strength level Earthquake (SLE):** The strength level earthquake is defined probabilistically as an earthquake producing ground motion with a mean recurrence at a minimum interval of 100 years.

281 **Structural concrete** is defined as a cementitious composite material and is the main ingredient for construction of concrete structures.

282 **Submerged zone:** The part of the unit which is below the splash zone, including buried parts.

283 **Survival condition:** A condition during which a unit may be subjected to the most severe environmental loadings for which the unit is designed. Drilling or similar operations may have been discontinued due to the severity of the environmental loadings. The unit may be either afloat or supported on the sea bed, as applicable.

284 **Target safety level:** A nominal acceptable probability of structural failure.

285 **Temporary conditions:** Design conditions not covered by operating conditions, e.g. conditions during fabrication,
mating and installation phases, transit phases, accidental

**286 Temporary Phase:** Reference is made fabrication, mating, transit/towing and installation phases.

**287 Test report:** A document made by the Manufacturer which contains the results of control tests on current production, carried out on products having the same method of manufacture as the consignment, but not necessarily from the delivered products themselves.

**288 Tensile strength:** Minimum stress level where strain hardening is at maximum or at rupture for steel. For concrete it is the direct tensile strength of concrete.

**289 Transit conditions:** All unit movements from one geographical location to another.

**290 Ultimate Limit States (ULS):** Corresponding to the maximum load carrying resistance.

**291 Unit:** is a general term for an offshore structure.

**292 Utilisation factor:** The fraction of anode material that can be utilised for design purposes. For design of Terminal structures, the utilisation factor also means the ratio of used strength to failure strength of concrete, reinforcement or pre-stressing steel.

**293 Variable Functional Loads (Q):** Weight and loads caused by the normal operation of the Offshore Structure. Variable Functional Loads may vary in position, magnitude and direction during the operational period and includes modules, gas weight, stored goods, pressure of stored components, pressures from stored LNG, temperature of LNG, loads occurring during installation, operational boat impacts, mooring loads etc.

**294 Verification:** Examination to confirm that an activity, a product or a service is in accordance with specified requirements.

**295 Works’ certificate:** A document made by the Manufacturer which contains the results of all the required tests and which certifies that the tests have been carried out by the Manufacturer on samples taken from the delivered products themselves.

### D. Abbreviations and Symbols

#### D 100 Abbreviations

**101** Abbreviations as shown in Table D1 are used in this standard.

<table>
<thead>
<tr>
<th>Table D1 Abbreviations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Abbreviation</strong></td>
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<tr>
<td>A</td>
</tr>
<tr>
<td>ACI</td>
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<td>AISC</td>
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<td>DLE</td>
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<td>DNV</td>
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<td>E</td>
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<tr>
<td>EN</td>
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<tr>
<td>ETM</td>
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<tr>
<td>ESD</td>
</tr>
<tr>
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<td>FTM</td>
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<tr>
<td>G</td>
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<tr>
<td>HAT</td>
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<td>HAZOP</td>
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<td>HISC</td>
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<td>IG</td>
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<td>IMO</td>
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<tr>
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<td>LA</td>
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<tr>
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<td>LNG</td>
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<tr>
<td>SLE</td>
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<tr>
<td>SMYS</td>
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<tr>
<td>UN</td>
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<tr>
<td>SN-curves</td>
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</table>

#### D 200 Symbols

**201 Latin characters**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Accidental loads</td>
</tr>
<tr>
<td>A1</td>
<td>loaded area</td>
</tr>
<tr>
<td>A2</td>
<td>assumed distribution area</td>
</tr>
<tr>
<td>Ac</td>
<td>concrete area of a longitudinal section of the beam web</td>
</tr>
<tr>
<td>Acf</td>
<td>cross-sectional area of uncracked concrete</td>
</tr>
<tr>
<td>Acf</td>
<td>effective cross section area of the flange, h, beff</td>
</tr>
<tr>
<td>As</td>
<td>cross section area of properly anchored reinforcement on the tension side (mm²)</td>
</tr>
<tr>
<td>As</td>
<td>the reinforcement area that is sufficiently anchored on both sides of the joint and that is not utilized for other purposes</td>
</tr>
<tr>
<td>A5V</td>
<td>Amount of shear reinforcement</td>
</tr>
<tr>
<td>A5x</td>
<td>amount of reinforcement in x-direction</td>
</tr>
<tr>
<td>A5y</td>
<td>amount of reinforcement in y-direction</td>
</tr>
<tr>
<td>A6t</td>
<td>the area of transverse reinforcement not utilized for other tensile forces and having a spacing not greater than 12 times the diameter of the anchored reinforcement. If the reinforcement is partly utilized, the area shall be proportionally reduced</td>
</tr>
<tr>
<td>a</td>
<td>distance from the face of the support</td>
</tr>
<tr>
<td>a</td>
<td>vertical acceleration</td>
</tr>
<tr>
<td>σeff</td>
<td>the part of the slab width which according to Sec.6 A400 is assumed as effective when resisting tensile forces</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>---------</td>
<td>----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>b</td>
<td>length of the side of the critical section (Sec.6 F510)</td>
</tr>
<tr>
<td>b_1</td>
<td>length of the side perpendicular to b</td>
</tr>
<tr>
<td>b_w</td>
<td>width of beam (web) (mm)</td>
</tr>
<tr>
<td>C</td>
<td>concrete grade (normal weight concrete)</td>
</tr>
<tr>
<td>C_f</td>
<td>factor on Wohler curves concrete (Sec.6 M200)</td>
</tr>
<tr>
<td>C_r</td>
<td>factor on Wohler curves concrete (Sec.6 M200)</td>
</tr>
<tr>
<td>C_k</td>
<td>factor on Wohler curve reinforcement (Sec.6 M200)</td>
</tr>
<tr>
<td>c</td>
<td>the least of the dimensions c_1, c_2 and (s_1 - θ)/2 given in Fig.13</td>
</tr>
<tr>
<td>c_1</td>
<td>minimum concrete cover, see Sec.6 Table Q1</td>
</tr>
<tr>
<td>c_2</td>
<td>actual nominal concrete cover</td>
</tr>
<tr>
<td>d</td>
<td>deformation load</td>
</tr>
<tr>
<td>D_1</td>
<td>diameter of the concrete core inside the centroid of the spiral reinforcement, A_{ss}</td>
</tr>
<tr>
<td>d</td>
<td>distance from the centroid of the tensile reinforcement to outer edge of the compression zone</td>
</tr>
<tr>
<td>d_1</td>
<td>1 000 mm</td>
</tr>
<tr>
<td>e</td>
<td>eccentricity of loading</td>
</tr>
<tr>
<td>E</td>
<td>environmental load</td>
</tr>
<tr>
<td>E_{cd}</td>
<td>design value of Young’s Modulus of concrete used in the stress-strain curve</td>
</tr>
<tr>
<td>E_{cm}</td>
<td>normalized value of Young’s Modulus used in the stress-strain curve</td>
</tr>
<tr>
<td>E_{cd}</td>
<td>design value of Young’s Modulus of reinforcement</td>
</tr>
<tr>
<td>E_{ck}</td>
<td>characteristic value of Young’s Modulus of reinforcement (200 000 MPa)</td>
</tr>
<tr>
<td>f_{cc}</td>
<td>concrete related portion of the design bond strength in accordance with Sec.6 K16</td>
</tr>
<tr>
<td>f_{bd}</td>
<td>design bond strength, calculated in accordance with Sec.6 K16</td>
</tr>
<tr>
<td>f</td>
<td>concrete cylinder strength</td>
</tr>
<tr>
<td>f_{ck2}</td>
<td>characteristic concrete compressive strength</td>
</tr>
<tr>
<td>f_{ck1}</td>
<td>characteristic strength of the taken specimens converted into cylinder strength for cylinders with height/diameter ratio 2:1</td>
</tr>
<tr>
<td>f_{ck1}</td>
<td>characteristic compressive strength cylinder at 28 days based on in-situ tests</td>
</tr>
<tr>
<td>f_{cd}</td>
<td>design compressive strength of concrete</td>
</tr>
<tr>
<td>f_{cd}</td>
<td>truss analogy: design compressive strength (Sec.6 F308)</td>
</tr>
<tr>
<td>f_{cd}</td>
<td>general: reduced design compressive strength (Sec.6 H107) = f_{cd}(0.8 + 100 φ) &lt; f_{cd}</td>
</tr>
<tr>
<td>f</td>
<td>reference stress for the type of failure in question (Sec.6 M200)</td>
</tr>
<tr>
<td>f_{cd}</td>
<td>design strength of concrete in uni-axial tension</td>
</tr>
<tr>
<td>f</td>
<td>tensile strength of concrete</td>
</tr>
<tr>
<td>f_k</td>
<td>characteristic tensile strength of concrete</td>
</tr>
<tr>
<td>f_k</td>
<td>for structures exposed to pressure from liquid or gas in the formula for calculating the required amount of minimum reinforcement (Sec.6 Q503)</td>
</tr>
<tr>
<td>f_{bd}</td>
<td>design strength of reinforcement</td>
</tr>
<tr>
<td>f_{bd}</td>
<td>design strength of the spiral reinforcement, A_{ss}</td>
</tr>
<tr>
<td>f_{ks}</td>
<td>characteristic strength of reinforcement</td>
</tr>
<tr>
<td>F</td>
<td>compressive capacity</td>
</tr>
<tr>
<td>F_d</td>
<td>design load</td>
</tr>
<tr>
<td>F_1</td>
<td>force in accordance with Fig.15</td>
</tr>
<tr>
<td>F_p</td>
<td>characteristic load</td>
</tr>
<tr>
<td>ΣF_{vn}/s</td>
<td>sum of forces F_{vn} corresponding to shear failure at cross wire welds within the development length</td>
</tr>
<tr>
<td>F_{SV}</td>
<td>additional tensile force in longitudinal reinforcement due to shear</td>
</tr>
<tr>
<td>F_s</td>
<td>N_x +</td>
</tr>
<tr>
<td>F_s</td>
<td>N_x +</td>
</tr>
<tr>
<td>F</td>
<td>permanent load</td>
</tr>
<tr>
<td>g</td>
<td>acceleration due to gravity</td>
</tr>
<tr>
<td>h</td>
<td>cross-section height</td>
</tr>
<tr>
<td>h'</td>
<td>distance between the centroid of the reinforcement on the “tensile” and “compression” side of the member</td>
</tr>
<tr>
<td>h_1</td>
<td>1.0 m (Sec.6 D107)</td>
</tr>
<tr>
<td>h_2</td>
<td>thickness of the flange (the slab)</td>
</tr>
<tr>
<td>I_k</td>
<td>moment of inertia of A_y</td>
</tr>
<tr>
<td>k</td>
<td>number of stress-blocks (Sec.6 M107)</td>
</tr>
<tr>
<td>k_1</td>
<td>a factor depending of the type of reinforcement, given in Sec.6 Table K2</td>
</tr>
<tr>
<td>k_2</td>
<td>has the value 1.6 if the spacing s between the anchored bars exceeds 9φ or (3c + φ) whichever is the larger, k_2 has the value 1.0 if s is less than the larger of 5φ and (3c + φ). For intermediate values interpolate linearly (Sec.6 K116)</td>
</tr>
<tr>
<td>k_3</td>
<td>a factor dependent on the transverse reinforcement and its position as given in Fig.14. The factor k_3 is taken as zero for strands</td>
</tr>
<tr>
<td>k_A</td>
<td>100 MPa</td>
</tr>
<tr>
<td>k_t</td>
<td>factor used for prediction of Young’s modulus.</td>
</tr>
<tr>
<td>k</td>
<td>a factor depending on the number of bars in the bundle and is taken as:</td>
</tr>
<tr>
<td></td>
<td>— 0.8 for bundle of 2 bars</td>
</tr>
<tr>
<td></td>
<td>— 0.7 for bundle of 3 bars</td>
</tr>
<tr>
<td></td>
<td>— 0.6 for bundle of 4 bars</td>
</tr>
<tr>
<td>k_s</td>
<td>for slabs and beams without shear reinforcement the factor k_s is set equal to 1.5 – d/d_1, but not greater than 1.4 nor less than 1.0</td>
</tr>
<tr>
<td>k_w</td>
<td>coefficient dependent on cross-sectional height h=1.5 – h/h_1 &gt; 1.0, where h_1 = 1.0 m (Sec.6 O700)</td>
</tr>
<tr>
<td>l_1</td>
<td>distance between zero moment points</td>
</tr>
<tr>
<td>l_2</td>
<td>development length bond – bars or bundle of bars</td>
</tr>
<tr>
<td>l_h</td>
<td>development length for welded wire fabric</td>
</tr>
<tr>
<td>l_p</td>
<td>development length for the prestressing force</td>
</tr>
<tr>
<td>l_{ek}</td>
<td>effective length, theoretical buckling length</td>
</tr>
<tr>
<td>l_{sk}</td>
<td>the influence length of the crack, some slippage in the bond between reinforcement and concrete may occur (Sec.6 O700)</td>
</tr>
<tr>
<td>M</td>
<td>moment</td>
</tr>
<tr>
<td>M_f</td>
<td>total moment in the section acting in combination with the shear force V_f</td>
</tr>
<tr>
<td>M_a</td>
<td>-N_f Wc/A_c</td>
</tr>
<tr>
<td>M_{OA}</td>
<td>numerical smallest member end moment calculated from 1. order theory at end A</td>
</tr>
<tr>
<td>M_{OB}</td>
<td>numerical largest member end moment calculated from 1. order theory at end B</td>
</tr>
<tr>
<td>m</td>
<td>numerical factor</td>
</tr>
<tr>
<td>n</td>
<td>number</td>
</tr>
<tr>
<td>n_l</td>
<td>number of cycles in stress-block I (Sec.6 M107)</td>
</tr>
<tr>
<td>n_r</td>
<td>N_r/E_{cd} A_c</td>
</tr>
<tr>
<td>N</td>
<td>design life of concrete subjected to cyclic stresses</td>
</tr>
<tr>
<td>N_x</td>
<td>design axial force (positive as tension)</td>
</tr>
<tr>
<td>N_t</td>
<td>number of cycles with constant amplitude which causes fatigue failure (Sec.6 M107)</td>
</tr>
<tr>
<td>N_y</td>
<td>axial force in x-direction</td>
</tr>
<tr>
<td>N_y</td>
<td>axial force in y-direction</td>
</tr>
<tr>
<td>N_w</td>
<td>shear force in the x-y plane</td>
</tr>
<tr>
<td>P</td>
<td>load</td>
</tr>
<tr>
<td>P_i</td>
<td>design pressure</td>
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<tr>
<td>Q</td>
<td>variable functional load</td>
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### Greek characters

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
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<tbody>
<tr>
<td>α</td>
<td>angle between transverse shear reinforcement and the longitudinal axis</td>
</tr>
<tr>
<td>α</td>
<td>the angle between the reinforcement and the contact surface, where only reinforcement with an angle between 90° and 45° (to the direction of the force) shall be taken into account</td>
</tr>
<tr>
<td>α</td>
<td>a factor given in Sec.6 Table K1</td>
</tr>
<tr>
<td>α</td>
<td>1.3 – 0.3 β &gt; 1.0 (Sec.6 M302)</td>
</tr>
<tr>
<td>β</td>
<td>ratio between the numerically largest and smallest stresses acting simultaneously in the local compressive concrete zone. The distance between the points used when calculating β shall not exceed 300 mm (0 &lt; β &lt; 1.0) (Sec.6 M302)</td>
</tr>
<tr>
<td>β</td>
<td>a factor given in Sec.6 Table K1</td>
</tr>
<tr>
<td>β</td>
<td>opening angle of the bend (Sec.6 L111)</td>
</tr>
<tr>
<td>Δσ</td>
<td>stress variation of the reinforcement (MPa) (Sec.6 M202)</td>
</tr>
<tr>
<td>ε</td>
<td>strain</td>
</tr>
<tr>
<td>ε₁</td>
<td>- 1.9 % (Sec.6 C301)</td>
</tr>
<tr>
<td>ε₁</td>
<td>average principal tensile strain (Sec.6 H107)</td>
</tr>
<tr>
<td>ε₂₀</td>
<td>ε₁ - k₂ε₂₀ (Sec.6 C301), - 2 % (Sec.6 C302)</td>
</tr>
<tr>
<td>εₜₘᵡ</td>
<td>max strain, NW concrete (2.5 m – 1.5)εₑₑ (Sec.6 C301)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>max strain, LWA concrete (Sec.6 C303)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>- εₑₑ / Eₑₑ (Sec.6 O700)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>mean stress dependent tensile strain in the concrete at the same layer and over the same length as εₑₑ (Sec.6 O700)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>free shrinkage strain of the concrete (negative value) (Sec.6 O700)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>tensile strain in reinforcement slightly sensitive to corrosion on the side with highest strain (Sec.6 O206)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>tensile strain at the level of the reinforcement sensitive to corrosion (Sec.6 O206)</td>
</tr>
<tr>
<td>εₑₑ</td>
<td>mean principal tensile strain in the reinforcement in the crack’s influence length at the outer layer of the reinforcement (Sec.6 O700)</td>
</tr>
<tr>
<td>η</td>
<td>ratio of fatigue utilization</td>
</tr>
<tr>
<td>γₑₑ</td>
<td>material coefficient concrete</td>
</tr>
<tr>
<td>γₙₙ</td>
<td>load factor</td>
</tr>
<tr>
<td>γₙₙ</td>
<td>material factor (material coefficient)</td>
</tr>
<tr>
<td>β</td>
<td>material coefficient reinforcement</td>
</tr>
<tr>
<td>λ</td>
<td>geometric slenderness ratio = 80 (1+4 ωₕ)⁰.⁵</td>
</tr>
<tr>
<td>λ</td>
<td>λ / i = (Iₚ/Aₚ)⁰.⁵</td>
</tr>
<tr>
<td>λₑₑ</td>
<td>force dependent slenderness = λ ( - nₑₑ / (1+4 ωₕ))⁰.⁵</td>
</tr>
<tr>
<td>θ</td>
<td>angle between the inclined concrete compression struts and the longitudinal axis in the truss model method</td>
</tr>
<tr>
<td>φ</td>
<td>diameter of the reinforcement bar</td>
</tr>
<tr>
<td>φₑₑ</td>
<td>equivalent diameter in term of reinforcement cross section</td>
</tr>
<tr>
<td>μ</td>
<td>friction coefficient</td>
</tr>
<tr>
<td>ρ</td>
<td>density</td>
</tr>
<tr>
<td>ρₙₙ</td>
<td>2 400 (Sec.5 D306), 2 200 (Sec.6 C102)</td>
</tr>
<tr>
<td>ρₑₑ</td>
<td>reinforcement ratio in x – direction = Aₑₑ / (b·d)</td>
</tr>
<tr>
<td>ρₑₑ</td>
<td>reinforcement ratio in y – direction = Aₑₑ / (b·d)</td>
</tr>
<tr>
<td>ρₑₑ</td>
<td>creep coefficient</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>concrete stress due to long-term loading</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>design stress</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>edge stress due to bending alone (tension positive) (Sec.6 O700)</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>numerically largest compressive stress, calculated as the average value within each stress-block</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>numerically least compressive stress, calculated as the average value within each stress-block</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>stress due to axial force (tension positive) (Sec.6 O700)</td>
</tr>
<tr>
<td>σₑₑ</td>
<td>the steel stress due to prestressing</td>
</tr>
<tr>
<td>τₑₑ</td>
<td>bond strength in accordance with Sec.6 Table J1</td>
</tr>
<tr>
<td>τₑₑ</td>
<td>maximum bond stress within fatigue stress block</td>
</tr>
<tr>
<td>τₑₑ</td>
<td>minimum bond stress within fatigue stress block</td>
</tr>
</tbody>
</table>

### Subscripts

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>d</td>
<td>design value</td>
</tr>
<tr>
<td>k</td>
<td>characteristic value</td>
</tr>
<tr>
<td>p</td>
<td>plastic</td>
</tr>
<tr>
<td>y</td>
<td>yield</td>
</tr>
</tbody>
</table>
SECTION 2
SAFETY PHILOSOPHY

A. General

A 100 Objective

101 The purpose of this section is to present the safety philosophy and corresponding design format applied in this standard.

102 This section applies to Offshore Concrete Structures which shall be built in accordance with this standard.

103 This section also provides guidance for extension of this standard in terms of new criteria etc.

104 The integrity of an Offshore Concrete Structures designed and constructed in accordance with this standard is ensured through a safety philosophy integrating different parts as illustrated in Figure 1.

105 An overall safety objective shall be established, planned and implemented, covering all phases from conceptual development until abandonment.

A 200 Systematic review

201 As far as practical, all work associated with the design, construction and operation of the Offshore Concrete Structure shall be such as to ensure that no single failure will lead to life-threatening situations for any person, or to unacceptable damage to the Structure or the environment.

202 A systematic review or analysis shall be carried out for all phases in order to identify and evaluate the consequences of single failures and series of failures in the Offshore Concrete Structure, such that necessary remedial measures can be taken. The extent of the review or analysis shall reflect the criticality of the Offshore Concrete Structure, the criticality of a planned operation, and previous experience with similar systems or operations.

Guidance note:
A methodology for such a systematic review is quantitative risk analysis (QRA). This may provide an estimation of the overall risk to human health and safety, environment and assets and comprises:
- hazard identification,
- assessment of probabilities of failure events,
- accident developments, and
- consequence and risk assessment.

It should be noted that legislation in some countries requires risk analysis to be performed, at least at an overall level to identify critical scenarios that might jeopardise the safety and reliability of the Structure. Other methodologies for identification of potential hazards are Failure Mode and Effect Analysis (FMEA) and Hazard and Operability studies (HAZOP).

A 300 Safety class methodology

301 The Offshore Concrete Structure is classified into the safety class 3 based on failure consequences. For definition see Table A1.

<table>
<thead>
<tr>
<th>Class for consequences of failure</th>
<th>Safety Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor seriousness</td>
<td>1</td>
</tr>
<tr>
<td>Serious</td>
<td>2</td>
</tr>
<tr>
<td>Very Serious</td>
<td>3</td>
</tr>
</tbody>
</table>

A 400 Quality assurance

401 The safety format within this standard requires that gross errors (human errors) shall be controlled by requirements for organisation of the work, competence of persons performing the work, verification of the design, and quality assurance during all relevant phases.

402 For the purpose of this standard, it is assumed that the owner of the Offshore Concrete Structure has established a quality objective. The owner shall, in both internal and external quality related aspects, seek to achieve the quality level of products and services intended in the quality objective. Further, the owner shall provide assurance that intended quality is being, or will be, achieved.

403 The quality system shall comply with the requirements of ISO 9000 and specific requirements quoted for the various engineering disciplines in this Standard.

404 All work performed in accordance with this standard shall be subject to quality control in accordance with an implemented Quality Plan. The Quality Plan should be in accordance with the ISO 9000 series. There may be one Quality Plan covering all activities, or one overall plan with separate plans for the various phases and activities to be performed.

405 The Quality Plan shall ensure that all responsibilities are defined. An Interface Manual should be developed that defines all interfaces between the various parties and disciplines involved, and ensure that responsibilities, reporting and information routines as appropriate are established.

A 500 Health, safety and environment

501 The objective of this standard is that the design, materials, fabrication, installation, commissioning, operation, repair, re-qualification, and abandonment of the Offshore Concrete Structure are safe and conducted with due regard to public safety and the protection of the environment.

A 600 Qualifications of personnel

601 All activities that are performed in the design, construction, transportation, inspection and maintenance of offshore structures according to this Standard shall be performed by qualified personnel with the qualifications and experience necessary to meet the objectives of this Standard. Qualifications and relevant experience shall be documented for all key personnel and personnel performing tasks that normally require special training or certificates.

602 National provisions on qualifications of personnel such as engineers, operators, welders, divers, etc. in the place of use...
apply. Additional requirements may be given in the project specification.

B. Design Format

B 100 General

101 The design format within this standard is based upon a limit state and partial safety factor methodology, also called Load and Resistance Factor Design format (LRFD).

The design principles are specified in Sec.2 of DNV-OS-C101. The design principle is based on LRFD, but design may additionally be carried out by both testing and probability based design.

The aim of the design of the Offshore Concrete Structure and its elements are to:

— sustain loads liable to occur during all temporary operating and damaged conditions if required
— maintain acceptable safety for personnel and environment
— have adequate durability against deterioration during the design life of the Offshore Concrete Structure.

102 The design of a structural system, its components and details shall, as far as possible, account for the following principles:

— resistance against relevant mechanical, physical and chemical deterioration is achieved
— fabrication and construction comply with relevant, recognised techniques and practice
— inspection, maintenance and repair are possible.

103 Structures and elements thereof, shall possess ductile resistance unless the specified purpose requires otherwise.

104 Requirements to materials are given in Sec.4, Loads and Analyses Requirements in Sec.5, Detailed Design of Offshore Concrete Structures in Sec.6, Construction in Sec.7 and In-service Inspection, Maintenance and Conditioned Monitoring in Sec.8.

105 Additionally, in Appendices A to F, guidelines are given for:

— environmental loading (A)
— structural analysis – modelling (B)
— structural analyses (C)
— seismic analyses (D)
— use of alternative design standard (E)
— crackwidth calculation (F).

There should be a clear and documented link between major accident hazards and the critical elements.

103 The following inputs are normally required in order to develop the list of critical elements:

— description of Structure and mode(s) of operation, including details of the asset manning
— equipment list and layout
— hazard identification report and associated studies
— safety case where applicable.

104 The basic criteria in establishing the list of critical elements is to determine whether the system, component or equipment which – should they fail – have the potential to cause, or contribute substantially to, a major accident. This assessment is normally based upon consequence of failure only, not on the likelihood of failure.

105 The following methodology should be applied for confirming that prevention, detection, control or mitigation measures have been correctly identified as critical elements:

— identify the major contributors to overall risk,
— identify the means to reduce risk,
— link the measures, the contributors to risk and the means to reduce risk to the assets’ systems – these can be seen to equate to the critical elements of the asset.

106 The record of critical elements typically provides only a list of systems and types of equipment or structure etc. In order to complete a meaningful list, the scope of each element should be clearly specified such that there can be no reasonable doubt as to the precise content of each element.

107 The above processes should consider all phases of the lifecycle of the Structure.

108 The hazard assessment shall consider, as a minimum the following events:

— damage to the primary structure due to:
  — extreme weather
  — ship collision
  — dropped objects
  — helicopter collision
  — exposure to unsuitable cold/warm temperature
  — exposure to high radiation heat.

— fire and explosion
— loss of Primary Liquid Containment (duration shall be determined based on an approved contingency plan)
— oil/gas leakage
— release of flammable or toxic gas to the atmosphere or inside an enclosed space
— loss of stability
— loss of any single component in the Station Keeping/Mooring system
— loss of ability to offload oil/gas
— loss of any critical component in the process system
— loss of electrical power.

109 The results of the Hazard Identification and Risk Assessment shall become an integral part of the structural design of the Offshore Concrete Structure.

C. Identification of Major Accidental Hazards

C 100 General

101 The Standard identified common accidental hazards for an Offshore Concrete Structure. The designer shall ensure itself of its completeness by documenting through a hazard identification and risk assessment process that all hazards which may be critical to the safe operation of the Offshore Concrete Structure have been adequately accounted for in design. This process shall be documented.

102 Criteria for the identification of major accident hazards shall be:

— significant damage to the asset
— significant damage to the environment.

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— loss of electrical power.

109 The results of the Hazard Identification and Risk Assessment shall become an integral part of the structural design of the Offshore Concrete Structure.
SECTION 3
DESIGN DOCUMENTATION

A. Overall Planning

A 100 General

101 A fixed/floating concrete offshore Structure shall be planned in such a manner that it can meet all requirements related to its functions and use as well as its structural safety and durability requirements. Adequate planning shall be done before actual design is started in order to have sufficient basis for the engineering and by that obtain a safe, workable and economical structure that will fulfill the required functions.

102 The initial planning shall include determination and description of all the functions the structure shall fulfill, and all the criteria upon which the design of the structure are based. Site-specific data such as water depth, environmental conditions and soil properties shall be sufficiently known and documented to serve as basis for the design. All functional and operational requirements in temporary and service phases as well as robustness against accidental conditions that can influence the layout and the structural design shall be considered.

103 All functional requirements to the Structure affecting the layout and the structural design, shall be established in a clear format such that it can form the basis for the engineering process and the structural design.

104 Investigation of site-specific data such as seabed topography, soil conditions and environmental conditions shall be carried out in accordance with requirements of DNV-OS-C101, ISO 19901-1 and ISO 19901-4.

A 200 Description of Offshore Concrete Structure

201 The objective is to provide an overview of the offshore Structure, highlighting key assumptions and operational phases of the development.

202 The overview should be presented in three sections:
   a) Overview of facility
   b) Development bases and phases
   c) Staffing philosophy and arrangements.

Cross-references to data sources, figures etc. should be provided.

A 300 Meteorological and ocean conditions

301 The objective is to summarise key design parameters with cross-references to key technical documents.

302 The metocean/climatology conditions section should cover at least the following:
   — storm/wave/current conditions
   — wind
   — seawater/air temperature
   — earthquakes
   — cyclones
   — other extreme conditions
   — seabed stability
   — tsunami
   — atmospheric stability
   — range and rates of changes of barometer pressure
   — rainfall, snow
   — corrosive characteristics of the air
   — frequency of lightning strikes
   — relative humidity.

303 Seismology for Gravity Based Structure in Seismic Active Zones:

An earthquake is defined by the horizontal and vertical accelerations of the ground. These accelerations are described by their:
   — frequency spectrum
   — amplitude.

Guidance note:
A site specific earthquake analysis shall be performed. This analysis shall be reported in a Seismic Report where geological and seismic characteristics of the location of the gravity based facilities and the surrounding region as well as geotectonic information from the location have to be taken into account. As a conclusion this report shall recommend all seismic parameters required for the design.

The potential of earthquake activity in the vicinity of the proposed site is determined by investigating the seismic history of the region (320 km radius) surrounding the site, and relating it to the geological and tectonic conditions resulting from the soil survey.

These investigations involve thorough research, review and evaluation of all historically reported earthquakes that have affected, or that could reasonably be expected to have affected the site.

The geological, tectonic and seismological studies help to establish:
   - strength level earthquake (SLE)
   - ductility level earthquake (DLE).

SLE and DLE shall be established as either:
   — probabilistically, as those that produce ground motions with the mean recurrence as a minimum interval of 10 000 years for the DLE and 100 years for the SLE, and/or
   — deterministically, assuming that earthquakes which are analogous to maximum historically known earthquakes are liable to occur in future with an epicentre position which is the most severe with regard to its effects in terms of intensity on the site, while remaining compatible with geological and seismic data. In this case, the SLE accelerations shall be one-half those determined for the DLE.

A 400 Layout of the offshore concrete structure

401 The objective is to provide a description of the Offshore Structure, its unique features (if any), equipment layout for all decks, and interaction with existing offshore/onshore facilities.

402 This section should include a description of at least the following (where applicable):
   — General:
      — structure/platform
      — geographical location
      — water depth.
   — Layout:
      — orientation of the structure
      — elevation/plan views
      — equipment
      — escape routes
      — access to sea deck
      — emergency assembly area etc.
      — structural details, including modelling of structure and loadings.
   — Interaction with existing facilities:
— physical connections
— support from existing facilities.
— Interaction with expected facilities (where applicable).

A 500 Primary functions

501 To provide a description of the functions of the Facility by describing key processes:
— oil storage system
— pipeline systems
— marine and helicopter operations.

This information is required as background information essential for identification of structural hazards of importance for the design of the structural load bearing structure of the terminal.

502 The primary functions section should include a description of at least the following (where applicable):

Process systems:
— process description (overview)
— process control features
— safety control systems for use during emergencies e.g. controls at the TR or emergency assembly area.

Oil storage system
— oil storage tank
— piping
— layout
— electrical
— monitoring.

Pipeline and riser systems:
— location, separation, protection
— riser connect/disconnect system.

Utility systems:
— power generation and distribution
— communications
— other utility systems (e.g. instrument air, hydraulics, cranes).

Inert gas systems
— safety features (e.g. blow-out prevention systems)
— integration with platform systems.

Workover and wireline systems:
— extent and type of activity planned
— integration with platform systems.

Marine functions/systems:
— supply
— standby vessels
— diving
— ballast and stability systems
— mooring systems
— oil/gas offloading system
— oil/gas vessel mooring system.

Helicopter operations:
— onshore base
— capability of aircraft
— helicopter approach.

A 600 Standards

601 A design brief document shall include references to Standards and design specifications.

A 700 Documentation

701 Documentation shall be prepared for all activities that shall be performed in the design, construction, transportation and installation of offshore concrete structure. Documentation shall also be prepared showing records of all inspection and control of materials used and execution work performed that has an impact on the quality of the final product. The documentation shall be to a standard suitable for independent verification.

702 Necessary procedures and manuals shall be prepared to ensure that the construction, transportation, installation and in-service inspection are performed in a controlled manner in full compliance with all assumptions of the design.

703 The most important assumptions, on which the design, construction and installation work is based with regard to the Offshore Concrete structure, shall be presented in a Summary Report. The Summary Report shall be available and suitable for use in connection with operation, maintenance, alterations and possible repair work. The summary report will normally be based on the documentation identified in A800 and A900.

A 800 Documentation required prior to construction

801 The technical documentation of concrete structure, available prior to construction, shall comprise:
— design calculations for the complete structure including individual members
— project specification and procedures
— drawings issued for construction and approved by design manager.

802 All technical documentation shall be dated, signed and verified.

803 The Project Specification shall comprise:
— construction drawings, giving all necessary information such as geometry of the structure, amount and position of reinforcing and prestressing steel and for precast concrete elements, tolerances, lifting devices, weights, inserts, etc.
— description of all products to be used with any requirements to the application of the materials. This information should be given on the drawings and/or in the work description. Material specifications, product standards, etc., shall be included
— work description (procedures) related to the construction activity.

804 The work description should also include all requirements to execution of the work, i.e. sequence of operation, installation instructions for embedment plates, temporary supports, work procedures, etc.

805 The work description shall include an erection specification for precast concrete elements comprising:
— installation drawings consisting of plans and sections showing the positions and the connections of the elements in the completed work
— installation data with the required materials properties for materials applied at site
— installation instructions with necessary data for the handling, storing, setting, adjusting, connection and completion works with required geometrical tolerances
— quality control procedures.

A 900 AS-BUILT documentation

901 The As-Built documentation shall comprise:
— quality records
— method statements
— sources of materials, material test certificates and/or suppliers’ attestation of conformity, mill certificate, approval documents
— applications for concessions and responses
— as-built drawings or sufficient information to allow for preparation of as-built drawings for the entire structure including any precast elements
— a description of non-conformities and the results of possible corrective actions
— a description of accepted changes to the project specification
— records of possible dimensional checks at handover
— a diary or log where the events of the construction process are reported
— documentation of the inspection performed
— Geotechnical Design Report (GDR).

A 1000 Inspection/monitoring plans for structure in service

1001 Plans for monitoring and inspection of the installation shall be prepared.
SECTION 4
STRUCTURAL CONCRETE AND MATERIALS

A. Classification

A 100 Application
101 The requirements in this Chapter apply to concrete, grout and reinforcement materials to be used in structural concrete.

A 200 Concrete constituents and reinforcements
201 Approval of concrete constituents and reinforcements is based on material testing where chemical composition, mechanical properties and other specified requirements are checked against this Standard and other approved specification(s).

202 Material certification requirements for concrete constituent and reinforcements are given in this Chapter.

B. Definitions

B 100 Documentation
101 Test report is a document made by the Manufacturer which contains the results of control tests on current production, carried out on products having the same method of manufacture as the consignment, but not necessarily from the delivered products themselves. Inspections and tests shall be witnessed and signed by a qualified department different from the production department.

102 Works’ certificate is a document made by the Manufacturer which contains the results of all the required tests and which certifies that the tests have been carried out by the Manufacturer on samples taken from the delivered products themselves.

103 Mill certificate is a document as for B102, but applying to the production of cement.

B 200 General terms
201 Structural concrete is defined as a cementitious composite material and is the main ingredient for construction of concrete structures.

202 Grout is a cementitious material and includes the constituent materials; cement, water and admixture. Appropriate aggregates may be included.

203 Reinforcement is defined as the constituents of structural concrete providing the tensile strength that will give the concrete its ductile characteristics. In these Rules reinforcement is categorised as:

a) Ordinary reinforcement
b) Prestressing reinforcement
c) Special reinforcement.

204 Cement is the binder component in a structural concrete mix.

205 Aggregates are the main ingredient both with respect to volume and weight in a structural concrete mix.

206 Non-cementitious materials are defined within the context of this Standard as materials such as epoxy and polyurethane which are specially made for use together with structural concrete either to improve the concrete properties or to supplement, repair or replace the concrete.

C. Material Requirements

C 100 General
101 Requirements to material properties, composition, extent of testing, inspection etc. are given in this sub-section.

102 The materials selected for the load-bearing structures shall be suitable for the purpose. The material properties and verification that these materials fulfil the requirements shall be documented.

103 The materials, all structural components and the structure itself shall be ensured to maintain the specified quality during all stages of construction and for the intended structural life.

104 Material specifications shall be established by the Owner for all relevant materials to be used in the manufacture of structural concrete. The specification shall comply with the requirements in this Standard.

105 Materials complying with recognized standards may be accepted as an alternative to this Standard.

106 Material with properties other than specified in this Chapter may be accepted after special consideration.

107 Material may be rejected during manufacture or, after being delivered to the construction site, notwithstanding any previous acceptance or certification if it is established that the conditions upon which the approval or certification was based were not fulfilled.

108 All testing is to be performed in accordance with recognized standards and documented in accordance with the requirements of this standard. In addition, relevant requirements stated in this section and in Sec.6 and 7 shall be complied with.

109 Constituent materials for structural concrete are cement, aggregates and water. Structural concrete may also include admixtures and additions.

110 Constituent materials shall be sound, durable, free from defects and suitable for making concrete that will attain and retain the required properties. Constituent materials shall not contain harmful ingredients in quantities that can be detrimental to the durability of the concrete or cause corrosion of the reinforcement and shall be suitable for the intended use.

111 Approval of concrete constituents and reinforcements shall be based on material testing where chemical composition, mechanical properties and other specified requirements are tested according to, and are checked against, applicable International Standards and approved specifications. In lieu of relevant International Standards for specific test methods and requirements, other recognized national standards shall be used. In the absence of such standards, also recognized recommendations from international or national bodies may be used.

112 Material specifications shall be established for all materials to be used in the manufacture of concrete, in the reinforcement system and the prestressing system.

113 All testing shall be performed in accordance with recognized standards as stated in the project specification or otherwise agreed upon.

C 200 Cement
201 Only cement with established suitability shall be used. Its track record for good performance and durability in marine environments, and after exposure to stored oil if relevant, shall be demonstrated. Cement shall be tested and delivered in
accordance with a standard recognized in the place of use.

202 Cement is to be tested according to an approved method. Table C1 gives the tests and the preferred method of testing required for documentation (references to recognized standards are given).

<table>
<thead>
<tr>
<th>Property</th>
<th>Method/Apparatus</th>
<th>Code References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fineness</td>
<td>Blaine</td>
<td>C204-75, 12:2:71, 1164:4, 3049, R679-68</td>
</tr>
<tr>
<td>Oxide composition</td>
<td>C1 14-69, 4550:2:70, 1164:5, 3049</td>
<td></td>
</tr>
<tr>
<td>Normal consistency</td>
<td>Vicat</td>
<td>C187-74, 12:2:71, 1164:5, 3049</td>
</tr>
<tr>
<td>Volumetric stability</td>
<td>Le Chatelier</td>
<td>12:2:71, 3049</td>
</tr>
<tr>
<td>Initial/final set</td>
<td>Vicat</td>
<td>C191-74, 12:2:71, 1164:5, 3049</td>
</tr>
<tr>
<td>Strength in mortar</td>
<td>Rilem</td>
<td>1164:7, 3049, R679-68</td>
</tr>
</tbody>
</table>

203 The compound (mineral) composition of cements may be calculated with sufficient accuracy from Bogue's unmodified formulae, as given in ASTM C 150-74.

Guidance note:
The tricalcium aluminate (C3A) content calculated according to C202 should preferably not exceed 10%. However, as the corrosion protection of embedded steel is adversely affected by a -low C3A-content, it is not advisable to aim for values lower than approx. 5%. The imposed limits should not be too strictly enforced, but should be evaluated in each case.

---end-of-Guidance note---

204 Cement is to be delivered with a Mill Certificate containing, at least, the following information:

— Physical properties. i.e. fineness, setting times, strength in mortar, volumetric stability, normal consistency and soundness.
— Chemical composition, including mineralogic composition, loss on ignition, insoluble residue, sulphate content, chloride content and pozzolanity.

The certificate should, in addition to confirming compliance with the specified requirements, also to state the type/grade with reference to the approved standard/specification, batch identification and the tonnage represented by the document.

205 The following types of Portland cement are, in general, assumed to be suitable for use in structural concrete and/or grout in a marine environment if unmixed with other cements:

— Portland cements
— Portland composite cements
— Blastfurnace cements, with high clinker content.

Provided suitability is demonstrated also the following types of cement may be considered:

— Blastfurnace cements
— Pozzolanic cements
— Composite cement.

The above types of cement have characteristics specified in international and national standards. They can be specified in grades based on the 28-day strength in mortar. Cements shall normally be classified as normal hardening, rapid hardening or slowly hardening cements.

Guidance note:
Low heat cement may be used where heat of hydration may have an adverse effect on the concrete during curing.

---end-of-Guidance note---

C 300 Mixing water

301 Only mixing water with established suitability shall be used. The mixing water shall not contain constituents in quantities that can be detrimental to the setting, hardening and durability of the concrete or can cause corrosion of the reinforcement. Drinking water from public supply may normally be used without further investigation.

302 The required water content is to be determined by considering the strength and durability of hardened concrete and the workability of fresh concrete. The water to cement ratio by weight may be used as a measure. For requirements to W/C ratio, see D303.

303 Water resulting in a concrete strength of less than 90% of that obtained by using distilled water, shall not be used, neither shall water that reduces the setting time to less than 45 min. or change the setting time by more than 30 min. relative to distilled water, be used.

304 Salt water (e.g. raw seawater) shall not be used as mixing or curing water for structural concrete.

305 Water source(s) shall be investigated and approved for their suitability and dependability for supply.

306 Icy water may be used as mixing water provided the water melts before or during the mixing process, ensuring a resulting good mixture of the water, cement, aggregate and admixture.

C 400 Normal weight aggregates

401 Aggregate source(s) (sand and gravel) shall be investigated and approved for their suitability and dependability for supply.

Only aggregates with established suitability shall be used. Aggregates for structural concrete shall have sufficient strength and durability. They shall not become soft, be excessively friable or expand.

They shall be resistant to decomposition when wet. They shall not react with the products of hydration of the cement-forming products and shall not affect the concrete adversely. Marine aggregates shall not be used unless they are properly and thoroughly washed to remove all chlorides.

402 Aggregates shall be delivered with a test report containing, at least, the following listed information:

— Description of the source.
— Description of the production system.
— Particle size distribution (grading) including silt content.
— Particle shape, flakiness, etc.
— Porosity and water absorption.
— Content of organic matter.
— Density and specific gravity.
— Strength in concrete and mortar.
— Potential reactivity with alkalis in cement.
— Petrographical composition and properties that may affect the durability of the concrete.

403 Normal weight aggregates shall, in general, be of natural mineral substances. They shall be either crushed or uncrushed with particle sizes, grading and shapes such that they are suitable for the production of concrete. Relevant properties of aggregate shall be defined, e.g. type of material, shape, surface...
texture, physical properties and chemical properties. Aggregates shall be free from harmful substances quantities that can affect the properties and the durability of the concrete adversely. Examples of harmful substances are claylike and silty particles, organic materials, and sulphates and other salts.

404 Aggregates shall be evaluated for risk of Alkali Silica Reaction (ASR) in concrete according to Internationally recognized test methods. Suspect aggregates shall not be used unless specifically tested and approved. The approval of an aggregate that might combine with the hydration products of the cement to cause ASR shall state which cement the approval applies to. The aggregate for structural concrete shall have sufficient strength and durability.

405 An appropriate grading of the fine and coarse aggregates for use in concrete shall be established. The grading and shape characteristics of the aggregates shall be consistent throughout the concrete production.

406 Aggregates of different grading shall be stockpiled and transported separately.

407 Aggregates may generally be divided into two groups, these being:

- sand or fine aggregate (materials less than 5 mm)
- coarse aggregate (materials larger than 5 mm).

408 Maximum aggregate size is to be specified based on considerations concerning concrete properties, spacing of reinforcement and cover to the reinforcement.

409 Testing of aggregates is to be carried out at regular intervals both at the quarry and on construction site during concrete production. The frequency of testing is to be determined taking the quality and uniformity of supply and the concrete production volume into account. The frequency of testing shall be in accordance with International standards.

C 500 Lightweight aggregates

501 Lightweight aggregates in load bearing structures shall be made from expanded clay, expanded shale, slate or sintered pulverized ash from coal-fired power plants, or from other aggregates with corresponding documented properties. Only aggregates with established suitability shall be used.

502 Lightweight aggregates shall have uniform strength properties, stiffness, density, degree of burning, grading, etc. The dry density shall not vary more than ±7.5%.

C 600 Additions

601 Additions shall conform to requirements of International standards, and only additions with established suitability shall be used.

602 Additions shall not be harmful or contain harmful impurities in quantities that can be detrimental to the durability of the concrete or the reinforcement. Additions shall be compatible with the other ingredients of the concrete. The use of combinations of additions and admixtures shall be carefully considered with respect to the overall requirements of the concrete. The effectiveness of the additions shall be checked by trial mixes.

603 Latent hydraulic or pozzolanic supplementary materials such as silica fume, pulverized fly ash and granulated blast furnace slag may be used as additions. The amount is dependent on requirements to workability of fresh concrete and required properties of hardened concrete. The content of silica fume used as additions should normally not exceed 10% of the weight of Portland cement clinker. When fly ash, slag or other pozzolana is used as additions, their content should normally not exceed 35% of the total weight of cement and additions. When Portland cement is used in combination with only ground granulated blast furnace slag, the slag content may be increased. The clinker content shall, however, not be less than 30% of the total weight of cement and slag.

604 The total amount of chlorides in the fresh concrete, calculated as free calcium chloride, is not to exceed 0.3% of the weight of cement.

C 700 Admixtures

701 Admixtures to be used in concrete shall be tested under site conditions to verify that these products will yield the required effects, without impairing the other properties required. A test report is to be prepared to document such verification. The test report is to form a part of the concrete mix design documentation.

702 Relevant test report(s) from a recognized laboratory shall be submitted before use of an admixture.

703 The extent of testing is normally to be in accordance with the requirements given in recognized International Standards.

704 Air-entraining admixtures may be used to improve the properties of hardened concrete with respect to frost resistance, or to reduce the tendency of bleeding, segregation or cracking.

705 For investigations carried out under site conditions, the following properties shall be tested:

- consistence, e.g. at 5 and 30 minutes after mixing.
- water requirement for a given consistence.
- shrinkage/swelling.
- strength in compression and tension (bending) at 1-3, 28 and 91 days.

C 800 Repair materials

801 The composition and properties of repair materials shall be such that the material fulfills its intended use. Only materials with established suitability shall be used. Emphasis shall be given to ensure that such materials are compatible with the adjacent material, particularly with regard to the elasticity and temperature dependent properties.

802 Requirements for repair materials shall be subject to case-by-case consideration and approval. Deterioration of such materials when applied for temporary use shall not be allowed to impair the function of the structure at later stages.

803 The extent of testing of repair materials shall be specified in each case.

C 900 Non-cementitious materials

901 The composition and properties of non-cementitious materials shall be determined so that the material fulfills its intended use. Special emphasis is to be given to ensure that such materials are as similar as possible to the adjacent material, particular in the sense of elasticity and temperature dependent properties. Its properties shall be documented with respect to its intended application.

C 1000 Equivalent materials

1001 When using equivalent material based on experience, the equivalence shall be documented. Such documentation shall as a minimum identify the main properties including project specific requirements and parameters affecting these. It shall be demonstrated that the experience is relevant for all identified parameters.

D. Concrete

D 100 General considerations

101 Material specifications for structural concrete shall be established for all materials to be used.
D200  Concrete categorization

201  Normal Strength Concrete is a concrete of Grade C30 to C65. The Concrete Grade is derived from the characteristic cylinder strength of concrete in accordance with Sec.6, Table C1 “Concrete Grade and structural strength (MPa)”.

202  High Strength Concrete is a concrete of Grade in excess of C65.

203  Light Weight Aggregate Concrete (LWA) is a concrete made with lightweight aggregates conforming to requirements contained in recognized standards, e.g. relevant ASTM, ACI or EN standard.

204  LWA concrete may be composed using a mixture of LWA and normal weight aggregate.

D300  Concrete mix

301  The concrete composition and the constituent materials shall be selected to satisfy the requirements of this Standard and the project specifications for the fresh and hardened concrete such as consistency, density, strength, durability and protection of embedded steel against corrosion. Due account shall be taken of the methods of execution to be applied. The requirements of the fresh concrete shall ensure that the material is fully workable in all stages of its manufacture, transport, placing and compaction.

302  The required properties of fresh and hardened concrete shall be specified. These required properties shall be verified by the use of recognized testing methods, International Standards or recognized national standards. Recognized standard is relevant ASTM, ACI and EN standard.

303  Compressive strength shall always be specified, in addition tensile strength, modulus of elasticity (E-modulus) and fracture energy may be specified. Properties which can cause cracking of structural concrete shall be accounted for, i.e. creep, shrinkage, heat of hydration, thermal expansion and similar effects. The durability of structural concrete is related to permeability, absorption, diffusion and resistance to physical and chemical attacks in the given environment, a low water/cement-binder ratio is generally required in order to obtain adequate durability. The concrete shall normally have a water/cement-binder ratio not greater than 0.45. In the splash zone, this ratio shall be not higher than 0.40.

The tensile strength shall be determined based on splitting strength test.

The tests shall be carried out in accordance with the relevant International recognised standard, ASTM, ACI, EN or ISO.

304  If pozzolanic or latent hydraulic additions are used in the production of concrete, in combination with Portland cement or Portland composite cement, these materials may be included in the calculation of an effective water/cement (W/C) binder ratio. The method of calculation of effective W/C ratio shall be documented.

305  The durability of structural concrete is to be related to permeability and resistance against physical and chemical attacks.

Guidance note:
To protect the reinforcement against corrosion, and to give the concrete sufficient durability, the coefficient of permeability of concrete should be low ($10^{-12} - 10^{-8}$ m/sec). The test shall be carried out in accordance with relevant ACI, ASTM, EN or ISO standard.

This is normally obtained by use of:
- Sound and dense aggregates
- Proper grading of fine and coarse aggregates
- Rich mixes with a minimum cement content of 300kg/m$^3$
- Low water-cement ratio; i.e. not greater than 0.45

---end---of---Guidance---note---

306  Concrete subjected to freezing and thawing is to have adequate frost resistance. This requirement may be considered to be satisfied if the air content in the fresh concrete made with natural aggregates is at least 3% for a maximum particle size of 40 mm, or at least 5% for a maximum particle size of 20 mm. The air pores should be evenly distributed, with a calculated spacing factor not exceeding 0.25 mm.

307  To improve the resistance against attacks from salts in the seawater, cement with a moderate C3A content may be used, see Sec. 4 C200.

308  The total chloride ion content of the concrete shall not exceed 0.10% of the weight of cement in ordinary reinforced concrete and in concrete containing prestressing steel.

309  In the splash zone the cement content is not to be less than 400 kg/m$^3$. For reinforced or prestressed concrete not within the splash zone, the cement content is dependent on the maximum size of aggregate, as follows:
- up to 20 mm aggregate requires a minimum cement content of 360 kg/m$^3$
- from 20 mm to 40 mm aggregate requires a minimum cement content of 320 kg/m$^3$
- from 40 mm and greater the minimum required cement content is to be established by appropriate testing.

310  The concrete grades are defined as specified in Sect. 6. The properties of hardened concrete are generally related to the concrete grade. For concrete exposed to sea water the minimum grade is C45. For concrete which is not directly exposed to the marine environment, the concrete grade shall not be less than C30.

311  Where lightweight aggregates with a porous structure is used, the mean value of oven dry (105°C) density for two concrete specimens after 28 days shall not deviate by more than 50 kg/m$^3$ from the required value. Any individual value shall not deviate by more than 75 kg/m$^3$. The mean value for the entire production should lie within +20 kg/m$^3$ to -50 kg/m$^3$.

312  If the water absorption of the concrete in the final structure is important, this property shall be determined by testing under conditions corresponding to the conditions to which the concrete will be exposed.

E. Grout and Mortar

E100  General considerations

101  The mix design of grout and mortar shall be specified for its designated purpose.

102  The constituents of grout and mortar shall meet the same type of requirements for their properties as those given for the constituents of concrete.

103  The properties of fresh and hardened grout and mortar shall be specified as required for the intended use. The cement grout for injection in prestressing ducts and the ingredients used shall have adequate properties complying with the specifications from design. The strength and stiffness shall be given due attention, and be consistent with design requirements. In order to obtain stiffness commensurable with that of the concrete, special fine grain aggregates or admixtures may have to be considered.

104  All materials shall be batched by mass, except the mixing water, which may also be batched by volume. The batching shall be within an accuracy of 2% for cement and admixtures
and 1% for water. The water/cement ratio should not be higher than 0.45. Batching and mixing shall be such so that specified requirements for fluidity and bleeding in the plastic condition, volume change when hardening, and strength and stiffness when hardened are complied with.

F. Reinforcement

F 100 Reinforcement steel

101 Reinforcement shall be suitable for their intended service conditions and are to have adequate properties with respect to strength, ductility, toughness, weldability, bond properties (ribbed), corrosion resistance and chemical composition. These properties shall be specified by supplier or determined by recognized test methods.

102 Reinforcing steel shall comply with ISO 6935, Parts 2 and 3 or relevant international standards on reinforcing steel.

103 Consistency shall be ensured between material properties assumed in the design and requirements of the standard used. In general, hot-rolled, ribbed bars of weldable quality and with high ductility shall be used. Where the use of seismic detailing is required, the reinforcement provided shall meet the ductility requirements of the reference standard used in the design.

104 The design assumption made in applying the SN curves shall be consistent with the approach used in deriving these curves.

105 Reinforcement shall be delivered with a Works Certificate. The requirement for a Works Certificate may be waived if the reinforcement is produced and tested under a national or international certification scheme, and all the required test data are documented based on statistical data from the producer. All steel shall be clearly identifiable.

106 Galvanised reinforcement may be used where provisions are made to ensure that there will be no reactions with the cement that has a detrimental effect on the bond to the galvanised reinforcement.

107 Stainless steel may be used provided the requirements to mechanical properties for ordinary reinforcing steel, are met.

108 Epoxy coated reinforcement may be used provided the requirements to mechanical properties for ordinary reinforcing bars, are met.

109 Temcore reinforcement may be used provided the requirement to mechanical properties for ordinary reinforcing bars, are met.

F 200 Mechanical splices and end anchorages for reinforcement

201 Anchorage devices or couplers shall comply with national standards and be as defined in the project specification. Fatigue properties and SN-curves shall be consistent with the assumptions of the design and be documented for the actual combinations of rebars, couplers or end anchorages.

202 Mechanical splices and end anchorages shall be delivered with Works Certificate.

203 Friction welded end anchorages on rebars (T-heads) shall be qualification tested in advance with the actual type of rebar and be routinely tested during production. The test program shall include a tension test and a bend test to document strength and ductility of the connection. The friction weld shall not fail before the rebar.

F 300 Approval of welding procedures

301 Welding procedures, together with the extent of testing for weld connections relevant to reinforced concrete and concrete structures, shall be specified and approved in each case.

G. Prestressing Steel

G 100 General

101 Prestressing steel as a product shall comply with ISO 6934 and/or relevant International standards on prestressing steel.

102 Prestressing steel shall be delivered with a Works Certificate.

103 The fatigue properties (S-N curves) for the prestressing steel shall be documented.

G 200 Components for the prestressing system

201 Tendons (wires, strands, bars), anchorage devices, couplers and ducts or sheaths are part of a prestressing system described in the project specification. All parts shall be compatible and clearly identifiable.

202 Prestressing systems shall comply with the requirements of project specifications by design and shall have the approval of an authorized institution or the national authority.

203 Sheaths for post-tensioning tendons shall in general be of a semi rigid or rigid type, water tight and with adequate stiffness to prevent damages and deformations. The ducts shall be of steel unless other types are specified by design.

204 Components for the prestressing system shall be delivered with a Works Certificate.

205 Fatigue properties (S-N curves) for the complete assembly system shall be documented.

206 Friction loss parameters between the prestressing steel and the ducts/sheaths shall be documented.

H. Embedded Materials

H 100 General

101 Embedded materials, such as steel and composites, shall be suitable for their intended service conditions and shall have adequate properties with respect to strength, ductility, toughness, weldability, laminar tearing, corrosion resistance and chemical composition. The supplier shall document these properties.

I. Testing of Materials

I 100 Testing of freshly mixed concrete

101 Requirements to the testing of freshly mixed concrete are given in Sec.7 D and E.

I 200 Testing of concrete in the structure

201 Requirements to the testing of concrete in structures are given in Sec.7 F.

I 300 Grout for prestressing tendons

301 The requirements for the cement used in the grout are given in Sec.7 F. For testing of the grout, see Sec.7 F.

I 400 Reinforcement steel

401 Reinforcement steel is to be delivered with a Works Certificate. See F105.

I 500 Prestressing steel

501 Prestressing steel is to be delivered with a Works Certificate. See G102.

I 600 Mechanical splices for reinforcement
601 Mechanical splices shall be delivered with Works Certificate. See F202.

I 700 Components for the prestressing system
701 Components for the prestressing system shall be delivered with Works Certificate. See G204.

I 800 Welding procedures
801 Welding procedures together with the extent of testing (for weld connections relevant to reinforced concrete manufacture) shall be documented.

I 900 Testing of repair materials
901 The repair materials shall be documented in accordance with relevant recognised International standard, i.e. ASTM, ACI, EN and ISO.
A. Requirements to Design

100 General

The engineering of a fixed/floating offshore concrete platform shall be performed in such a way that all functional and operational requirements relating to the safety of the installation and its operation are met, as well as those requirements relating to its functions as an offshore facility.

102 The functional requirements will affect the layout of the structure including the loading scenarios that will have to be considered in the design of the structure. The functional requirements will be related to both the site-specific conditions as well as the requirements to the platform as a production facility for the production of hydrocarbons, or other activities in the operations of a field.

Site related Functional Requirements

103 The platform shall be positioned and oriented on site such that it takes account of the reservoir, other platforms, governing wind and wave direction, accessibility of ships and helicopters and safety in case of fire or leakages of hydrocarbons.

Environmental Considerations

104 There shall be a site-specific evaluation of all types of environmental conditions that can affect the layout and design of the structure, including rare events with a low probability of occurrence.

105 The deck elevation shall be determined in order to give an adequate air gap, based on site-specific data, allowing the passage of extreme wave crests higher than the design wave crest and taking due account of possible interacting ice or icebergs (if relevant). Interaction with deck supports and underwater caisson effects.

106 The water depth used in establishing layout and in the design shall be based on site-specific data taking due account of potential settlements, subsidence, etc.

Facility Operational Requirements

107 The functional requirements to be considered related to the production/operational system are such as:

a) layout of production wells, risers and pipelines, etc.

b) storage volume, compartmentation, densities, temperatures, etc. in case of stored products

c) safeguards against spillage and contamination

d) access requirements both internal and external, both for operation, inspection and condition monitoring, etc.

e) interface to topsides/plant

f) installations for supply boats and other vessels servicing the platform/installation.

108 All hazard scenarios that can be associated with the operations/maloperations and the functions of the platform shall be established and evaluated, such as fire, explosions, loss of intended pressure differentials, flooding, leakages, rupture of pipe systems, dropped objects, ship impacts, etc. The platform/installation shall be designed to give adequate safety to personnel and an adequate safety against damage to the structure or pollution to the environment.

A 200 Structural requirements

201 Structures and structural members shall perform satisfactorily during all design conditions, with respect to structural strength, mooring, stability, ductility, durability, displace-

ments, settlements and vibrations. The structure and its layout shall be such that it serves as a safe and functional base for all mechanical installations that are needed for the facility to operate. Adequate performance shall be demonstrated in design documentation.

Structural Concept Requirements

202 The structural concept, details and components shall be such that the structure:

a) has adequate robustness with small sensitivity to local damage

b) can be constructed in a controlled manner

c) provides simple stress paths that limit stress concentra-
tions

d) is resistant to corrosion and other degradation

e) is suitable for condition monitoring, maintenance and repair

f) remain stable in a damaged condition

g) fulfils requirements for removal if required.

Materials Requirements

203 The materials selected for the load-bearing structures shall be suitable for the purpose. The material properties and verification that these materials fulfil the requirements shall be documented. Requirements to concrete and reinforcement materials are given in Sec.4.

204 The materials, all structural components and the structure itself shall be ensured to maintain the specified quality during all stages of construction. The requirement to quality assurance is given in Sec.2.

Execution Requirements

205 Requirements to execution, testing and inspection of the various parts of the structure shall be specified on the basis of the significance (risk level) of the various parts with regard to the overall safety of the completed and installed structure as well as the structure in temporary conditions. See Sec.4, 7 and 8.

Temporary Phases Requirements

206 The structure shall be designed for all stages with the same intended reliability as for the final condition unless otherwise agreed. This applies also for moorings or anchorage systems applied for stages of construction afloat. Reference is made to DNV Rules for the Planning and Execution of Marine Operations.

207 For floating structures and all floating stages of the marine operations and construction afloat of fixed installations, sufficient positive stability and reserve buoyancy shall be ensured. Both intact and damaged stability should be evaluated on the basis of an accurate geometric model. Adept freeboard shall be provided. One compartment damage stabili-
ty should normally be provided except for short transient phases. The stability and freeboard shall be in accordance with DNV-OS-102 “Rules for Classification of Production and Storage Units”.

208 Weight control required for floating structures and tempo-

ry phases of fixed installations should be performed by means of well-defined, documented, robust and proven weight control. The system output should be up-to-date weight reports providing all necessary data for all operations.
A 300 Design principles

General

301 The design shall be performed according to the limit state design as detailed in DNV-OS-C101 Sec.2. The design shall provide adequate strength and tightness in all design situations such that the assumptions made are complied with:

— the design of concrete structures shall be in accordance to this Standard
— the foundation design shall be in accordance DNV-OS-C101 Sec.11
— the design of steel structures shall be in accordance to DNV-OS-C101 Sec.4, 5, 6 and 9
— the possible interface between steel structure and concrete structure shall be included in the design
— the design for load and load effects shall be in accordance with DNV-OS-C101 Sec.3. See also special requirements to concrete structures in this section
— the design for accidental limit states shall be in accordance with DNV-OS-C101 Sec.7. See also identifications of hazards in this Standard and Sec.5 for reinforced concrete design
— the cathodic protection shall be designed in accordance with DNV-OS-C101 Sec.10
— stability of the structure afloat shall be calculated in accordance with DNV-OS-102 “Rules for Classification of Production and Storage Units”.

302 Design Loads

The representative values of loads shall be selected according to DNV-OS-C101 Sec.3 and this standard.

The partial safety factors for loads shall be chosen with respect to the limit states and the combination of loads. Values are generally given in DNV-OS-C101 Sec.2 Design by LRFD Method and specifically for Reinforced Concrete in Sec.5 C100.

303 Design Resistance

The characteristic resistance of a cross-section or a member shall be derived from characteristic values of material properties and nominal geometrical dimensions.

The design resistance is obtained by amending the characteristic values by the use of appropriate partial safety factors for materials.

The design resistance shall be determined using this standard.

B. Load and Load Effects

B 100 General

101 The load and load effects shall be in accordance with DNV-OS-C101 Sec.3. The loads are generally classified as:

a) Environmental, E
b) Functional

— permanent, G
— variable, Q
— imposed deformation, D
— accidental, A.

102 The loads shall include the corresponding external reaction. The level of the characteristic loads shall be chosen according to the condition under investigation:

— under temporary conditions (construction, towing and installation)
— during operation
— when subject to accidental effects
— in a damaged condition
— during removal.

103 The load effects shall be determined by means of recognized methods that take into account the variation of the load in time and space, the configuration and stiffness of the structure, relevant environmental and soil conditions, and the limit state that is to be verified.

104 Simplified methods to compute load effects may be applied if it can be verified that they produce results on the safe side.

105 If dynamic or non-linear effects are of significance as a consequence of a load or a load effect, such dynamic or non-linear effects shall be considered.

106 Load effects from hydrodynamic and aerodynamic loads shall be determined by methods which accounts for the kinematics of the liquid or air, the hydrodynamic load, and the interaction between liquid, structure and soil. For calculation of global load effects from wind, simplified models may normally suffice.

107 Seismic load effect analyses shall be based on characteristic values described by an applicable seismic response spectrum or a set of carefully selected real or artificially simulated earthquake time histories. A combination of these methods may be used if such combination will produce a more correct result. The analysis shall account for the effects of seismic waves propagation through the soil, and the interaction between soil and structure.

Gravity Based Structures located in seismically active area shall be designed to possess adequate strength and stiffness to withstand the effect of strength level earthquake (SLE) as well as sufficient ductility to remain stable during the rare motions of greater severity associated with ductility level earthquake (DLE). The sufficiency of the structural strength and ductility is to be demonstrated by strength and, as required, ductility analyses.

The strength level earthquake (SLE) is defined as an earthquake with a recurrence period of 100 years. The ductility level earthquake (DLE) is defined as an earthquake with an individual frequency of occurrence of $10^{-4}$ per year.

108 The soil-structure interaction shall be assessed in the determination of the soil reactions used in the calculation of load effects in the structure. Parameters shall be varied with upper and lower bound values to ensure that all realistic patterns of distribution are enveloped, considering long and short term effects, unevenness of the seabed, degrees of elasticity and plasticity in the soil and, if relevant, in the structure. See DNV-OS-C101 Sec.11.

B 200 Environmental loads

201 Wind, wave, tide and current are important sources of environmental loads (E) on many structures located offshore. See Appendix A for more details. In addition, depending on location, seismic or ice loads or both can be significant environmental loads.

202 Procedures for the estimation of seismic actions are provided in DNV-OS-C101 Sec.3.

203 The computation of ice loads is highly specialized and location dependent and is not covered in detailed by this Standard. Ice loads shall be computed by skilled personnel with appropriate knowledge in the physical ice environment in the location under consideration and with appropriate experience in developing loads based on this environment and the load return periods in accordance with DNV-OS-C101 Sec.3.

B 300 Extreme wave loads

301 Wave loads from extreme conditions shall be determined by means of an appropriate analysis procedure supplemented, if required, by a model test program. Global loads on the structure shall be determined. In addition, local loads on
various appurtenances, attachments and components shall be determined. For more details see Appendix A.

B 400  Diffraction analysis

401 Global loads on large volume bodies shall generally be estimated by applying a validated diffraction analysis procedure. In addition, local kinematics, required in the design of various appurtenances, shall be evaluated including incident, diffraction and (if appropriate) radiation effects. For more details, see Appendix A.

B 500  Additional requirements for dynamic analysis under wave load

501 In cases where the structure can respond dynamically, such as in the permanent configuration (fixed or floating), during wave load or earthquakes or in temporary floating conditions, additional parameters associated with the motions of the structure shall be determined. Typically, these additional effects shall be captured in terms of inertia and damping terms in the dynamic analysis.

502 Ringing can control the extreme dynamic response of particular types of concrete gravity structure. A ringing response resembles that generated by an impulse excitation of a linear oscillator: it features a rapid build up and slow decay of energy at the resonant period of the structure. If it is important, ringing is excited by non-linear (second, third and higher order) processes in the wave loading that are only a small part of the total applied environmental load on a structure.

503 The effects of motions in the permanent configuration such as those occurring in an earthquake, floating structures or in temporary phases of fixed installations during construction, tow or installation, on internal fluids such as ballast water in tanks, shall be evaluated. Such sloshing in tanks generally affects the pressures, particularly near the free surface of the fluid.

B 600  Model testing

601 The necessity of model tests to determine extreme wave loads shall be determined on a case-by-case basis. See Appendix A for more details.

B 700  Current load

701 Currents through the depth, including directionality, shall be combined with the design wave conditions. The characteristic current load shall be determined in accordance with DNV-OS-C101 Sec.3. For more details, see Appendix A.

702 If found necessary scour protection should be provided around the base of the structure. See DNV-OS-C101 Sec.11.

B 800  Wind Loads

801 Wind loads may be determined in accordance with DNV-OS-C101 Sec.3 E700.

802 Wind forces on a Offshore Concrete Structure (OCS) will consist of two parts:

a) Wind forces on topside structure.

b) Wind forces on concrete structure above sea level.

For more details, see Appendix A.

B 900  Functional loads

901 Functional loads are considered to be all loads except environmental loads, and include both permanent and variable loads. The functional loads are defined in DNV-OS-C101 Sec.3 C “Permanent Loads” and D “Variable Functional Loads”.

902 Permanent loads (G) are loads that do not vary in magnitude, position or direction during the time period considered. These include:

- self weight of the structure
- weight of permanent ballast
- weight of permanently installed parts of mechanical outfitting, including risers, etc.
- external hydrostatic pressure up to the mean water level
- prestressing force (may also be considered as deformation loads).

903 Variable Functional Loads (Q) originate from normal operations of the structure and vary in position, magnitude, and direction during the period considered. They include loads from:

- personnel
- modules, parts of mechanical outfitting and structural parts planned to be removed during the operation phase
- weight of gas and liquid in pipes and process plants
- stored goods, tanks, etc.
- weight and pressure in storage compartments and ballasting systems
- temperatures in storages, etc. (may also be considered as deformation loads)
- loads occurring during installation and drilling operations, etc.
- ordinary boat impact, rendering and mooring.

904 The assumptions that are made concerning variable loads shall be reflected in a Summary Report and shall be compiled with in the operations. Possible deviations shall be evaluated and, if appropriate, shall be considered in the assessment of accidental loads.

905 Certain loads, which can be classified as either permanent or variable, may be treated as imposed deformations (D). Load effects caused by imposed deformations shall be treated in the same way as load effects from other normal loads or by demonstration of strain compatibility and equilibrium between applied loads, deformations, and internal forces.

906 Potential imposed deformations are derived from sources that include:

- thermal effects
- prestressing effects
- creep and shrinkage effects
- differential settlement of foundation components.

See also D401.

B 1000  Accidental loads

1001 The Accidental Loads (A) are generally defined in DNV-OS-C101 Sec.3 G Accidental Loads.

1002 Accidental loads can occur from extreme environmental conditions, malfunction, mal operation or accident. The accidental loads to be considered in the design shall be based on an evaluation of the operational conditions for the structure, due account taken to factors such as personnel qualifications, operational procedures, installations and equipment, safety systems and control procedures.

1003 Primary sources of accidental loads include:

- rare occurrences of extreme environmental loads
- fires
- flooding
- explosions
- dropped objects
- collisions
- unintended pressure difference changes.

1004 Rare occurrences of extreme environmental loads

This will include extreme environmental loads such as the extreme seismic action and all other extreme environmental loads when relevant.
The principal fire and explosion events are associated with hydrocarbon leakage from flanges, valves, equipment seals, nozzles, ground, etc. The following types of fire scenarios (relevant for offshore oil/gas production structures) should among others be considered:

- Burning blowouts in wellhead area.
- Fire related to releases from leaks in risers; manifolds, loading/unloading or process equipment, or storage tanks, including jet fire and fire ball scenarios.
- Burning oil/gas on sea.
- Fire in equipment or electrical installations.
- Pool fires on deck or sea.
- Fire jets.

The fire load intensity may be described in terms of thermal flux as a function of time and space or, simply, a standardized temperature-time curve for different locations. The fire thermal flux may be calculated on the basis of the type of hydrocarbons, release rate, combustion, time and location of ignition, ventilation and structural geometry, using simplified conservative semi-empirical formulae or analytical/numerical models of the combustion process.

### Explosions

The following types of explosions should be considered:

- Ignited gas clouds
- Explosions in enclosed spaces, including machinery spaces and other equipment rooms as well as oil/gas storage tanks.

The overpressure load due to expanding combustion products may be described by the pressure variation in time and space. It is important to ensure that the rate of rise, peak overpressure and area under the curve are adequately represented. The spatial correlation over the relevant area that affects the load effect, should also be accounted for. Equivalent constant pressure distributions over panels could be established based on more accurate methods.

The damage due to explosion should be determined with due account of the dynamic character of the load effects. Simple, conservative single degree of freedom models may be applied. When necessary non-linear time domain analyses based on numerical methods like the finite element method should be applied.

Fire and explosion events that result from the same scenario of released combustibles and ignition should be assumed to occur at the same time, i.e. to be fully dependent. The fire and blast analyses should be performed by taking into account the effects of one on the other.

The damage done to the fire protection by an explosion preceding the fire should be considered.

### Collisions

The impact loads are characterised by kinetic energy, impact geometry and the relationship between load and indentation. Impact loads may be caused by:

- Vessels in service to and from the installation, including supply vessels
- Tankers loading at the field
- Ships and fishing vessels passing the installation
- Floating installations, such as flotels
- Aircraft on service to and from the field
- Dropped or sliding objects
- Fishing gear
- Icebergs or ice.

The collision energy can be determined on the basis of relevant masses, velocities and directions of ships or aircraft that may collide with installation. When considering the installation, all traffic in the relevant area should be mapped and possible future changes in vessel operational pattern should be accounted for. Design values for collisions are determined based on an overall evaluation of possible events. The velocity can be determined based on the assumption of a drifting ship, or on the assumption of uncontrolled operation of the ship.

In the early phases of platform design, the mass of supply ships should normally not be selected less than 5000 tons and the speed not less than 0.5 m/s and 2 m/s for ULS and ALS design checks, respectively. A hydrodynamic (added) mass of 40% for sideways and 10% for bow and stern impact can be assumed.

The most probable impact locations and impact geometry should be established based on the dimensions and geometry of the structure and vessel and should account for tidal changes, operational sea-state and motions of the vessel and platform, which has free modes of behaviour. Unless more detailed investigations are done for the relevant vessel and platform, the impact zone for supply vessel on a fixed Offshore Structures should be considered to be between 10 m below LAT and 13 m above HAT.

### Dropped objects

Loads due to dropped objects should for instance include the following types of incidents:

- Dropped cargo from lifting gear
- Failing lifting gear
- Unintentionally swinging objects
- Loss of valves designed to prevent blow-out or loss of other drilling equipment.

The impact energy from the lifting gear can be determined based on lifting capacity and lifting height, and on the expected weight distribution in the objects being lifted.

Unless more accurate calculations are carried out, the load from dropped objects may be based on the safe working load for the lifting equipment. This load should be assumed to be failing from lifting gear from highest specified height and at the most unfavourable place. Sideways movements of the dropped object due to possible motion of the structure and the crane hook should be considered.

The trajectory and velocity of a falling object will be affected by entering into water. The trajectories and velocity of objects dropped in water should be determined on the basis of the initial velocity, impact angle with water, effect of water impact, possible current velocity and the hydrodynamic resistance. It is considered non-conservative for impacts in shallow water depths to neglect the above effects.

The impact effect of long objects such as pipes should be subject to special consideration.

### Unintended pressure difference changes

Changes in intended pressure differences or buoyancy caused for instance by defects in or wrong use of separation walls, valves, pumps or pipes connecting separate compartments as well as safety equipment to control or monitor pressure, shall be considered.

Unintended distribution of ballast due to operational or technical faults should also be considered.

### Floating structure in damaged condition

Floating structures, which experience buoyancy loss, will have an abnormal floating position. The corresponding abnormal-variable and environmental loads should be considered.

Adequate global structural strength should be documented for abnormal floating conditions considered in the damage stability check, as well as tightness or ability to handle potential...
leakages in the tilted condition.  

1011 Combination of accidental loads  
When accidental loads occur simultaneously, the probability level \(10^{-4}\) applies to the combination of these loads. Unless the accidental loads are caused by the same phenomenon (like hydrocarbon gas fires and explosions), the occurrence of different accidental loads can be assumed to be statistical independent. However, due attention shall be taken to the result of any quantitative risk assessment.  

Guidance note:  
While in principle, the combination of two different accidental loads with exceedance probability of \(10^{-2}\) or one at \(10^{-2}\) and the other at a \(10^{-1}\) level, correspond to a \(10^{-4}\) event, individual accidental loads at a probability level of \(10^{-4}\), commonly will be most critical.

C. Load Combinations and Partial Safety Factors  

C 100 Partial load factors, \(\gamma_f\)  
101 The Load Factors are specified in DNV-OS-C101 Sec.2 D “Design by LRFD Method” and in Table C1.  
102 The Load factors shall be calibrated, if an alternative national standard is used as a reference standard for the detailed design of the concrete structure, this to provide an equivalent level of safety. The equivalent safety shall be documented. Requirements to special evaluations are given in Appendix E.  
103 When checking the serviceability limit state, SLS, the partial load factor \(\gamma_s\) shall be 1.0 for all loads.  
104 When checking the fatigue failure limit state, FLS, the partial load factor \(\gamma_f\) shall be 1.0 for all loads.  
105 The ultimate limit state, ULS, shall be checked for two load combinations, (a) and (b), with load factors according to Table C1 (Table D1 of DNV-OS-C101 Sec.2).  

<table>
<thead>
<tr>
<th>Combination of design loads</th>
<th>Load categories</th>
</tr>
</thead>
<tbody>
<tr>
<td>(G)</td>
<td>(Q)</td>
</tr>
<tr>
<td>a)</td>
<td>1.3</td>
</tr>
<tr>
<td>b)</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Load categories:  
- \(G\) = permanent load  
- \(Q\) = variable functional load  
- \(E\) = environmental load  
- \(D\) = deformation load  

\(^{a}\) Factor may have to be amended for areas with other long term distribution functions than North Sea conditions.  

106 The loads shall be combined in the most unfavourable way, provided that the combination is physically possible and permitted according to the load specifications. Loading conditions that are physically possible but not intended or permitted to occur in expected operations shall be included by assessing probability of occurrence and accounted for as either accidental conditions in the accidental damage limit state (ALS) or as part of the ordinary design conditions included in the ULS. Such conditions may be omitted in cases where the annual probability of occurrence can be assumed to be less than \(10^{-4}\).  

107 For permanent loads, a load factor of 1.0 in load combination a) shall be used where this gives a more unfavourable load effect. For external hydrostatic pressure, and internal pressures from a free surface, an load factor of 1.2 may normally be used provided that the load effect can be determined with normal accuracy. Where second order effects are important, a load factor of 1.3 shall be used.  
108 A load factor of 1.0 shall be applied to the weight of soil included in the geotechnical calculations.  
109 Prestressing loads may be considered as imposed deformations. Due account shall be taken of the time dependent effects in calculation of effective characteristic forces. The more conservative value of 0.9 and 1.1 shall be used as a load factor in the design.  
110 The definition of limit state categories is valid for the foundation design with the exception that failure due to cyclic loading is treated as an ULS, alternatively as an ALS, using load and material coefficients as defined for these limit state categories.  
111 Where a load is a result of high counteracting and independent hydrostatic pressures, the pressure difference shall be multiplied by the load factor. The pressure difference shall be taken as no less than the smaller of either one tenth of the highest pressure or 100 kPa. This does not apply when the pressure is balanced by direct flow communication. The possibility of communication channel being blocked shall then be part of the risk assessment.  
112 In the ALS, the Load factor shall be 1.0 for all loads.  

C 200 Combinations of loads  
201 Table B2 of DNV-OS-C101 Sec.3 B gives a more detailed description of how loads shall be combined. When environmental and accidental loads are acting together, the given probabilities apply to the combination of these loads.  
202 For temporary phases, where a progressive collapse in the installation does not entail the risk of loss of human life, injury, or damage to people or the environment, or of significant financial losses, a shorter return period than that given in Table B2 of DNV-OS-C101 for environmental loads may be considered.  
203 The return conditions to be considered should be related to the duration of the operation. As a general guidance, the criteria given in Table C2 may be applied:  

<table>
<thead>
<tr>
<th>Table C2 Environmental criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Duration of use</strong></td>
</tr>
<tr>
<td>Up to 3 days</td>
</tr>
<tr>
<td>3 days to 1 week</td>
</tr>
<tr>
<td>1 week to 1 month</td>
</tr>
<tr>
<td>1 month to 1 year</td>
</tr>
<tr>
<td>More than 1 year</td>
</tr>
</tbody>
</table>

C 300 Consequence of failure  
301 Structures can be categorised by various levels of exposure to determine criteria that are appropriate for the intended service of the structure. The levels are determined by consideration of life safety and consequences of failure.  
302 Life safety considers the manning situation in respect of personnel on the terminal when the design environmental event would occur.  
303 Consequences of failure consider the potential risk to life of personnel brought in to react to any incident, the potential risk of environmental damage and the potential risk of economic losses.
D. Structural Analysis

D 100 General

101 Structural analysis is the process of determining the load effects within a structure, or part thereof, in response to each significant set of loads. This clause specifies requirements for the various forms of structural analysis necessary to define the response of the structure during each stage of its life. Load effects calculated by structural analysis shall be used as part of the design.

102 Complex or unusual structural types can require forms of analysis, which are not described within this Standard. These shall be performed in accordance with the principle of providing sufficient analyses to accurately assess all significant load effects within the structure.

103 In order to ascertain successful analysis of an offshore concrete structure it is required that:

— all necessary analyses are performed on the basis of an accurate and consistent definition of the structure and assessment of loads thereon
— these analyses are performed using appropriate methods, have accurate boundary conditions and are of suitable type
— suitable verified results are available in due time for use in design or reassessment.

104 Interfaces with structural designers, topsides designers, hydrodynamics, geotechnical engineers and other relevant parties shall be set up. The schedule of supply of data regarding loads (including reactive actions) shall be determined and monitored. Such an interface shall ensure that this data is in the correct format, covers all necessary locations and is provided for all required limit states and for all significant stages in the lifetime of the structure.

105 The number and extent of analyses to be performed shall cover all components of the structure through all stages of its life, i.e. construction, installation, in-service conditions and removal/retrieval/relocation. However, if it can be clearly demonstrated and documented that particular stages in the life of a component will not govern its design, such stages need not be analysed explicitly for this component.

106 Sufficient structural analyses shall be performed to provide load effects suitable for use when checking all components of the primary structure for the required design conditions and limit states. At least one such analysis should normally represent global behaviour of the structure for each significant stage of its life.

107 Secondary components of the structure shall be assessed, by analysis if necessary, to determine their integrity and durability, and to quantify the distribution of load effects on the primary structure. Such analyses may be performed in isolation of the primary structure analysis, but shall include deformations of the supporting primary structure, where significant.

108 When present, the stiffness of the topsides and other primary structures shall be simulated in global analyses in sufficient detail to adequately represent the interface with the concrete substructure, such that all loads from the topsides are appropriately distributed to the concrete substructure. The relative stiffness of topsides and concrete substructure shall be accurately simulated where this has a significant effect on global load paths and load effects. Particular attention shall be paid to relative stiffness when assessing dynamic response.

109 Where appropriate, the analysis shall include a representation of its foundation, simulated by stiffness elements or by reactive loads.

110 All structural analyses required for design of the structure shall be carried out in accordance with the planned analysis schedule using the most recent geometric, material, boundary condition, load and other data.

111 The structure shall be analysed for significant loads during each stage of its life. Where simultaneous loading is possible, these loads shall be applied in combination in such a way as to maximize load effects at each type of location to be checked. The loads that contribute to these combinations shall include appropriate load factors for each limit state being checked.

112 Where assumptions are made to simplify the analysis and enable a particular calculation method, these shall be clearly recorded in the documentation or calculations. The effects of such assumptions on load effects shall be quantified and incorporated as necessary.

113 Analysis of the global structure or local components is normally performed by the finite element method. Computer software used to perform finite element analysis shall comply with a recognized international quality standard, such as ISO 9000-3 or shall be verified for its intended use prior to the start of the analysis. Element types, load applications, meshing limits and analysis types to be used in the structural analysis shall all be included in the verification.

114 Where finite element analysis is performed, consideration shall be given to the inaccuracy inherent in the element formulation, particularly where lower order elements or coarse element meshes are used. Verification and "benchmark" testing of the software shall be used to identify element limitations and the computer modelling shall be arranged to provide reliable results.

115 Hand calculations are generally limited to simple components of the structure (beams, regular panels, secondary structures, etc.) under simplified loads (i.e. uniform pressure, point or distributed loads). The methodology used shall reflect standard engineering practice with due consideration for the conditions of equilibrium and compatibility. Elastic or plastic design principles may be adopted dependent on the limit state being checked and the requirements for the analysis being performed.

116 Computer spreadsheets are electronic methods of performing hand calculations and shall be subject to the same requirements. Where such spreadsheets do not produce output showing the methodology and equations used, adequate supporting calculations shall be provided to verify the results of comprehensive test problems. Sufficient checks shall be provided to verify all facilities in the spreadsheet that will be used for the component being assessed.

117 Special forms of analysis for concrete structures, such as the strut and tie approach, may also be used, but must conform to contemporary, accepted theories and shall adhere to the general principles of civil/structural engineering. Unless the method is well known and understood throughout the industry, references to source material for the method being used shall be provided in the documentation or calculations.

118 Non-linear finite element analysis may be used to demonstrate ultimate capacity of the structure or the capacity of complicated 2-D and 3-D (discontinuity) regions. Software used for this purpose shall be subject to the same verification requirements as above. Verification of non-linear analysis software used in this way shall include at least one comparison against experimental results or a reliable worked example of a similar detail.

119 Structural analyses shall be thoroughly verified to provide confidence in the results obtained. Verification is required to check that input to the calculations is correct and to ensure that sensible results have been obtained.

120 Input data for a particular structural analysis shall be subject to at least the following checks:

— that the model adequately represents the geometry of the intended structure or component.
Verification of the results of an analysis shall be recorded and pertinent parties informed of results and conclusions so that implications for the design process are formally recognized.

Each structural analysis shall be thoroughly documented to record its extent, applicability, input data, verification and results obtained. The following information shall be produced as a minimum to document each analysis:

- the purpose and scope of the analysis and the limits of its applicability
- references to methods used and the justification of any assumptions made
- the assumed geometry, showing and justifying any deviations from the current structural geometry
- material properties used in the analysis
- boundary conditions applied to the structure or component
- the summed magnitude and direction of all loads
- pertinent results from the analysis and crosschecks to verify the accuracy of the simulation
- a clear presentation of those results of the analysis that are required for further analysis, structural design or reassessment.

Results of the analysis will normally take the form of load effects, for which the structure shall be designed to withstand. Typical load effects required for the design of fixed offshore concrete structures include the following:

- displacements and vibrations, which shall be within acceptable limits for operation of the platform
- section forces, from which the capacity of concrete sections and necessary reinforcement requirements can be determined
- section strains, used to determine crack widths and water tightness; stress occurrences, used to check the fatigue life of the structure.

Modulus of elasticity to be used in load effect analyses

In the calculation of strains and section forces under short-term loading in the serviceability limit state the relation between modulus of elasticity of concrete $E_c$ and compressive cylinder strength $f_{ck}$ is given by the equation

$$E_c = k_c f_{ck}^{0.3}$$

201 The factor $k_c$ may be determined by testing E-modulus at unloading in accordance with appropriate International standard. The strength $f_{ck}$ is determined with the same cylinder samples. The factor shall be determined as the mean value of the test results from at least 5 concrete test mixes with the same aggregates and strength which will be used in the prospective concrete.

For concrete Grades C30 to C85 (see Sec.6 Table C1), the factor $k_c$, may be taken as 9 500 (N/mm²) ^ {0.3}, if the factor is not determined by testing.

202 Calculations in the serviceability limit state shall be performed using the characteristic value $E_{ck}$, computed using the actual characteristic cylinder strength $f_{ckj}$ in the formula. In order to calculate the most probable displacement the mean cylinder strength may be used.

203 To consider loading of early age concrete the characteristic cylinder strength at the actual time of loading $f_{ckj}$, may be used.

204 The characteristic modulus of elasticity of concrete $E_{ck}$ can be used for calculations of forces and moments in the ultimate limit state, except in cases where structural displacements cause increased forces and moments, see D2100.

205 If the modulus of elasticity of lightweight aggregate concrete is not determined by testing, the modulus of elasticity shall be reduced by multiplying by a factor $(\rho / \rho_1)^{1.5}$ where $\rho_1 = 2400$ kg/m³.

206 For impact type of loading or rapid oscillations the modules of elasticity calculated according to D201 or D202 can be increased by up to 15 %, dependent on strain rate.

207 If the modulus of elasticity of lightweight aggregate concrete is not determined by testing, the modulus of elasticity shall be reduced by multiplying by a factor $(\rho / \rho_1)^{1.5}$ where $\rho_1 = 2400$ kg/m³.

208 For impact type of loading or rapid oscillations the modules of elasticity calculated according to D201 or D202 can be increased by up to 15 %, dependent on strain rate.

209 The modulus of elasticity predicted in D201 may be used for a temperature range from -50°C to 100°C. For short-term temperatures (fire) that range from 100°C to 200°C the modulus of elasticity can be taken as 90 per cent of $E_{ck}$ given in D201. For temperatures above 200°C the concrete strain properties, including creep and thermal strain, shall be determined specially.

210 The characteristic modulus of elasticity of non-prestressed reinforcement may be taken as $E_{sk} = 200 000$ MPa.

211 At high temperatures of short duration (fire) the modulus of elasticity of steel may be taken according to D210 for temperatures up to 200°C as long as more precise values are not known. For temperatures above 200°C the strain properties of steel shall be determined separately.

212 For prestressed reinforcement the force-strain relationship shall be known for the steel type and make in question.

Effects of temperature, shrinkage, creep and relaxation

The linear coefficient of thermal expansion ($\alpha$) for both NW concrete and reinforcement shall be taken as 10^-5 per °C when calculating the effects of thermal loads, unless there is adequate basis for selecting other values.

The linear coefficient of thermal expansion for LWA concrete shall be determined for the actual concrete mix design.

Where the temperature induced loads are significant testing is normally to be carried out to determine ($\alpha$).

Values of concrete creep and shrinkage shall be chosen on the basis of the climatic surroundings of the structure, sectional dimensions, concrete mixture and age.

The creep strain is assumed to be proportional to the concrete stress when load effects are calculated. At constant concrete stress, the creep strain is

$$\varepsilon_{cc} = \phi \varepsilon_{c} = \phi \sigma_{c}/E_{ck}$$

where:

- $\phi$ is the creep coefficient
- $\varepsilon_c$ is the concrete stress due to long-term loading

For all loads the creep strain shall be calculated in proportion to the duration of the load.

If creep is considered in the calculation of forces due to shrinkage, it can be assumed that both creep and shrinkage...
have the same time dependent development.

306 For lightweight aggregate concrete the creep coefficient \( \phi \) can be assumed equal to the value of normal concrete multiplied by a factor \((\rho / \rho_1)^{1.5}\) for \( \rho > 1800 \text{ kg/m}^3 \). For lightweight concrete, \( \rho < 1500 \text{ kg/m}^3 \), a factor 1.2 \((\rho / \rho_1)^{1.5}\) can be used. For intermediate values of \( \rho_1 \) linear interpolation may be applied.

\[
\rho_1 = 2400 \text{ kg/m}^3
\]

307 The effect of relaxation in prestressed reinforcement shall be calculated in proportion to the time period over which the relaxation occurs. If there are no exhaustive test results available for the steel type and make in question, the values given in Figure 1 can be used. Normally, testing is expected to be based on at least 10,000 hours loading.

308 If the steel experiences a temperature, \( T \), higher than \( T_1 \) = 20°C for a long period of time, a quantity \( k_1(T-T_1) \) shall be added to the relaxation in percent of relaxation stress found in the figure, where the factor \( k_1 \) for

- cold drawn, untempered steel is 0.15% per °C
- cold drawn, tempered steel is 0.10% per °C.

These values shall not be used, if the steel temperatures exceed 80°C, for long periods of time.

![Figure 1](image-url)

**Figure 1**

Long-term relaxation in prestressing steel

**D 400 Special load effects**

**Deformation Loads**

401 Deformation induced loads created by imposed deformations in the structure, are loads to be treated as either deformation loads (D) or as Functional Loads. See B900.

Examples of such loads may be:

- differential settlement
- temperature effects
- shrinkage
- loads in flexible members connected to stiff members may in some cases be seen as deformation induced loads
- changes in strain due to absorption.

In case of a ductile mode of failure, and where second order effects are negligible, the effect of deformation loads may normally be neglected.

A typical example of a ductile mode of failure is a flexural failure provided sufficient rotational capacity exists. Verification of sufficient rotational capacity may in most cases be based on simplified considerations.

402 Imposed deformations normally have a significant influence on the shear resistance of a section, and shall be duly considered in the design.

The characteristic value of deformation imposed loads is normally evaluated on the basis of defined maximum and minimum values for the parameters governing its magnitude.

An accurate calculation of deformation loads caused by temperature effects can only be obtained from a non-linear analysis, reflecting realistic material properties of reinforced concrete.

In practice, 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 by reducing the flexural and axial stiffness to account for cracking of the concrete. Special considerations and documentation of the stiffness shall be required.

403 Creep effects shall be considered where relevant. An accurate calculated assessment of creep in shell structures can only be obtained by computer calculations using non-linear finite-element programs. However, rough estimates of creep effects may be obtained by methods originally developed for simple columns. In the following two methods are referred to, the so-called “creep factor method” and the “creep eccentricity method”. Reference is made to Sec.6 C705.

**Effect of Water Pressure**

404 The effect of water pressure in the concrete is to be fully considered when relevant.

405 The effect of hydrostatic pressure on the concrete strength is to be evaluated where relevant. (For lightweight aggregate concrete, this effect may be significant.)

406 The effect of hydrostatic forces acting on the faces of cracks is to be taken into account in the analytical models used for prediction of concrete cross-sectional strength. This effect is also to be taken into account when actual load effects are evaluated. Effects of water pressure in cracks may be neglected for structural elements exposed to less than 100m of waterhead.

**Loss of Intended Underpressure**

407 For structures designed with an intended underpressure, relative to external pressure, a design condition where the intended underpressure is lost is to be evaluated.

This load effect may be categorized as an accidental load effect. Load combinations, and load and material factors are then to be taken according to ALS criteria.

More stringent criteria may be specified by the Client for this situation (e.g. increased material factor, load factors etc.) due to e.g. costly and excessive repair or if the structure is storing oil (risk of oil spillage).

**Weight of Concrete**

408 The long-term effect of water absorption is to be considered in the estimation of concrete weights in particular for floating structures.

**D 500 Physical representation**

501 Dimensions used in structural analysis calculations shall represent the structure as accurately as necessary to produce reliable estimates of load effects. Changes in significant dimensions as a result of design changes shall be monitored both during and after the completion of an analysis. Where this impacts on the accuracy of the analysis, the changes shall be incorporated by reanalysis of the structure under investigation. For more details see Appendix B.
D 600 Loads

601 Loads shall be determined by recognized methods, taking into account the variation of loads in time and space. Such loads shall be included in the structural analysis in a realistic manner representing the magnitude, direction and time variance of such loads. For more details see Appendix B.

D 700 Mass simulation

701 A suitable representation of the mass of the structure shall be required for the purposes of dynamic analysis, motion prediction and mass-acceleration loads while floating. For more details see Appendix B.

D 800 Damping

801 Damping arises from a number of sources including structural damping, material damping, radiation damping, hydrodynamic damping and frictional damping between moving parts. Its magnitude is dependent on the type of analysis being performed. In the absence of substantiating values obtained from existing platform measurements or other reliable sources, a value not greater than 3% of critical damping may be used.

D 900 Linear elastic static analysis

901 It is generally acceptable for the behaviour of a structure or component to be based on linear elastic static analysis unless there is a likelihood of significant dynamic or non-linear response to a given type of loading. In such cases, dynamic or non-linear analysis approaches shall be required. For further details with respect to structural analyses, see Appendix C.

D 1000 Dynamic analysis

1001 Fixed structures with natural periods of the global structure greater than 2.5s can be susceptible to dynamic response due to wave load during in-service conditions, at least for fatigue assessment. Structures in shallow water or subject to extreme wave conditions may exhibit significant dynamic response at lower periods due to the higher frequency content of shallow water or particularly steep waves. For further details with respect to dynamic analyses, see Appendix C.

D 1100 Pseudo-static analysis

1101 In this context, pseudo-static analysis refers to any analysis where dynamic loads are represented approximately by a factor on static loads or by equivalent quasi-static loads. The former approach is appropriate where static and dynamic load effects give an essentially similar response pattern within the structure, but differs in magnitude. For further details, see Appendix C.

D 1200 Non-linear analysis

1201 Non-linear behaviour shall be considered in structural analysis when determining load effects in the following cases: where significant regions of cracking occur in a structure such that global load paths are affected:

- where such regions of cracking affect the magnitude of loads (temperature loads, uneven seabed effects, dynamic response, etc.)
- where the component depends upon significant non-linear material behaviour to resist a given set of loads, such as in response to accidents or ductility level seismic events
- for slender members in compression, where deflection effects are significant.

For further details, see Appendix C.

D 1300 Probabilistic analysis

1301 It is generally acceptable to base in-service structural analysis of an offshore concrete structure subjected to wave load on the principles of deterministic analysis, predicting response to specific events. However, where stochastic or probabilistic methods are shown to be more appropriate for a particular limit state (i.e. fatigue), these shall be substituted as needed. Spectral fatigue analysis is normally required where structural dynamics are significant.

1302 Such methods typically linearize load effects. This can restrict their use where non-linear response of the structure or component is significant. If non-deterministic analysis methods are still to be used, time domain response to transient loading might be necessary.

1303 Where spectral analysis methods are used for calculating response to random wave load, sufficient wave conditions shall be analysed to ensure that dynamic response close to structural natural periods and peak wave energy is accurately assessed.

D 1400 Reliability analysis

1401 Reliability assessment of structures is permitted under these rules to assess the risk of failure of a structure and ensure that this fails below acceptable levels. Such analysis shall be performed in accordance with acceptable current practice.

D 1500 Analyses requirements

1501 All structural analyses performed shall simulate, with sufficient accuracy, the response of the structure or component for the limit state being considered. This may be achieved by appropriate selection of the analysis type with due regard to the nature of loads applied and the expected response of the structure.

1502 The following table gives general guidance as to the type of analysis that shall be adopted for each design condition for the structure. Further details are provided in D1600 to D2300.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Appropriate types of analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>Linear static analysis is generally appropriate.</td>
</tr>
<tr>
<td>Towing to location</td>
<td>Linear static analysis is generally appropriate. Dynamic effects may be significant in response to hydrodynamic motions. These can normally be simulated by pseudo-static analysis.</td>
</tr>
<tr>
<td>Installation</td>
<td>Linear static analysis is generally appropriate.</td>
</tr>
<tr>
<td>In-service strength and Serviceability</td>
<td>Linear, static or pseudo-static analysis is generally appropriate for global load path analysis.</td>
</tr>
<tr>
<td>Fatigue</td>
<td>Linear analysis is generally appropriate. Dynamic effects may be significant for short period waves. A pseudo-static deterministic approach is normally acceptable.</td>
</tr>
<tr>
<td>Seismic</td>
<td>Dynamic analysis is normally required, where seismic ground motion is significant. Non-linear effects might need to be considered for ductility level earthquakes.</td>
</tr>
<tr>
<td>Accidental</td>
<td>Non-linear analysis is normally required for significant impacts. Dynamic response can be significant.</td>
</tr>
<tr>
<td>Removal/ reuse</td>
<td>As per transportation and installation.</td>
</tr>
</tbody>
</table>

D 1600 Analysis of construction stages

1601 Sufficient analyses shall be performed on components of the structure during construction to ensure their integrity at all significant stages of the construction and assembly process and to assess built-in stresses from restrained deformations. Construction stages shall include onshore and inshore operations.

1602 Consideration shall be given to the sequence of construction in determining load effects and to the age of the concrete in determining resistance. Specific consideration shall be given to the stability of components under construction. Adequate support for temporary loads, such as crane footings, shall be provided in the analysis.
Assessment of the structure during construction stages may normally be performed using static analysis. However, dynamic response to wind turbulence might need to be calculated for tall, slender structures and consideration shall be given to other possible dynamic load effects, such as earthquakes, occurring during the construction phase.

Long term stress redistribution shall be considered for the complete structure considering creep effects on the built stresses accumulated during construction.

D 1700 Transportation analysis

1701 Analysis of a fixed concrete structure shall include the assessment of structural integrity during significant stages of the sea tow of the structure, whether it is self-floating, barge supported or barge assisted. The representation of the structure during such operations shall be consistent with the stage being represented, incorporating the correct amount of ballast and simulating only those components of the topsides actually installed.

1702 Analysis during sea tow should normally be based on linear static analysis, representing the motion of the concrete structure by peak heave, sway, surge, pitch and roll accelerations as predicted by hydrodynamic analysis. For such analysis to be valid, it shall be demonstrated that motions in the natural periods of major components of the structure, such as the shafts, will not be significantly excited by this global motion. If dynamic effects are deemed important, they shall be incorporated in accordance with D1000. The analysis of the tow shall be in accordance with the DNV Rules for Planning and Execution of Marine Operations.

1703 Fatigue damage can result from extreme tow duration in heavy seas. If this is significant, fatigue damage accrued shall be accumulated together with that calculated for in-service conditions in accordance with D2000.

1704 Consideration shall be given to possible damage scenarios during sea tow. Sufficient structural analyses should be performed to ensure adequate integrity of the structure, preventing complete loss in the event of collision with tugs or other vessels present during the transportation stage. In particular, progressive collapse due to successive flooding of compartments shall be prevented.

D 1800 Installation and deck mating analysis

1801 Structural analysis shall be performed at critical stages of the deck mating and installation stages. Such analyses shall, as a minimum, cover times of maximum pressure differential across various components of concrete structure. Once again, the configuration of the structure at each stage of the setting down operation should reflect the planned condition and inclination of the structure and the associated distribution of ballast.

1802 Deck mating, ballasting down and planned setting down on the sea floor shall normally be analysed by a linear static approach. As these phases normally represent the largest external water heads, implosion or buckling should be analysed. The effect of unevenness in the seabed shall be considered in assessing seabed reactions in an ungrouted state.

D 1900 In-service strength and serviceability analyses

1901 At least one global analysis of the structure shall be performed in its in-service configuration suitable for subsequent strength and serviceability assessment. The structure shall also be analysed for extreme wave effects using ALS load factors, unless it can be conclusively demonstrated that this limit state is always less onerous than the corresponding ULS condition.

1902 Local analysis shall be performed to assess secondary structure and details that appear from the global analysis to be heavily loaded or that are complex in form or loading. Such analyses may be based on non-linear methods if these are more appropriate to the component behaviour.

1903 It is generally acceptable to base all strength analysis of an in-service concrete platform on deterministic analysis, predicting response to specific extreme waves. Sufficient wave periods, directions and wave phases shall be considered to obtain maximum response in each type of component checked. Consideration shall be given to waves of lower than the maximum height if greater response can be obtained due to larger dynamic effects at smaller wave periods.

D 2000 Fatigue analysis

2001 When required, detailed fatigue analysis shall be based on a cumulative damage assessment performed over the proposed lifetime of the structure. The analysis shall include transportation stages, if significant, and should consider the effects of the range of sea states and directions to which the structure will be subjected.

2002 A linear representation of the overall structure is generally acceptable for the evaluation of global load paths for fatigue analysis. The structural analysis shall include the effects of permanent, live, hydrostatic and deformational loads. It shall be justifiable to use reduced topside and other loads in the fatigue analysis, on the basis that typical rather than extreme loads through its life are required. Significant changes in static load during the lifetime of the structure shall be analysed separately and fatigue damage shall be accumulated over each phase.

2003 Dynamic amplification is likely to be more significant for the relatively short wave periods causing the majority of fatigue damage. Fatigue analysis shall therefore consider the effects of dynamic excitation in appropriate detail, either by pseudo-static or by dynamic response analysis. Deterministic or stochastic types of analysis are both permissible, subject to the following provisions.

2004 For deterministic analysis, the selected individual waves to which the structure is subjected shall be based on a representative spread of wave heights and periods. For structures that are dynamically sensitive, these shall include several wave periods at or near each natural period of the structure, to ensure that dynamic effects are accurately assessed. Consideration shall also be given to the higher frequency content in larger waves that may cause dynamic excitation.

2005 Sufficient wave cases shall be analysed for probabilistic analysis to adequately represent the stress transfer functions of the structure. Non-linear response of the structure shall be incorporated into the analysis using appropriate methods, if significant.

D 2100 Seismic analysis

2101 Two levels of seismic loading on an offshore concrete structure shall be considered:

— strength level earthquake (SLE), which shall be assessed as a ULS condition
— ductility level earthquake (DLE), for which ductile behaviour of the structure assuming extensive plasticity is permissible provided the structure survive.

For further details, see Appendix D.

D 2200 Accidental and overload analyses

2201 Analysis of the structure under accidental conditions, such as ship collision, helicopter impact or iceberg collision, shall consider the following:

— local behaviour of the impacted area
— global strength of the structure against overall collapse
— post-damage integrity of the structure.

2202 The resistance of the impact area may be studied using
local models. The contact area and perimeter shall be evaluated based on predicted non-linear behaviour of the structure and of the impacting object. Non-linear analyses may be required since the structure will generally deform substantially under the accidental loads. Appropriate boundary conditions shall be provided far enough away from the damaged region for inaccuracies to be minimized.

2203 Global analysis of the structure under accidental loads may be required to ensure that a progressive collapse is not initiated. The analysis should include the weakening effect of damage to the structure in the impacted area. When large deformations of the structure is likely for the impact loads, a global non-linear analysis may be required to simulate the redistribution of load effects caused by the large deformations. The global analysis may be based on a simple representation of the structure sufficient to simulate progressive collapse. Deflection effects shall be included, if significant.

2204 Energy absorption of the structure will arise from the combined effect of local and global deformation. Sufficient deformation of the structure to absorb the impact energy from the collision not absorbed by the impacting object shall be documented.

2205 Analysis of the structure in its damaged condition may normally be performed using linear static analysis. Damaged components of the structure shall be removed from this analysis, or appropriately weakened to simulate their reduced strength and stiffness.

D 2300 Platform removal/reuse

2301 Analysis of the structure for removal shall accurately represent the structure during this phase. The analysis shall have sufficient accuracy to simulate pressure differential effects that are significant during this stage. The analysis shall include suction forces that shall be overcome prior to separation from the sea floor, if appropriate. Suitable sensitivity to the suction coefficient shall be incorporated. The possibility of uneven separation from the seabed and drop-off of soil or underbase grout shortly after separation shall be considered and structural response to subsequent motions shall be evaluated.

2302 Weights of accumulated debris and marine growth shall also be considered if these are not to be removed. Items to be removed from the structure, such as the topsides, conductors, and risers, shall be omitted from the analysis.

2303 The condition of the concrete and reinforcement should account for degradation of the materials during the life of the platform. If the analysis is carried out immediately prior to removal, then material degradation shall take account of the results from recent underwater surveys and inspections.

E. Topside Interface Design

E 100 Introduction

101 The design of the interface between a steel topsides structure and a concrete substructure requires careful consideration by both the topsides and substructure designers.

102 Particular attention shall be paid to ensure that all relevant information is exchanged between the topsides and substructure design teams.

103 If topside and substructure construction are separate contracts, special care shall be taken to handle the interface responsibility. It shall be clear who is responsible for input to and from the topside engineering contractor as part of a technical coordination procedure.

E 200 Basis for design

201 As part of establishing and maintaining adequate handling of topside/substructure interface throughout the design process, all necessary design information must be defined. Plans must be prepared in order to secure timely supply of data. The interface shall define format of data, ensure consistency with respect to locations and elevations, and that data is provided for all required limit states and significant stages in the lifetime of the structure such as:

- installation/mating of topside
- the platform transportation and installation
- the platform operating phase
- decommissioning.

202 Important aspects related to these phases are time-dependent deformations such as creep, effect of varying water pressure at different drafts, varying ground-pressure distribution under the base, accelerations and possible inclination during tow as well as resulting from accidental flooding. Varying shaft inclination in temporary phases prior to installation/mating of the topside might cause built-in stresses to be dealt with in the design of topside, substructure and the deck-shaft connection. It is of vital importance that the design assumptions are consistent.

203 The structural analysis of the concrete substructure may consider the topside in varying detail and sophistication depending on its effect on the design of different structural parts. Typically the design of upper parts of the substructure (shaft) is based on FE-analysis comprising also the topside stiffness matrix. It is required that the stiffness of the topside and the load effects imposed by the topside is represented in sufficient detail to ensure adequate distribution between topside and substructure, as well as within the substructure.

204 The documentation to be provided as basis for proper interface design must also cover:

- shaft configuration
- top of shaft layout
- deck elevation
- loads to be applied on top of concrete structure from topside (i.e. topside weights for design purposes incl. CoG, etc.)
- tolerances (i.e. for concrete geometry, tie bolts, tendons, bearing tubes, embedment plates, etc.)
- deck mating tolerances to allow for deformations during load transfer.

E 300 Deck/shaft structural connection

301 Several alternatives are viable for the structural connection between the topside and the substructure. The detailing must consider initial contact and ensure load distribution as presumed in structural analysis and design.

302 The physical interface is very often present between a steel Module Support Frame (MSF) and the Offshore Concrete Structure. Typically temporary tubular bearings (steel pipes) resting on embedded steel plates are used for transferring the deck weight on top of OCS shafts. The area between the tubular bearings is typically grouted before activation of pre-stressed anchor bolts.

303 The design of intersection between MSF, grout and top of shaft(s) must take due account of shear forces (friction check) arising from tilt in temporary phases or platform accelerations in the operational phase. Compression check is required for the grout. Eventual uplift must also be accounted for.

304 If a non-rigid topside to substructure connection is selected, such as an array of elastomeric bearings, consideration should be given to the expansion and contraction of oil risers heated by hot products and the interaction between rigid pipes and a flexible structural connection.

305 Depending on the connection selected, the detailing and...
layout must allow for necessary inspection and maintenance. Special consideration should be given to gaining access to fatigue prone details and, if access is not possible, a suitably large design fatigue life should be selected. Any materials used should be assessed for chemical stability under the effects of high heat, moisture and hydrocarbon contamination. The means of corrosion control selected for the concrete substructure (such as cathodic protection) should be clearly communicated.

**E 400 Topsides - substructures mating**

401 While the selection of an installation method affects both substructure and topside design, one must ensure that such consequences are addressed at an early stage.

402 Typical items and effects to be considered are:

— dynamic response to waves and currents of the submerged structure if a float-in installation is required
— dynamic response to wave, winds and currents of a partially submerged substructure for a lift installation of topsides
— design of installation aids for both lift and float-in installations.

Sufficient tolerances shall be incorporated in the design for the mating operation.

**E 500 Transportation**

501 The dynamic motions during the towage of fixed concrete installations are usually small. Accelerations, tilting angles in intact and damaged conditions, must be accurately defined and the consequences for design of topside, substructure and their connections, must be dealt with.

**F. Barge Type Structures**

**F 100 General**

101 Response values for barge type structures are given in “DNV Rules for Ships” January 2001 Pt.3 Ch.1 Sec.4 (Design Loads), Sec.5 (Longitudinal Strength) and Sec.6 (Bottom Structures). The design values for motions and accelerations are extreme values (Probability level = 10⁻⁸).

102 The Design pressures caused by sea, liquid cargos, dry cargoes, ballast and bunkers are based on extreme conditions, but are modified to equivalent values corresponding to the stress levels stipulated in the Ship Rules. Normally, this involves a reduction of the extreme values given for motions and accelerations to a 10⁻⁴ probability level.

103 The Offshore Concrete Structure will normally be exposed to environmental loads different from the basis for the ship rules which covers general environmental loads in open sea. The design response given in “DNV Rules for Ships” should be used for guidance only. The design load response using “DNV Rules for Ships” are generally considered to be conservative compared with a direct calculation of load response in accordance with the principle outlined in this section.

104 For final documentation of the barge type structure, it is recommended to use the analyses approaches outlined in Sec.5 and Detailed Design in accordance with Sec.6.
SECTION 6
DETAILED DESIGN OF OFFSHORE CONCRETE STRUCTURES

A. General

101 This detailed Standard for design of Offshore Concrete Structures is prepared based on more than 30 years experience with design of Offshore Concrete Structures. These structures can be any type of structure (gravity based and floating) including shell type structures exposed to extreme environmental wave loading.

102 The first DNV Standard for Design of Concrete Offshore Structures for Oil Production Platforms was issued in 1974. This Standard was later updated in 1977 and 1992. The latest issue of Norwegian Standard NS3473 rev. 5 “Concrete Structures – Design Rules” was issued in November 1998.

The DNV Fixed Offshore Rule and NS3473 have with time developed in similar trends. This revision of the DNV Standard is based on both these standards.

103 The experience using the above set of Standards for detailed design of Reinforced and Prestressed Concrete in North Sea Structures has been very good related to strength and durability.

104 Other design standards may as an alternative be used for detailed design of Offshore Concrete Structures due to local preferences. An opening is made for this within this standard provided the requirements to the detailed standard given in Appendix E “Use of Alternative Detailed Design Standard” are sufficiently covered. The level of safety shall be as required by this standard. The compliance with this requirement shall be documented.

A 200 Material

201 The requirements to materials given in Sec.4 shall apply for structures designed in accordance with this section.

202 For definition of Normal Strength Concrete, High Strength Concrete and Lightweight concrete see Sec.4 D200.

203 Prestressed reinforced concrete structures shall not be designed with concrete of Grade less than C35 (see Table C1).

204 For concrete exposed to sea water, the minimum concrete Grade is C45 (see Table C1).

A 300 Load effects

301 Load effects shall be calculated in accordance with the methods outlined in Sec.5. Cracking of the concrete, where that has a significant influence on the load effects, shall be taken into account.

302 In slender structures the effect of the structural displacements shall be accounted for in the calculation of forces and moments.

303 Load effects from imposed deformations shall be considered when relevant. Restraint forces caused by imposed deformations such as support settlements, imposed or restrained axial deformations, rotation etc. shall be considered. When calculating the action effects due to restraint forces, potential cracking may be considered in accordance with O700. In the ultimate limit state the non-linear behaviour of the structure may be considered in the calculation of the effects of imposed strains and deformations.

304 The capacity of a structure may be checked by assuming plastic regions in the calculation of forces and moments. It shall be demonstrated that the necessary displacements are possible in these regions.

305 Moments and shear forces from concentrated loads on slabs can be calculated assuming a load spread of 45° from the loaded surface to the reinforcement on the opposite side of the slab.

306 Calculation of load effects in shear walls and shells may be based on assumptions other than the theory of elasticity if sufficient knowledge is available, on the stress conditions of the actual structure, based on tests or nonlinear calculations. Force models as indicated in Sec.6,1 “Regions with Discontinuity in Geometry and Loads” may be used if relevant models can be established for the structure in question.

307 Unless otherwise documented, pressure from liquids and gases is, in addition to acting on the surface, also be assumed to act internally on the entire cross section or in the cracks, whatever is the most unfavourable

A 400 Effective flange width

401 A cross section subjected to bending with a flange in the compression zone may be assumed to have an effective flange width on each side outside the web equal to the smallest of the following values:

— actual width of flange
— 10% of the distance between the beam's points of zero moment
— 8 times the flange thickness.

402 If the flange has a haunch of width exceeding the height of the flange, the effective flange width may be increased by the height of the haunch, but shall not exceed the actual width of the flange.

403 In a cross section with flange on only one side of the web and not braced laterally, skew bending and torsion shall be considered. Furthermore, effective flange width shall not exceed 7.5% of the distance between the beam points of zero moment.

404 If the flange is located in the tension zone the reinforcement located inside a width as given for a compression zone may be considered fully effective.

405 Values documented by more accurate calculations may be used instead of those given above.

A 500 Composite structures

501 Structures where members of concrete act together with members of structural steel shall be designed in accordance with DNV-OS-C101 or an International standard. The same safety level shall be achieved as in this standard. Concrete members shall be designed in accordance with the regulations of this standard. The general requirements of this standard still apply.

502 In the ultimate limit state, a composite structure of concrete members can be assumed to perform as a monolithic unit for the entire load if the forces can be transferred between the members by reinforcement, shear keys, or by other devices. These forces shall be calculated assuming the structure to behave as a monolithic unit for the entire load, unless more precise calculations are performed. The force in the shear connectors shall be calculated in accordance with an International recognized standard for composite structures.

503 The capacity of the individual structural members shall be checked also for the loads applied on the members before they are acting as a unit.

504 In the serviceability limit state, it shall be considered whether the respective loads are applied before or after the members are acting together.
The design of the composite strength may be designed in accordance with an International standard, i.e. EN standard.

A 600  Prestressed structures with unbonded tendons

601 Structural members exposed to environment class (see Clause O200) SA, MA and NA shall not be prestressed using unbonded tendons.

A 700  Yield line theory

701 Yield line theory may be used as the basis for design in the ULS and ALS conditions provided the following conditions are satisfied:

— The load carrying capacity will be governed by a ductile mode of failure (structural detail has sufficient capacity in shear and moment to accommodate the required rotation).
— Second order effects will be negligible (No buckling mode of failure).
— The plastic hinges along the yield lines will allow sufficient rotation prior to structural failure of the hinge.

Compliance with the above requirements is to be documented.

B. Design Principles

B 100  General

101 Design in compliance with this standard can be based either on calculations or on testing, or a combination of these.

B 200  Limit states

201 Structures shall satisfy the requirements in the following limit states:

— ultimate limit state (ULS)
— accidental limit state (ALS)
— fatigue limit state (FLS)
— serviceability limit state (SLS).

202 In ULS and ALS, the capacity is demonstrated by testing or by calculation based on the strain properties and design material strengths.

203 In FLS, it shall be demonstrated that the structure can sustain the expected load cycles at the applied load levels for the intended service life.

The documentation shall include:

— bending moment
— axial force
— shear force
— torsional moment
— anchorage of reinforcement
— partial loading.

and combinations of these.

204 The design in SLS shall demonstrate that the structure, during its service life, will satisfy the functional requirements related to its use and purpose. Serviceability limit state requirements shall also ensure the durability and strength of the structure.

The documentation shall include:

— cracks
— tightness/leakage
— strains
— displacements
— dynamic effects
— compression zone.

B 300  Characteristic values for material strength

301 The characteristic strength of materials shall be determined according to design standards and recognized standards for material testing (ASTM, ACI, EN, ISO).

302 For concrete, the 28 days characteristic compressive strength , defined as a 5% fractile value (5th percentile) found from statistical analysis of tests on cylindrical specimens with diameter 150 mm and Height 300 mm. Alternatively, the characteristic strength may be determined from tests on standard cubes of dimensions 100x100 mm.

303 In Table C1, a normalized compressive strength, is tabulated based on this characteristic concrete cylinder strength. The normalized compressive strength of concrete, , is less than the characteristic cylinder strength, , and considers transition of test strength into in situ strength, ageing effects due to high-sustained stresses etc.

304 For reinforcement steel, the minimum yield stress shall be taken as characteristic strength , determined as the 5% defectve fractile. The reinforcement shall be of quality in accordance with Sec.4 F Reinforcement.

305 For prestressed reinforcement the in situ strength is taken equal to the yield strength or the 0.2-proof stress. The quality of prestressing steel and anchorage shall be in accordance with Sec.4 G Prestressing Steel.

306 For geotechnical analyses, the characteristic material resistance shall be determined so that the probability of more unfavourable materials occurring in any significant extent is low. Any deteriorating effects during the operation phase shall be taken into consideration. See DNV-OS-C101.

307 For the fatigue limit state FLS, the characteristic material strength shall be determined statistically as a 2.5% fractile for reinforcement, prestressing assemblies, couplers, welded connections, etc. unless other values are specified in the reference standard for that design. For concrete, design material strength shall be used. For soil, the characteristic strength shall be used. For other materials, acceptance criteria shall be specified which offer a safety level equivalent to that of the present provision.

308 Where high resistance of a member is unfavourable (e.g. in weak link considerations), an upper value of the characteristic resistance shall be used in order to give a low probability of failure of the adjoining structure. The upper value shall be chosen with the same level of probability of exceedance as the probability of lower values being underscored. In such cases, the material factor shall be 1.0 in calculating the resistance that is applied as a load on adjoining members.

B 400  Partial safety factors for materials

401 The partial factors for the materials, , in reinforced concrete shall be chosen in accordance with this standard and for the limit state considered.

402 For structural steel members, the material factor shall be in accordance with DNV-OS-C101.

403 Foundation design shall be performed in accordance with DNV-OS-C101 Sec.11. The soil material factors shall also be in accordance with Sec.11.

B 500  Design by testing

501 If the loads acting on a structure, or the resistance of materials or structural members cannot be determined with reasonable accuracy, model tests can be carried out. Reference is made to Sec.5 P.

502 Characteristic resistance of structural details or structural members or parts may be verified by a combination of tests and calculations.

503 A test structure, a test structural detail or a test model shall be sufficiently similar to the installation to be considered.
The results of the test shall provide a basis for a reliable interpretation, in accordance with a recognized standard.

**B 600  Design material strength**

**B 601** The design material strength shall be taken as a normalized in-situ strength in accordance with Table C1 divided by a material coefficient $\gamma_m$.

The design strength in compression, $f_{cd}$, is found by dividing the normalized compressive strength $f_{cn}$ by the material coefficient, $\gamma_c$, in Table B1.

The characteristic tensile strength, $f_{tk}$, and the normalized tensile strength, $f_{tn}$, in the structure are defined in Table C1 and are both derived from the characteristic strength of concrete in compression.

**B 602** In design by testing the requirements given in P500 shall be applied.

**B 603** If a high design strength is unfavourable, a special appraisal of the material coefficients and the nominal value of the in-situ strength, shall be performed. An example is the design of a potential plastic hinge as part of the ductility design of a structure in a seismic active area.

**B 604** The material coefficients, $\gamma_m$, take into account the uncertainties in material strength, execution, cross-sectional dimensions and the theory used for calculation of the capacity. The material coefficients are determined without accounting for reduction of capacity caused by corrosion or mechanical deterioration.

**B 605** Design values for the concrete are:

\[
E_{cd} = \frac{E_{cn}}{\gamma_c}
\]
\[
f_{cd} = \frac{f_{cn}}{\gamma_c}
\]
\[
f_{td} = \frac{f_{tn}}{\gamma_c}
\]

where:

- $E_{cd}$ = Design value of Young’s Modulus of concrete used in the stress-strain curve
- $E_{cn}$ = Normalized value of Young’s Modulus used in the stress-strain curve
- $f_{cd}$ = Design compressive strength of concrete
- $f_{cn}$ = Normalized compressive strength of concrete
- $f_{td}$ = Design strength of concrete in uni-axial tension
- $f_{tn}$ = Normalized tensile strength of concrete
- $\gamma_c$ = Material strength factor concrete

In the ultimate limit state and the accidental limit state, the Young’s Modulus for Concrete, $E_c$, is taken equal to the normalized value, $E_{cn}$.

In the fatigue limit state and the serviceability limit state, the Young’s Modulus for Concrete, $E_c$, is taken equal to the characteristic value, $E_{ck}$.

**B 606** Design values for the reinforcement are:

\[
E_{sd} = \frac{E_{sk}}{\gamma_s}
\]
\[
f_{sd} = \frac{f_{sk}}{\gamma_s}
\]

where:

- $E_{sd}$ = Design value of Young’s Modulus of reinforcement
- $E_{sk}$ = Characteristic value of Young’s Modulus of reinforcement
- $f_{sd}$ = Design strength of reinforcement
- $f_{sk}$ = Characteristic strength of reinforcement
- $\gamma_s$ = Material coefficient Reinforcement

**C 100  Concrete grades and in situ strength of concrete**

**C 101** The characteristic strength of concrete cylinders is defined in B300.

In Table C1, normalized values for in situ strength of concrete are given. The given tensile strength is valid for concrete in uni-axial tension.

Normal weight concrete has grades identified by C and lightweight aggregate concrete Grades are identified by the symbol LC. The grades are defined in Table C1 as a function of the Characteristic Compression cylinder strength of concrete, $f_{cck}$.
102 The strength values given in Table C1 apply to lightweight aggregate concrete with the following limitations and modifications:

\[ f_{ck} \leq f_{ck2}\left(\rho/\rho_1\right)^2 \]

where \( f_{ck2} = 94 \text{ MPa} \) and \( \rho_1 = 2200 \text{ kg/m}^3 \)

Tensile strength, \( f_{tk} \), and in situ strength, \( f_{tn} \), shall be multiplied by the factor \( (0.15 + 0.85 \rho/\rho_1) \)

Where:

\( \rho_1 = 2200 \text{ kg/m}^3 \) if the tensile strength is not determined by testing

\( \rho = \) Density of the lightweight concrete.

For lightweight aggregate concrete with intended concrete strength \( f_{ck} > f_{ck2}\left(\rho/\rho_1\right)^2 \), where \( f_{ck2} = 64 \text{ MPa} \) and \( \rho_1 = 2200 \text{ kg/m}^3 \), it shall be shown by test samples that a characteristic strength, 15% higher than the intended, can be achieved. The test shall be carried out on concrete samples with the same material composition as intended used.

103 For normal density concrete of grade higher than C85 and lightweight aggregate concrete of all grades, it shall be documented by testing that the concrete satisfies the requirements on the characteristic compressive cylinder strength. This also applies if the regular compliance control of the concrete production is performed by testing the compressive cube strength.

104 For concrete at high temperatures for a short period (fire), it may be assumed, provided more accurate values are not known, that the compressive strength reduces linearly from full value at 350°C to zero at 800°C. The tensile strength may be assumed to decrease from full value at 100°C to zero at 800°C. If the concrete is exposed to temperatures above 200°C for a longer period of time, the strength properties of the concrete shall be based on test results.

105 For concrete exposed to temperatures below -60°C, the possible strength increase in compressive and tensile strength may be utilized in design for this conditions provided the strength are determined from relevant tests under same conditions (temperature, humidity) as the concrete in the structure.

An increase in tensile strength of concrete caused by low temperatures will generally tend to increase the distance between the cracks, hence increase the crackswidth.

106 The characteristic tensile strength of the concrete, \( f_{tk} \), may be determined by testing of the splitting tensile strength for cylindrical specimens at 28 days in accordance with ISO 4108. The characteristic tensile strength, \( f_{tk} \), shall be taken as 2/3 of the splitting strength determined by testing.

107 By rehabilitation or by verifying the capacity in structures where the concrete strength is unknown, the strength shall be determined on the basis of drilled core specimens taken from the structure.

The extent of testing shall be chosen so that it gives a satisfactory knowledge of the strengths in the structural members to be examined.

Provided the smallest dimension is not less than 40 mm the following specimen size factor can be used in predicting the cylinder strength.

\[ f_{cck} = 1.2 f_{ckj} - 4 \text{ MPa} \]

if \( f_{ckj} \) is equal to or less than 40 MPa and

\[ f_{cck} = 1.1 f_{ckj} \]

if \( f_{ckj} \) is larger than 40 MPa

where:

\( f_{ckj} \) is the characteristic strength of the taken specimens converted into cylinder strength for cylinders with height/diameter ratio 2:1.

\( f_{cck} \) is the characteristic compressive cylinder strength at 28 days based on in-situ tests.

For design, \( f_{cck} \) replaces, \( f_{ckj} \), the characteristic concrete compressive strength in Table C1

C 200 Reinforcement steel

201 The reinforcement shall be in accordance with Sec.4 F Reinforcement.

202 The in situ structural strength of non-prestressed reinforcement is taken equal to the characteristic yield strength, \( f_{sk} \), determined as the 5% defective fractile.

203 For prestressed reinforcement the in situ strength is taken equal to the yield strength \( f_{sy} \) or the 0.2% proof stress.

### Table C1 Concrete grades and structural strength (MPa).

<table>
<thead>
<tr>
<th>Strength</th>
<th>Concrete grade (MPa) 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic Comp. cube strength, ( f_{ck} )</td>
<td>LC15 LC25 LC35 LC45 LC55 LC65 LC75 LC85 LC95 LC105</td>
</tr>
<tr>
<td>Characteristic Comp. cylinder strength, ( f_{ckt} )</td>
<td>15 25 35 45 55 65 75 85</td>
</tr>
<tr>
<td>Norm. structural comp. strength, ( f_{tn} )</td>
<td>11.2 16.8 22.4 28 33.6 39.2 44.8 50.4 56 61.6</td>
</tr>
<tr>
<td>Characteristic Tensile strength, ( f_{tk} )</td>
<td>1.55 2.10 2.55 2.95 3.30 3.65 4.00 4.30 4.60 4.90</td>
</tr>
<tr>
<td>Norm. structural tensile strength, ( f_{tn} )</td>
<td>1.00 1.40 1.70 2.00 2.25 2.50 2.60 2.70 2.70 2.70</td>
</tr>
</tbody>
</table>

1) Concrete grades are related to the characteristic compressive cylinder strength and is denoted by C for normal dense aggregate concrete and LC for lightweight aggregate concrete. The grades are defined in this Table.

2) The given tensile strength applies to concrete subjected to uni-axial tension.

### Table C2 Scaling factor on drilled core results

<table>
<thead>
<tr>
<th>Height/diameter ratio</th>
<th>2.00</th>
<th>1.75</th>
<th>1.50</th>
<th>1.25</th>
<th>1.10</th>
<th>1.00</th>
<th>0.75</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scaling factor on strength values</td>
<td>1.00</td>
<td>0.97</td>
<td>0.95</td>
<td>0.93</td>
<td>0.89</td>
<td>0.87</td>
<td>0.75</td>
</tr>
</tbody>
</table>

The cylinder strength in the structure is obtained by multiplying the results from drilled cores with the appropriate scaling factor based on the height /diameter ratio of the test specimen.

The concrete is considered to satisfy the requirements to characteristic strength given in Table C1 provided the characteristic value of the cylinder strength in the structure is at least 85% of the required characteristic strength for cylinders for assumed strength class shown in Table C1.

For concrete specimens that have gained at least the 28 days strength, the (equivalent) characteristic cylinder strength, \( f_{cck} \) used in the design may be taken as

\[ f_{cck} = 1.2 f_{ckj} - 4 \text{ MPa} \]

if \( f_{ckj} \) is equal to or less than 40 MPa and

\[ f_{cck} = 1.1 f_{ckj} \]

where:

\( f_{ckj} \) is the characteristic strength of the taken specimens converted into cylinder strength for cylinders with height/diameter ratio 2:1.

\( f_{cck} \) is the characteristic compressive cylinder strength at 28 days based on in-situ tests.

For design, \( f_{cck} \) replaces, \( f_{ckj} \), the characteristic concrete compressive strength in Table C1.
C 300  Concrete stress – strain curves for strength design

301  The shape of the stress/strain relationship for concrete in compression of a specified grade is to be chosen such that it results in prediction of behavioural characteristics in the relevant limit states that are in agreement with results of comprehensive tests. In lieu of such data, the general relationship given in Figure 1 may be used.

![Figure 1](image)

**Figure 1**
General stress-strain diagram for calculation of resistance of normal dense aggregate concrete in compression.

Note: Compression is defined as negative and hence the values of $\varepsilon$ and $\sigma$ are negative for concrete subject to compression.

For $\varepsilon_{cu} < \varepsilon_c \leq \varepsilon_{co}$

then

$$\sigma_c = - f_{cn}$$

For

$$\varepsilon_{co} \geq \varepsilon_c \leq -0.6 \frac{f_{cn}}{E_{cn}}$$

then

$$\sigma_c = E_{cn} \varepsilon_c + (m - 1) f_{cn} \left( \frac{E_{cn} \varepsilon_c + 0.6 f_{cn}}{(0.6 - m) f_{cn}} \right)^{\frac{m - 0.6}{m - 1}}$$

For

$$-0.6 \frac{f_{cn}}{E_{cn}} \leq \varepsilon_c < 0$$

then

$$\varepsilon_{cu} = (2.5 m - 1.5) \varepsilon_{cn}$$

$$\varepsilon_{cn} = - \frac{f_{cn}}{E_{cn}}$$

where:

$$m = \varepsilon_{co} / \varepsilon_{cn}$$

For normal dense aggregate concrete where $f_{ck} \leq 85$ MPa, it may be assumed that

$$E_{cn} = k_E (f_{cn})^{0.3}$$

where $k_E$ is assumed to be equal to 10 000 (MPa)$^{0.7}$.

$$\varepsilon_{co} = \varepsilon_i - k_c f_{cn}$$

where $\varepsilon_i = -1.9$ % and $k_c = 0.004$ %.\%

For concrete grades larger than C65 and for all lightweight aggregate concretes, the values of $E_{cn}$ and $\varepsilon_{co}$ shall be determined by testing of the type of concrete in question. Concrete subject to tensile strains is to be assumed stressless if not otherwise stated.

302  For normal dense concrete of grades between C25 and C55, the following simplified stress/strain diagram may be used.

![Figure 2](image)

**Figure 2**
Simplified stress-strain diagram for normal density concrete of grades between C25 and C55 subject to compression.

$$\sigma_c = \varepsilon_c \frac{2 - \varepsilon_c}{\varepsilon_{co}}$$

$\varepsilon_{co} = -2$ % is strain at the point of maximum stress.

303  For lightweight aggregate concrete of grades between LC15 and LC45, a simplified bilinear stress – strain diagram may be applied for calculation of capacities. The maximum strain limit for LWA concrete in compression is

$$\varepsilon_{cu} = \varepsilon_i \left(0.3 + \frac{0.7 \rho}{\rho_1}\right)$$

where $\varepsilon_i = -3.5$ %, $\rho_1 = 2400$ kg/m$^3$ and $\rho$ = density of LWA.

![Figure 3](image)

**Figure 3**
Simplified stress-strain diagram for lightweight aggregate concrete of grades LC15 to LC45 subject to compression.

304  For calculation of capacities for axial forces and bending moments, different stress distributions from those given herein (C301, C302, C303) may be applied as long as they do not result in a higher sectional capacity.

305  For reinforcement, a relationship between force and strain which is representative for the type and make in question shall be used.
The stress-strain diagram for design is found by dividing the steel stress $\sigma_s$ by the material coefficient $\gamma_s$.

Any deviating properties in compression shall be considered.

306 Where the assumed composite action with the concrete does not impose stricter limitations, the strain in the reinforcement shall be limited to 10‰. For prestressed reinforcement, the prestressing strain is added to this limit.

307 For reinforcement in accordance with Sec.4, the steel stress may be assumed to increase linearly from 0 to $f_{sd}$ when the strain increases from 0 to $\varepsilon_{sy} = f_{sk}/E_{sk}$.

The reinforcement stress may be assumed to be equal to $f_{sd}$ when the strain varies between $\varepsilon_{sy}$ and $\varepsilon_{su}$. The strain $\varepsilon_{su}$ shall not exceed 10 ‰.

The steel can be assumed to have the same strain properties and yield stress in both compression and tension.

308 For temperatures above 150°C, the stress-strain diagram for ribbed bars in accordance with Sec.4 can be assumed to be in accordance with Figure 5.

![Stress-strain diagram for reinforcement in accordance with Sec.4](image)

Figure 4
Stress-strain diagram for reinforcement in accordance with Sec.4

The diagram in Figure 5 does not include thermal strain or creep strain caused by high temperature.

309 For reinforcement exposed to low temperature shall remain ductile under the applicable temperature range.

310 For spiral reinforcement in columns, shear reinforcement, torsional reinforcement, and reinforcement in construction joints, calculated in accordance with the methods in Sub- Secs. D, F, G and J. The utilized stress shall not exceed the stress corresponding to 2.5 % strain. For prestressed reinforcement, the prestressed strain is added.

C 400 Geometrical dimensions in the calculation of sectional capacities

401 When allowing larger deviations in dimensions than those specified in Table C3, the deviations in sectional dimensions and reinforcement position shall be considered in the design. Smaller deviations than the specified tolerances may be considered.

### Table C3 Acceptable Deviations

<table>
<thead>
<tr>
<th>Type of Dimensional Deviation</th>
<th>Maximum Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall dimension</td>
<td>± 25 mm</td>
</tr>
<tr>
<td>Cross-sectional</td>
<td>± 8%</td>
</tr>
<tr>
<td>Perpendicularity</td>
<td>8 ‰</td>
</tr>
<tr>
<td>Inclination</td>
<td>3 ‰</td>
</tr>
<tr>
<td>Local variations (1 m)</td>
<td>8 mm</td>
</tr>
<tr>
<td>Local variations (2 m)</td>
<td>12 mm</td>
</tr>
</tbody>
</table>

For structures of special shapes and geometry alternative tolerances may be specified from a strength point of view provided the capacity calculated based on the specified tolerances does not reduce the capacity with more than 10%.

402 If the most unfavourable combinations of specified tolerances for sectional dimensions and reinforcement positions are considered and conformity control verifies that the actual deviations do exceed those specified, then the increased material coefficients in accordance with Table C1 shall be used.
should be re-evaluated in all relevant limit states.

403 For structures cast under water, the outer 100 mm of concrete at horizontal construction joints and in the contact area between the ground and the concrete shall not be taken into account as effective cross section for transfer of forces. If the structure is set at least 100 mm into rock, the entire concrete section can be calculated as effective for transfer of forces to the ground.

C 500 Tension in structural members

501 Tensile forces shall be provided for by reinforcement with the following exceptions:
- tension caused by shear force, anchorage or splicing of reinforcement, and by partially loaded areas, which may be assumed transferred by the concrete design in accordance with this standard.

C 600 Fatigue strength relationships

601 Fatigue strength relationships (S-N curves) for concrete are to take into account all relevant parameters, such as:
- concrete quality
- predominant load effect (axial, flexural, shear, bond or appropriate combinations of these)
- state of stress (cycling in pure compression or compression/tension)
- surrounding environment (air, submerged).

C 700 Deformation induced loads, prestressing and creep

701 Deformation induced loads created by imposed deformations in the structure, are loads to be treated as deformation loads (D), and not as a force which requires equilibrium.

702 In case of a ductile mode of failure, and where second order effects are negligible, the effect of deformation loads may normally be neglected.

A typical example of a ductile mode of failure is a flexural failure provided sufficient rotational capacity exists. Verification of sufficient rotational capacity may in most cases be based on simplified considerations.

703 Imposed deformations normally have a significant influence on the shear resistance of a section, and shall be duly considered in the design.

704 Calculation of deformation loads.

The characteristic value of deformation imposed loads is normally evaluated on the basis of defined maximum and minimum values for the parameters governing its magnitude.

An accurate calculation of deformation loads caused by temperature effects can only be obtained from a non-linear analysis, reflecting realistic material properties of reinforced concrete. In practice, 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.

The temperature expansion coefficient (α) may be taken as given in Sec. 5 D300.

For concrete exposed to low temperatures the temperature expansion coefficient (α), shall be determined by relevant tests of the material.

705 Creep effects shall be considered where relevant. Effect of creep

An accurate calculated assessment of creep in shell structures can only be obtained by computer calculations using non-linear finite-element programs. However, rough estimates of creep effects may be obtained by methods originally developed for simple columns. In the following two methods are referred to, the so-called “creep factor method” and the “creep eccentricity method”.

Guidance note:
“Creep factor method” The method utilizes a modified stress/strain diagram for concrete. In this diagram, the short term strains are multiplied by (1 + ϕ), ϕ being the creep factor, see Figure 6 and Figure 1.

The values of ϕ shall be carefully determined in accordance with recognized principles. The creep factor, ϕ, is to be determined for relevant temperature range, concrete grade and humidity.

“Creep eccentricity method”. In this method, the effect of creep is accounted for by introducing an additional eccentricity caused by creep. The method is convenient to use. Two important conditions with respect to application of the method, shall be noted:
- The value of the load causing creep is to be small enough so to avoid non-linear material behaviour under short term loading.

C 800 Effect of water pressure

801 The effect of water pressure in the concrete is to be fully considered when relevant.

802 The effect of hydrostatic pressure on the concrete strength is to be evaluated where relevant. (For lightweight aggregate concrete, this effect may be significant.)

803 The effect of hydrostatic forces acting on the faces of cracks is to be taken into account in the designs for ULS, SLS, ALS and FLS. Reference is made to Sec. 5 D406.

D. Bending Moment and Axial Force (ULS)

D 100 General

101 The capacity for bending moment and axial force can be determined by assuming that plane cross sections remain plane after straining, and that the stress and strain properties of the concrete and the reinforcement are as given in C.

When load effects are determined by applying plastic design analysis techniques. Such structures shall be composed of members which are able to develop well-defined plastic resistances and maintain these resistances during the deformation necessary to form a mechanism. The plastic resistances shall be adequately documented. See A700.
102. The average calculated compressive strain over the cross section shall not exceed \( \left( \varepsilon_{cc} + \varepsilon_{cu} \right) / 2 \). Strain caused by shrinkage and linear creep shall be added and the total strain shall be within the above limit.

103. When calculating the capacity of a cross section, resulting from an external axial load, the axial load shall be assumed to have a minimum eccentricity about the most unfavourable principle axis. The eccentricity shall not be taken less than the largest of 20 mm or 1/30 of the cross-sectional dimension in the direction of the eccentricity.

The requirements given in this sub-section are in general applicable to structural members where the ratio between the depth, \( h \), of the member and the distance between the points of zero bending moment is less than 0.5. If this ratio is greater than 0.5, assumptions relevant to other types of structural members such as deep beams, corbels etc. shall be applied.

104. If the area of compressive reinforcement exceeds 4% of the concrete area, the capacity calculation shall be based on the net area of concrete.

The net area of concrete is defined as the concrete area between the centroid of the reinforcement on “tensile” and “compression” side of the member. For members reinforced using bundled bars the centroid refers to the centroid of the bundle. For members with several layers of reinforcement, the centroid refers to the outer bar on the “tensile” and “compression” side.

105. In axially loaded structures such as columns and walls, the reinforcement shall only be considered effective in compression if sufficiently secured against buckling. The compressive reinforcement shall be braced, by crossing bars placed on the exterior side, unless otherwise is shown to be sufficient.

106. For columns with spiral reinforcement as described in R409 and with normal weight concrete of grades no higher than C55, the sectional resistance capacity can be calculated in accordance with this clause.

The axial capacity shall be calculated using an effective cross section, defined as the concrete core inside the centroid of the spiral reinforcement plus the equivalent concrete cross section of the longitudinal reinforcement based on modular ratios of concrete and reinforcement. For eccentricities less than 0.25\( D_k \), an increased compressive design strength of the concrete can be assumed equal to

\[
f_{cd} + 6 \cdot (f_{sd} \cdot A_{ss}) \left( 1 - 4 \cdot e / D_k \right) / D_k \cdot s
\]

where:
- \( s \) is the centre to centre distance between the spiral reinforcement, measured in the longitudinal direction of the column
- \( D_k \) is the diameter of the concrete core inside the centroid of the spiral reinforcement, \( A_{ss} \)
- \( f_{sd} \) is the design strength of the spiral reinforcement, \( A_{ss} \)
- \( e \) is the eccentricity of loading.

The strains \( \varepsilon_{cc} \) and \( \varepsilon_{cu} \) shall be assumed to increase at the same ratio as the design strength.

The capacity shall neither be taken as less than the capacity of the full cross section including the longitudinal reinforcement without adding for the effect of the spiral reinforcement, nor more than 1.5 times this capacity.

107. The capacity of an unreinforced cross section shall be determined with the concrete stress-strain relationship given in C300 assuming the concrete not to take tension.

The eccentricity shall not be larger than to give a compressive zone of at least a half of the cross sectional depth.

108. The tensile strength for fibre reinforced concrete containing at least 1 volume per cent steel fibre can be taken as \( k_{sw} f_{cd} \). For design of cross sections subjected to axial tension, the factor, \( k_{sw} \), shall be taken as 1.0, when designing for bending moment or bending moment in combination with axial compression the factor, \( k_{sw} \), shall be set at 1.5 – h/h, but no less than 1.0.

\[
h = \text{the cross-sectional height, and } h_1 = 1.0 \text{ m.}
\]

D 200  Ductility

201 Concrete structures shall be designed to have a ductile behaviour.

Ductile behaviour of concrete structures is required in order to ensure that the structure, to some extent, can withstand abnormal or accidental loads and that a redistribution of the loads can take place.

202 For members subjected to moment loading, it is to be ensured that the steel on the tension side of the member will achieve twice the yield strain before the concrete on the compressive side reaches its ultimate compressive strain.

Where this requirement cannot be fulfilled due to high axial loads, ductility is to be ensured by use of stirrups.

E. Slender Structural Members

E 100  General

101 For structural instability a simplified method of analysis will, in general, be considered acceptable if it can be adequately documented that, for the relevant deformation, the design loading effects will not exceed the corresponding design resistances for structural instability. General non-linear analyses are described in Sec.4 D1200.

Slender structural members subjected to axial compression or bending moment in combination with axial compression shall be dimensioned for these action effects and the effect of displacements of the structure (second order theory). The effect of concrete creep shall be accounted for if it has an unfavourable influence on the capacity.

102 Displacements caused by short-term actions shall be calculated in accordance with the stress-strain curve given in C.

103 The effect of creep shall be calculated in accordance with the history of actions on the structure and characteristic actions. See also Sec.5 D300.

104 A structural member shall be assumed as slender if, in accordance with E109-E112, the effect of displacements cannot be ignored.

Where second order effects may be significant such effects shall be fully considered. The design of neighbouring elements is to take into account possible second order effects transmitted at the connections.

105 Structures structurally connected with slender compressive members shall be designed for forces and bending moments in accordance with the assumed degree of restraint and the additional moments caused by the displacements in the connecting members.

The stiffness assumptions for the individual structural members shall be in accordance with the design action effects and the corresponding state of strain.

No less reinforcement shall be provided in any structural member than what was assumed when calculating the displacements.

106 The compressive force in slender compression members shall be assumed to have an unintended eccentricity calculated in accordance with specified tolerances for curvature and inclination for the individual members.

107 The eccentricity shall not be assumed to be less than the largest of 20 mm, \( L/300 \) or 1/30 of the cross sectional dimension in the direction of eccentricity, unless special conditions
provide basis for other values. The buckling length, $l_c$, is the length of a pin connected strut with the same theoretical buckling force (Euler-force) and direction of displacement as the structural member in question.

108 The unintended eccentricity shall be assumed to act along that principal axis of the cross section where the effect will be most unfavourable, considering simultaneously the effect of first and second order bending moments.

109 The force dependent slenderness in the direction with the smallest resistance against buckling shall normally not be greater than 45.

110 The geometrical slenderness $\lambda$ shall normally not exceed 80 $(1+4\omega_i)^{0.5}$.

Where:

$$\omega_i = \sum (f_\text{cd} \cdot A_i)/(f_\text{cd} \cdot A_c)$$

$A_s$ = the area of reinforcement

$A_c$ = the cross-sectional area of uncracked concrete.

The force dependent slenderness, $\lambda_N$, of a structural member is calculated from the equation

$$\lambda_N = \lambda \cdot (\frac{N_f}{(1+4\omega_i)})^{0.5}$$

where:

$$\lambda = \frac{l_c}{i}, i = (l_c/A_c)^{0.5}$$

$I_c$ = the moment of inertia of $A_c$

$n_f = \frac{N_f}{f_\text{cd} \cdot A_c}$

$N_f$ = design axial force

$l_c$ = effective length, theoretical buckling length.

The reinforcement area $A_s$ is introduced with its full value for rectangular sections with reinforcement in the corners, or with the reinforcement distributed along the faces perpendicular to the direction of the displacement. For other shapes of cross-sections or other reinforcement positions, the reinforcement area can be entered as two thirds of the total reinforcement area if more accurate values are not used.

111 The effect of displacements may be neglected if the force dependent slenderness $\lambda_N$ based on the design actions is less than 10.

112 For a structural member with braced ends, without lateral forces, this limit may be increased to $\lambda_N = 18 - 8 \mid \text{M}_{OA} \mid / \mid \text{M}_{OB} \mid$

where:

$\mid \text{M}_{OA} \mid$ = Numerical smallest member end moment calculated from 1. order theory

$\mid \text{M}_{OB} \mid$ = Numerical largest member end moment calculated from 1. order theory

113 If the force dependent slenderness calculated with axial forces based on the characteristic long-term force for the structure and the corresponding end moments does not exceed the values given in E110 the effect of creep may be ignored.

114 Beams and columns in which, due to the slenderness, considerable additional forces may occur due to torsional displacements of the structural member (lateral buckling or torsional buckling), shall be designed accordingly.

115 When designing thin-walled structures, consideration shall be made to local placements where this will influence the design action effects. The calculation shall be based on approved methods and the principles given in E101 through E109 where these apply.

116 If vital parts of the structure are in flexural or axial tension, and redistribution of forces due to cracking is expected, detailed non-linear (geometrical and material non-linearities) analyses of the reinforced concrete may be required.

F. Shear Forces in Beams and Slabs

F 100 Basis

101 The rules in this sub-section apply to beams, slabs and members where the ratio between span length and depth is at least 3.0 for two-sided supports and at least 1.5 for cantilevers. Structural members having a smaller ratio between length and depth shall be designed in accordance with Sub-sec. I “Regions with Discontinuity in Geometry or Loads”.

102 The capacity with respect to tensile failure ($V_{cd}+V_{sd}$) and compressive failure ($V_{cd}$) shall be checked. The calculation may be performed in accordance with the simplified methods in F200 truss model method in F300 or the general method given in Sub-sec. H.

103 In the case of haunches or prestressed reinforcement that are inclined compared to the longitudinal axis of the structural member, the component of forces perpendicular to the longitudinal axis shall be added to the design shear forces from the actions. If forces or support reactions are applied to the structural member in such a manner that internal tensile forces are imposed in the direction of the force, these internal forces shall be transferred by reinforcement.

104 In support regions an internal force system shall be chosen in accordance with Sub-sec. I.

Tensile failure capacity for direct force applied within a distance $c > 2d$ from the face of the support may, as a simplification be checked by demonstrating that the cross section has sufficient capacity for a part of the load equal to the load multiplied by the factor $a/2d$ when determining the shear force.

Where:

$a = \text{distance from the face of the support}$

$d = \text{distance from the centroid of the tensile reinforcement to outer edge of the compression zone}$

For distributed actions which are nearly uniform the value of the shear force at the distance $d$ from the face of support may, as a simplification, be used to check the capacity for tensile failure in cross sections closer to the support.

The capacity for compressive failure shall be verified at the face of the support for the entire shear force.

105 Shear reinforcement shall be included in the calculations of the capacity only if the provided reinforcement is at least as given in R306 and shall consist of stirrups or bent bars. In beams at least half of the shear capacity to be provided by shear reinforcement shall be stirrups.

The spacing between the stirrups measured along the longitudinal axis shall not be more than 0.6·$h'(1 + \cot \phi) \leq h'$ and not more than 500 mm, see R306. Only shear reinforcement of an angle between 45 and 90 degrees with the longitudinal axis shall be included in the calculations. Inclined shear reinforce-
ment shall be slanted to the same side of the cross section as the principal tensile stresses. The spacing between the stirrups shall neither exceed 0.4 h’(1 + cot α) nor 0.7 h’ if the shear force is greater than 2·fcd·bw·d or if in combination with shear force there is significant axial tension or if the action has fatigue effect. Perpendicular to the span direction of the structural member, the spacing shall neither exceed the depth of the beam nor be more than 600 mm.

Where:

- \( \alpha \) = the angle between shear reinforcement and the longitudinal axis
- \( h’ \) = the distance between the centroid of the reinforcement on the “tensile” and “compression” side of the member.

**106** For slabs, the capacity in any direction shall at least be equal to the design shear force for this direction. If the capacity is not sufficient without shear reinforcement, the area of shear reinforcement for the direction that has the greatest requirement shall be provided.

If the action is transferred to the supports primarily in one direction, it is sufficient to check the shear capacity for this direction.

If the slab is not subjected to in-plane membrane forces, the slab can be designed for the principal shear force at the considered position.

**107** A beam flange subjected to shear forces in its plane can be designed in accordance with the rules for combined action effects in H or I.

**F 200 Simplified method**

**201** For a structural member without shear reinforcement, the shear capacity at tensile failure can be taken as Vcd. The capacity for shear force without a coinciding axial force can be taken as

\[
V_{cd} = V_{co} = 0.33 \left( f_{id} + k_{A} A_{s} \right) / (\gamma c bw d) bw d k_{v} \leq 0.66 f_{id} bw d
\]

where:

- \( k_{A} = 100 \text{ MPa} \)
- \( A_{s} = \text{the cross section area of properly anchored reinforcement on the tension side (mm}^{2}\)\)
- \( bw = \text{width of beam (mm)} \)
- \( d = \text{distance from centroid of tensile reinforcement to compression edge (mm)} \)
- \( k_{v} = \text{For slabs and beams without shear reinforcement the factor } k_{v} \text{ is set equal to 1.5 - d/d}_{1}, \text{ but not greater than 1.4 or less than 1.0} \)
- \( d_{1} = 1 \text{ 000 mm.} \)

**202** The capacity at tensile failure for shear force in combination with axial compression may be taken as

\[
V_{cd} = V_{co} + 0.8 \cdot M_{0} \cdot V_{f} / \gamma l \leq (f_{id} k_{v} - 0.25 \cdot N_{f} / A_{c}) \cdot bw \cdot z_{1}
\]

where:

- \( M_{0} = -N_{f} / A_{c} \)
- \( N_{f} = \text{axial design load, positive as tension} \)
- \( V_{f} = \text{design shear force for the cross section under the considered condition} \)
- \( M_{f} = \text{total moment in the section acting in combination with the shear force } V_{f} \)
- \( N_{f} / A_{c} \) shall not be taken with a greater numerical value than 0.4 fcd
- \( W_{c} = \text{the section modulus of the concrete cross section with respect to the extreme tension fibre or the fibre with least compression} \)
- \( I_{c} = \text{the moment of inertia for the uncracked concrete section} \)

\[ S_{c} = \text{area moment about the centroid axis of the cross-section for one part of the concrete section} \]

\[ z_{1} = \text{the greater of 0.7 d and } I_{c} / S_{c} \]

\[ b_{w} = \text{width of beam web (mm)} \]

**203** The capacity for shear force with coinciding axial tension can be taken as

\[ V_{cd} = V_{co} (1- N_{f} / (1.5 \cdot f_{cd} A_{c})) \geq 0 \]

**204** In cross sections where there is a bending moment coinciding with the axial force, the capacity can be taken as

\[ V_{cd} = V_{co} (1 - M_{f} / | M_{f} | ) \geq 0 \]

if this gives greater capacity than the formula above.

Here

\[ M_{o} = N_{f} / W_{c} / A_{c} \]

\( W_{c} \) is the section modulus with respect to the extreme fibre having compression

When calculating \( V_{co} \) the part of the tension reinforcement required to transfer only the tensile force shall be neglected.

**205** The longitudinal reinforcement shall also be designed for any forces from imposed strains in addition to other action effects, and so that no part of the longitudinal reinforcement will have a greater calculated strain than \( \varepsilon_{sy} \).

**206** The capacity for structural members with transverse reinforcement (shear reinforcement) that is distributed along the longitudinal direction, may be assumed equal to the resistance \( V_{cd} \) plus an additional \( V_{sd} \) from the transverse reinforcement. When calculating \( V_{cd} \), \( k_{v} \) shall be set equal to 1.0.

**207** The capacity portion \( V_{sd} \) is determined by the force component in the direction of the shear force from transverse reinforcement crossing an assumed inclined crack at 45 degrees to the longitudinal axis of the structural member within a depth equal to \( z \) from the tension reinforcement

\[ V_{sd} = \sum \left( f_{sd} \cdot A_{SV} \cdot \sin \alpha \right) \]

For transverse reinforcement consisting of units with spacing \( s \) measured along the longitudinal axis, this becomes:

\[ V_{sd} = (f_{id} \cdot A_{SV} \cdot z) / (1 + \cot \alpha) \cdot \sin \alpha \]

\( z \) can be taken equal to 0.9 d if the cross section has a compressive zone. If the entire cross section has tensile strain, \( z \) shall be taken equal to the distance \( h’ \) between the utilized longitudinal reinforcement groups (centroid) on the upper and lower side relative to the plane of bending.

**208** The capacity for compression failure shall be taken as

\[ V_{cd} = \frac{0.30}{\gamma f_{cd}} \cdot f_{w} \cdot z \cdot (1 + \cot \alpha) \leq 0.45 \cdot f_{cd} \cdot b_{w} \cdot z \]

**F 300 Truss model method**

**301** The capacity for shear force only or in combination with other action effects can be calculated based on an assumed internal truss model with compressive concrete struts at an angle, \( \theta \), to the longitudinal axis of the beam. The shear reinforcement acts as tension ties, and the tensile and the compressive zone as chords in this assumed truss. A capacity portion \( V_{cd} \) in accordance with F200 shall not be included in the capacity.

**302** For members subjected to shear force not in combination with axial compression, the angle \( \theta \) shall be chosen between 25°and 60°.

**303** For members subjected to shear force with axial compression, the angle \( \theta \) may be chosen less than 25 degrees, but not less than that corresponding to the direction of the principal compression calculated for uncracked concrete.

**304** For members subjected to shear force in combination with not negligible axial tension, the angle shall normally be taken as \( \theta = 45° \).

**305** The shear capacity at tensile failure shall be calculated
from the force component in the direction of the shear force from the transverse reinforcement $A_{SV}$ crossing an assumed crack at an angle $\theta$ to the longitudinal axis for the structural member within a depth equal to $z$ from the tensile reinforcement:

$$V_{sd} = \Sigma f_{sd} \cdot A_{SV} \cdot \sin \alpha$$

where:

$\alpha$ is the angle between the transverse reinforcement and the longitudinal axis  
$\theta$ is the angle between the inclined concrete compression struts and the longitudinal axis

**306** For transverse reinforcement consisting of units with a spacing $s$ measured along the longitudinal axis, the shear capacity becomes

$$V_{sd} = (f_{sd} \cdot A_{SV} \cdot z/s) (\cot \theta + \cot \alpha) \sin \alpha$$

**307** The shear reinforcement for the most unfavourable load case may be designed for the smallest shear force within a length $z \cot \theta$, corresponding to projection of the inclined crack, measured along the longitudinal axis.

**308** The capacity at compression failure shall be taken as

$$V_{ced} = f_{cd} \cdot b_y \cdot z \frac{(\cot \theta + \cot \alpha)}{(1 + \cot^2 \theta)}$$

The design compressive strength, $f_{cd}$, in the compression field shall be determined for the calculated state of strain in accordance with $H$. When $\theta$ is assumed between 30 and 60 degrees, the design compressive strength can be assumed as

$$f_{cd} = 0.6 \cdot f_{cd}$$

**F 400 Additional force in the longitudinal reinforcement from shear force**

**401** When calculating according to the simplified method, the longitudinal reinforcement shall be designed for an additional tensile load, $F_{SV}$ caused by the shear force

$$F_{SV} = V_f$$

in structures without shear reinforcement

$$F_{SV} = V_f - 0.5 \cdot V_{sd} \cdot (1 + \cot \alpha) \geq 0$$

in structures with shear reinforcement

Where:

$V_f$ = Applied design shear force  
$V_{sd}$ = Shear carried by shear reinforcement (See F306)

The force $F_{SV}$ shall be assumed to act in both chords if this is unfavourable, i.e. areas near points with zero moment.

**402** When calculating according to the truss model method, a tensile force, $F_{SV}$ shall be assumed on both sides of the cross section:

$$F_{SV} = 0.5 \cdot V_f \cdot (\cot \theta - \cot \alpha) \geq 0$$

**403** The maximum force in the longitudinal reinforcement on the tension side shall not be taken at greater value than the value corresponding to the highest absolute moment in combination with the axial force found on the same part of the moment curve as the section examined.

**F 500 Slabs subjected to concentrated actions**

**501** The design of slabs subjected to concentrated actions causing compression perpendicular to the middle plane of the slab, i.e. column reactions or wheel actions, may be carried out in accordance with this section.

**502** The calculation can normally be based on a rectangular loaded area with equal area and equal ratio between the dimensions in the two main directions as the actual loaded area.

**503** The capacity at tensile failure for a concentrated action in the inner parts of a slab is determined based on an assumed governing rectangular section with boundaries at a distance 1.0 · $d$ from the loaded area.

The governing section shall be chosen in such a way that

- an area containing the loaded area is separated by the governing section from the remainder of the slab
- the governing section at no location is closer to the loaded area than 1.0·$d$
- the perimeter of the governing section shall be minimized, but straight edges may be assumed i.e. corners are not rounded, see Figure 8.

![Figure 7](image_url)

**Figure 7** Cross-section for Design check of shear capacity for concentrated load on Plates

![Figure 8](image_url)

**Figure 8** Cross-section for Design check of plates with columns at the corner

**504** For concentrated mobile load near supports, the governing action position will be such that the distance from the boundary of loaded area to the face of the support is equal to 2·$d$.

**505** When a concentrated load is applied in the vicinity of a free edge, in addition to the section given in F502, a governing section shall be assumed extending to the free edge and perpendicular to this, see Figure 7.

**506** Similar rules apply to corners of slabs, see Figure 8a. In this case the capacity shall also be checked for a section at a distance $d$ from the inner corner of the action. The section shall be assumed in the most unfavourable direction and in such a way that it separates the corner and the action from the remainder of the slab, see Figure 8b.

**507** Where the distance between the outline of an opening in the slab and the outline of the loaded area or column is less than or equal to 5 · $d$, the portion of the governing section located between two tangents to the outline of the opening, starting from the centre of gravity of the loaded area, shall be neglected when calculating the shear capacity, see Figure 9.
The distribution of shear forces along the critical section can be calculated in accordance with the theory for elastic plates.

In a simplified approach, a linear distribution of shear force along each of the faces of the governing section is usually assumed. A portion of the eccentricity moment caused by a moment introduced from a supporting column, an eccentrically located section enclosing a load at a free edge or similar shall be assumed to be balanced by a linear variation of the shear force in the critical section.

For a rectangular section, this portion of the moment can be taken as

\[ M_T / (1 + b_y / b_x) \]

Here, \( b_x \) is the length of the side of the critical section that is parallel to the moment axis and \( b_y \) is the side perpendicular to this. For other forms of the governing section, the portion of the moment is determined as for a rectangular section with equal area and equal side ratio.

The portion of the introduced moment that is assumed not to be introduced by a variation of the shear force shall be transferred by bending moments or torsional moments along the sides of the governing section.

The capacity \( V_{cd} \) per unit width of the governing section at tensile shear failure for a slab without shear reinforcement shall be determined in accordance with F201. The depth \( d \) is taken as the average, \( d = (d_x + d_y) / 2 \), where \( x \) and \( y \) refer to the reinforcement directions. For the reinforcement ratio, \( \rho_x = A_{svx} / (b \cdot d) \) and \( \rho_y = A_{svy} / (b \cdot d) \), the geometrical mean for the two directions of tension reinforcement shall be introduced: \( A_{sx} \) and \( A_{sy} \) are the amount of reinforcement in \( x \)-and \( y \)-direction, respectively.

\[ \rho = (\rho_x \cdot \rho_y)^{0.5} \]

The reinforcement ratios shall be determined as average values over a width of \( 2d \) to each side of the loaded area. The capacity shall be reduced in accordance with the regulations in F203, if the slab is subjected to axial tension.

The capacity shall be verified for the remaining loading conditions, including shear force in plane sections outside the governing section, according to F200.

If the shear capacity of a slab without shear reinforcement calculated in accordance with F501 – F512 is less than the calculated action effect, shear reinforcement shall be provided in areas where the shear capacity is insufficient.

The capacity at tensile shear failure per unit width of the governing section for slabs with shear reinforcement shall be taken equal to the sum of the capacity, \( V_{cd} \) calculated using \( k = 1.0 \) plus a contribution from the shear reinforcement given by:

\[ V_{sd} = \sum f_{sd} \cdot A_{sv} \cdot \sin \alpha \]

\( V_{sd} \) shall at least be equal to 0.75 \( V_{cd} \).

Outside the section 1.0 \( d \) from the face, the required shear reinforcement shall be calculated for plane sections in accordance with F206, F207 and be distributed in accordance with F105. The distance between the reinforcement units in the direction perpendicular to the governing section can be up to 0.75 \( d \) in the span direction.

The shear reinforcement in the area of concentrated actions may consist of stirrups, possibly combined with bent bars. Other types of steel reinforcement may be added provided the structural performance is verified by available documentation.

Compression failure caused by shear force shall be considered in accordance with F208 for sections at the face of the loaded area.

### G. Torsional Moments in Beams

#### G 100 General

The capacity for torsional moment shall be checked for tensile and compression failure. If the load transfer in the ultimate limit state is not dependent on the torsional capacity, the design can normally be performed without considering torsional moments.

The torsional capacity of the cross section shall be calculated based on an assumed closed hollow section with an outer boundary coinciding with the actual perimeter of the cross section. The wall thickness of the effective cross section shall be determined as the required thickness using a design compressive concrete stress limited to \( f_{cd} \), where \( f_{cd} \) equals the reduced design compression strength under biaxial tensile stress. However, for pure torsion the assumed wall thickness shall be limited to 0.2 multiplied by the diameter of the largest circle which can be drawn within the cross-section, and maximum equal to the actual wall thickness for real hollow sections. Concrete outside the outer stirrup shall not be included in the design if the distance from the centre line of the stirrup to the face of the concrete exceeds half of the assumed wall thickness, or if the total inclined compressive stress, from torsional moment and shear force exceeds 0.4 \( f_{cd} \). The concrete outside the stirrups shall always be neglected if the concrete surface is convex.

The individual cross-sectional parts can be designed for the calculated shear forces in accordance with the general method in H or in accordance with the requirements of G104 to G107.

Internal forces shall be determined in accordance with recognized methods based on the equilibrium requirements under the assumption that the concrete cannot carry tension. Where tensile strain occurs in the concrete, the forces shall be calculated as for a space truss model at the middle surface of the assumed walls. In this truss, all tensions shall be transferred by reinforcement while the concrete can transfer compression.

Compressive failure limits the torsional capacity of the cross section. The capacity at compressive failure for only torsional moment is the value giving a compressive concrete stress equal to \( f_{cd} \) according to H106-H107. The compressive stress is calculated for the assumed hollow section for the same equilibrium state as the one used to design the governing torsional reinforcement.

For torsional moment in combination with shear force or axial force, the capacity for compressive failure shall be determined...
by taking the maximum compressive concrete stress in the effective cross section as $f_{cd}$.

106 The capacity at tensile failure shall be determined by the maximum tensile forces that the torsional reinforcement can transfer in the assumed spacial truss. The design may be based on a consideration of shear walls. It shall be demonstrated that the corresponding internal forces in the corners can be transferred.

107 For torsional moment in combination with bending moment, axial force or shear force, the required reinforcement may be calculated as the sum of required reinforcement due to torsional moment and due to the other action effects.

108 Torsional reinforcement shall be provided as closed stirrups with proper anchorages. In structures or structural members which according to these regulations shall be designed for torsion, this stirrup reinforcement in each face shall have a minimum cross section of

$$0.25 A_e f_{tk}/f_{sk}$$

where $A_e$ is the concrete area of a longitudinal section calculated using the minimum wall thickness of a hollow section, or 0.2 multiplied by the diameter of an enclosed circle in accordance with G102 – G103 for a solid cross section. The tensile strength, $f_{tk}$, shall not be entered with a lesser value than 2.55 MPa.

109 If the load transfer is totally dependent on the capacity of the stirrup spacing between the stirrups shall not exceed 300 mm. If in addition the design torsional moment exceeds half of the capacity of the cross section calculated at compressive failure, the link spacing shall be less than 300 mm and at fully utilized concrete section not exceed 150 mm.

110 In addition to stirrup reinforcement, the torsional reinforcement shall consist of a longitudinal reinforcement, either nearly uniformly distributed or concentrated in the corners. The spacing shall not exceed that given for stirrups, and the longitudinal reinforcement shall have a cross-sectional area per unit length along the perimeter of the stirrup at least equal to the minimum area required per unit length for stirrups.

111 The longitudinal reinforcement may be less than this provided axial compression is acting simultaneously or the stirrup reinforcement is placed nearly parallel to the principal tensile stress direction, and provided that the capacity is sufficient. At least one bar shall be provided in each corner of the stirrups and having at least the same diameter as the stirrups.

112 Torsional reinforcement, both stirrups and longitudinal reinforcement, shall be distributed in the cross section in such a way that all cross-sectional parts get at least the required minimum reinforcement.

### H. General Design Method for Structural Members Subjected to In-plane Forces

#### H 100 General

101 Design for forces acting in the middle plane of a structural member may be performed by a method based on an assumed internal force model satisfying equilibrium conditions and compatibility requirements for the local region to be designed.

102 The concrete is assumed to transfer compression by compression fields, and the reinforcement in two or more directions transfers tension. Under certain conditions, a limited transfer of shear forces parallel to the cracks and tension in concrete between the cracks may be assumed.

103 Strains and stresses shall be calculated as average values over a cracked region. The strains can be assumed constant in local regions and through the thickness. Average strain in the reinforcement can be assumed equal to the average strain parallel to the direction of reinforcement for the region. Principal stress and principal strain of the concrete are assumed to have the same direction in the assumed compression field.

104 Design of shear walls, plates and shells can be based on forces acting in the plane. When members are subjected to moments in combination with membrane forces, the design may be performed by assuming the structural member divided into layers where the action effects are taken as membrane forces uniformly distributed through the thickness in each layer, and where the average strain in the layers satisfies the condition of linear strain variation through the thickness.

105 This method of calculation may also be used when designing for shear force in beams and slabs with shear reinforcement and for torsional moment in beams.

106 The design basis shall provide a relation between stress and strain for both reinforcement and concrete in areas subjected to a biaxial stress state in cracked concrete that is documented to give agreement between calculated capacity and tests. For reinforcement, the relation between average strain and average stress given in C307 can be assumed.

107 For concrete subjected to compression, the relation between stress and strain given in C300, with the stress ordinate reduced by the factor $f_{cd}/f_{ck}$ may be assumed.

For concrete in the assumed compression field a reduced design compressive strength shall be taken as

$$f_{cd} = f_{ck} (0.8 + 100 \times \varepsilon_0) \leq f_{cd}$$

where $\varepsilon_0$ is the average principal tensile strain.

108 The average tensile stresses between cracks shall be determined by relationships documented by representative tests.

109 It shall be demonstrated that the cracks can transfer both the shear stresses in the concrete and the tensile stresses in the reinforcement which are derived from the equilibrium requirements.

110 If the concrete tensile stresses between the cracks are not considered ($\sigma_t = 0$), the check of the stress condition in the cracks can be waived.

111 The stresses in the reinforcement at the cracks shall be determined from the equilibrium conditions and shall not exceed the design strength of the reinforcement.

#### H 200 Membrane (in-plane) shear resistance

201 Resistance to membrane forces in plates and shells is to be determined by recognized methods based on equilibrium considerations. The tensile strength of concrete is to be neglected.

202 For Membrane forces only, i.e. when the slab element is subjected to in-plane forces only (Figure 10) and the reinforcement is disposed symmetrically about mid-depth, the element may be designed as outlined below when at least one principal membrane force is tensile. The concrete is considered to carry compressive stress ($\sigma_c$) at angle $\theta$ to the x-axis (in the sense corresponding to the sign of $N_{xy}$).

The two sets of reinforcing bars are designed to carry the forces $F_x$ and $F_y$ where:

$$F_x = N_x + |N_{xy}| \cdot \cot \theta$$

$$F_y = N_y + |N_{xy}| \cdot \tan \theta$$

$$\sigma_c = |N_{xy}| / (b \cdot \sin \theta \cdot \cos \theta)$$

(valid for positive values of $F_x$ and $F_y$ and taking tensile stresses as positive.

The angle $\theta$ may be chosen arbitrarily for each loading case and each slab element, paying due regard to the requirements...
of Subheading R concerning minimum reinforcement.

For \( N_x < -|N_{xy}| \cot \theta \) no reinforcement is required in the x-direction. \( F_y \) and \( \sigma_c \) are then given by:

\[
F_y = N_y - \frac{N_{xy}^2}{N_x} \\
\sigma_c = \frac{(N_x + N_{xy}^2/N_x)}{h}
\]

For \( N_y < -|N_{xy}| \tan \theta \) no reinforcement is required in the y-direction \( F_x \) and \( \sigma_c \) are then given by:

\[
F_x = N_x - \frac{N_{xy}^2}{N_y} \\
\sigma_c = \frac{(N_y + N_{xy}^2/N_y)}{h}
\]

Finally, a situation may occur where both \( N_x \) and \( N_y \) are negative and \( N_x N_y > N_{xy}^2 \). No reinforcement is required and principal membrane forces may be calculated in accordance with conventional formulae.

### I. Regions with Discontinuity in Geometry or Loads

#### I.100 General

101 In areas with discontinuities in geometry or loads such that assumptions of plane sections remaining plane are invalid, the calculation may be based on force models in sufficient conformity with test results and theoretical considerations. The models might be truss systems, stress fields or similar that satisfy the equilibrium conditions.

If there is no recognized calculation model for the member in question, the geometry of the model may be determined from the stress condition for a homogeneous uncracked structure in accordance with the theory of elasticity.

102 The provisions of this sub-section shall be used to determine internal forces in the member at a distance less than \( d \) from the support or from concentrated loads. The internal forces may be used at distances up to \( 2 \cdot d \).

103 Internal forces shall be calculated based on an assumed force model of concrete compression struts and ties of reinforcement. Effective cross section for concrete compression struts shall be assumed in accordance with recognized calculation models.

104 Tensile forces caused by possible deviation in the assumed compressive field shall be considered.

The reinforcement shall be shaped in accordance with the analytical model and be anchored in accordance with the provisions of Sub-sec. K at the assumed joints.

105 Calculated concrete stresses in struts shall not exceed \( f_{c2a} \) as given in H107. When calculating \( f_{c2a} \), the average principal tensile strain is derived from the principal compressive strain in the strut and the tensile strain in the reinforcement crossing the strut.

106 It shall be demonstrated that the calculated forces in the assumed struts and ties can be transferred in the joints, with design concrete compressive strength in accordance with H105, and the other provisions of this standard. Increased design concrete compressive strength may be taken into account for par-
tially loaded areas. Where there is no special reinforcement or compressive stress normal to the compressive struts in the force model, reduced compressive concrete strength shall be assumed.

107 If the reduced compressive concrete strength $f_{cd}$ is not derived from the strain condition, the calculated compressive concrete stress in the assumed joints shall not exceed the following values:

- $1.1 \cdot f_{cd}$ in joints where no tensile reinforcement is anchored (bi- or triaxial compression)
- $0.9 \cdot f_{cd}$ in joints where tensile reinforcement in only one direction is anchored
- $0.7 \cdot f_{cd}$ in joints where tensile reinforcement in more than one direction is anchored.

J. Shear Forces in Construction Joints

J 100 General

101 In concrete joints between hardened concrete and concrete cast against it, the transfer of shear forces can be assumed in accordance with the provisions given in this clause.

102 Construction joints shall not be assumed to transfer larger forces than if the structure was monolithically cast.

103 A hardened concrete surface is classified as smooth, rough or toothed. A surface may be assumed as rough if it has continuously spread cavities of depth no less than 2 mm. When surfaces are assumed as toothed, the toothed shall have a length parallel with the direction of the force not exceeding 8 times the depth and the side surfaces of the toothed shall make an angle with the direction of the joint no less than 60°. The depth shall be minimum 10 mm.

104 The design shear strength of concrete, $\tau_{cd}$, can be taken into account only for contact surfaces that are cleaned and free of laitance before concreting, and where there are no tensile stresses perpendicular to the contact surface.

105 The shear force capacity parallel to a construction joint with an effective area $A_c$ and reinforcement area $A_s$ through the joint surface, shall be taken as:

$$ V_u = \tau_{cd} \cdot A_c + f_{cd} \cdot A_s \cdot (\cos \alpha + \mu \cdot \sin \alpha) \cdot \mu \cdot \sigma_c \cdot A_c < 0.3 \cdot f_{cd} \cdot A_c $$

where:

- $\alpha$ = the angle between the reinforcement and the contact surface, where only reinforcement with an angle between 90° and 45° (to the direction of the force) shall be taken into account
- $A_s$ = the reinforcement area that is sufficiently anchored on both sides of the joint and that is not utilized for other purposes
- $\mu$ = the friction factor
- $\sigma_c$ = the smallest simultaneously acting concrete stress perpendicular to the contact surface.

106 The reinforcement crossing the joint shall have a total cross-sectional area no less than 0.001 $A_c$, or there shall be a simultaneously acting compressive normal stress of minimum 0.4 MPa.

107 In joints parallel to the longitudinal axis the distance between the reinforcement units shall not exceed 4 times the minimum concrete thickness, measured perpendicular to the contact surface or 500 mm. The combination of values given in table J1 that gives the minimum capacity shall be used in the design.

K. Bond Strength and Anchorage Failure

K 100 General

101 The distances between the reinforcement bars shall be such as to ensure good bond.

102 Reinforcement in different layers is to be aligned in planes leaving sufficient space to allow for the passage of an internal vibrator.

103 Lap joints shall be made in a way that secures transfer of force from one rebar to another. The reduction of strength of a lap joint due to closely spaced lap joints is to be taken into account where relevant.

104 The lap joints shall be distributed. The maximum number of lap joints occurring at a given cross sectional plane is normally limited by the smaller of:

- 1/2 of the reinforcement area
- one reinforcement layer (the layer with largest reinforcement area).

Larger lengths of the lap joint than 80 $\phi$ are not to be utilized.

105 Resistance against bond and anchorage failure is to be determined by recognized methods. Both local bond and anchorage bond shall be investigated.

In zones of reduced bond (e.g. where gravitational settling of the concrete may reduce the compaction around the reinforcement) the design bond strength is not to be taken higher than

<table>
<thead>
<tr>
<th>Combination 1</th>
<th>Combination 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\tau_{cd}$</td>
<td>$\mu$</td>
</tr>
<tr>
<td>Smooth</td>
<td>0</td>
</tr>
<tr>
<td>Rough</td>
<td>0</td>
</tr>
<tr>
<td>Toothed</td>
<td>0</td>
</tr>
</tbody>
</table>
70% of the value for good bond zones.

Consideration is to be given to the state of stress in the anchorage zone. Adequate bond resistance is to be assured by transverse reinforcement, stirrups, spirals, hooks or mechanical anchorages.

Individual reinforcement bars shall have a development length no less than

\[ l_b = 0.25 \cdot \phi \cdot \sigma_b / f_{bd} + t \]

where:
- \( \phi \) is the diameter of the reinforcement bar
- \( \sigma_b \) is the calculated stress in the reinforcement bar in ultimate limit state at the cross section in question
- \( f_{bd} \) is the design bond strength, calculated in accordance with K116.
- \( t \) is the specified longitudinal tolerance for the position of the bar end. If such tolerances are not specified on the drawings the value of \( t \) shall not be taken less than \( 3 \cdot \phi \).

Required lap length when splicing shall be taken equal to the calculated development length. The lap length shall not be less than the greatest of 20\( \cdot \phi \) and 300 mm.

Bundled reinforcement bars shall have a development length no less than

\[ l_b = 0.25 \cdot \phi_e \cdot \sigma_b / (k_n \cdot f_{bc} + f_{bs}) + t \]

where:
- \( \phi_e \) = equivalent diameter in term of reinforcement cross section.
- \( f_{bc} \) and \( f_{bs} \) = design bond strengths in accordance with K116 with \( \phi = \phi_e \).
- \( k_n \) = a factor dependent on the number of bars in the bundle and is taken as: 0.8 for bundle of 2 bars 0.7 for bundle of 3 bars 0.6 for bundle of 4 bars
- \( t \) = the specified longitudinal tolerance for the position of the bar end, see K106.

The development length shall not be assumed to be effective over a length exceeding 80\( \cdot \phi \).

For lapped splices of bundled reinforcement with equivalent diameter larger than 32 mm, the bars shall be lapped individually and staggered at least the development length \( l_b \). When terminated between supports, the bars shall be terminated individually and staggered in the same way. The development length shall be calculated for each individual bar by entering the diameter of the bar in question for \( \phi_e \) in the formula.

The development length for welded wire fabric shall be no less than

\[ l_b = l_b - 0.3 \cdot \Sigma F_{vn} / (\gamma_k \cdot \phi \cdot f_{bd}) \]

where:
- \( \Sigma F_{vn} / \gamma_k \) = sum of forces \( F_{vn} \) corresponding to shear failure at cross wire welds within the development length
- \( l_b \) = development length in accordance with K106, shall not be taken as larger than the development length in accordance with K128.

\[ f_{bd} = \text{design bond strength calculated in accordance with K116, see also K106.} \]

For welded wire fabric \( F_{vn} = 0.2 \cdot A_s \cdot f_{bc} \geq 4 \text{kN} \), where \( A_s \) is the sectional area of the largest wire diameter.

Required lap length is equal to the calculated development length. The lap length shall not be less than the largest of 20\( \cdot \phi \) and 200 mm.

For individual prestressed reinforcement units, the development length for the prestressing force shall be taken as

\[ l_{bp} = \alpha \cdot \phi + \beta \cdot \sigma_p \cdot \phi / f_{bc} \]

where:
- \( \alpha \) is a factor given in table K1
- \( \beta \) is a factor given in table K1
- \( \phi \) is the nominal diameter of the reinforcement unit
- \( \sigma_p \) is the steel stress due to prestressing
- \( f_{bc} \) is the concrete related portion of the design bond strength in accordance with K1116

<table>
<thead>
<tr>
<th>Type of reinforcement</th>
<th>Smooth release of prestressing tension force</th>
<th>Sudden release of prestressing tension force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \alpha )</td>
<td>( \beta )</td>
</tr>
<tr>
<td>Plain wire</td>
<td>10</td>
<td>0.20</td>
</tr>
<tr>
<td>Indented wire</td>
<td>0</td>
<td>0.17</td>
</tr>
<tr>
<td>Strand</td>
<td>0</td>
<td>0.14</td>
</tr>
<tr>
<td>Ribbed bar</td>
<td>0</td>
<td>0.07</td>
</tr>
</tbody>
</table>

The part \( \alpha \cdot \phi \) in the formula for \( l_{bp} \) defines a length where no force transmission is assumed.

Post-tensioning anchorages shall be designed for the ultimate strength of the tendon. The anchorage unit is to be designed so that transfer of forces to the surrounding concrete is possible without damage to the concrete. Documentation verifying the adequacy of the anchorage unit is to be approved.

The design of anchorage zones is to be in accordance with recognized methods. Reinforcement is to be provided, where required, to prevent bursting or splitting.

The development length for such reinforcement is to be limited to 300 MPa.

The release of prestressing force may be assumed to be smooth if one of the following requirements is fulfilled:
- the prestressing force is released gradually from the abutments;
- the impact against the end of the concrete structure is damped by a buffer between the end of the concrete structure and the point where the reinforcement is cut;
- both concrete and prestressed reinforcement are cut in the same operation by sawing.

Development of tensile force caused by external loads shall be calculated in accordance with K106. Within the development length for prestressed tensile force, \( f_{bd} \) shall be reduced by the factor \( (1 - \sigma_p / f_{bc}) \). In this calculation, long-term reduction of \( \sigma_p \), caused by shrinkage, creep and relaxation shall be considered. The development length for the reduced prestressing force shall be assumed to be unchanged, equal to \( l_{bp} \).
Transverse tensile forces in the development zone shall be resisted by reinforcement, unless it is shown that reinforcement can be omitted.

The design bond strength $f_{bd}$ for ribbed bar, indented bar, indented wire and strand can be taken as:

$$f_{bd} = f_{bc} + f_{bs} \leq 2k_1f_{td}$$

where:

- $f_{bc} = k_1k_2f_{td}(l/3 + 2c/3\phi)$
- $f_{bs} = k_3(A_{st}/s_1\phi) \leq 1.5\text{ MPa}$
- $c$ is the least of the dimensions $c_1, c_2$ and $(s_1 - \phi)/2$ given in Figure K2
- $\phi$ is the diameter of the anchored reinforcement
- $k_3$ is a factor depending on the transverse reinforcement and its position as given in Figure K14. The factor $k_3$ is taken as zero for strands.
- $A_{st}$ is the area of transverse reinforcement not utilized for other tensile forces and having a spacing not greater than 12 times the diameter of the anchored reinforcement. If the reinforcement is partly utilized, the area shall be proportionally reduced
- $s_1$ is the spacing of the transverse reinforcement
- $k_2$ has the value 1.6 if the spacing $s$ between the anchored bars exceeds $9\cdot\phi$ or $(6c + \phi)$ whichever is the larger, $k_2$ has the value 1.0 if $s$ is less than the larger of $5\phi$ and $(3c + \phi)$. For intermediate values interpolate linearly.

For plain reinforcement take $f_{bd} = k_1f_{td}$

When calculating development of force in reinforcement which during concreting has an angle less than $20^\circ$ to the horizontal plane, the following reduction of the portion $f_{bc}$ of the design bond strength $f_{bd}$ according to K116 shall be made:

- if the concreting depth below the reinforcement exceeds 250 mm, the reduction for ribbed bars is 30% and for other types of bars 50%. If the concreting depth is 100 mm or less, no reduction is made. For intermediate values linear interpolation shall be performed.
- if there is a tensile stress perpendicular to the anchored reinforcement larger than $0.5 \cdot f_{td}$ in the development zone, the reduction is 20%.

The highest of the reductions given above shall be applied. The reductions shall not be combined.

At a simply supported end, the development length determined according to K106 through K115 may be reduced above the support, if the support reaction is applied as direct compression against the tension face. In this case the stirrups shall continue throughout the support region.

When calculating the development length, the value $f_{bc}$ may be
increased by 50%, but \( f_{cd} \) shall not have a higher value than what corresponds to the maximum value in accordance with K116.

120 Reinforcement that is taken into account at the theoretical support, shall normally be extended at least 100 mm beyond this. The position of the reinforcement shall be given on the drawings, with tolerance limits.

121 If reinforcement in several layers are spliced or anchored in the same section, the capacity shall be limited to the value that can be calculated for the bars in only one layer, using the layer that gives the highest capacity. This provision may be waived if otherwise demonstrated by a more accurate design.

122 Reinforcement can also be anchored with special anchor units such as end plates.

A combination of several anchorage methods may be utilized. The total anchorage capacity can be calculated as the entire capacity from the anchorage method giving the highest portion and half of the anchorage capacity from each of the remaining anchorage methods. For plain steel, a combination of bond and end anchorage shall not be utilized.

123 For tensile reinforcement of ribbed bar or indented bar with an anchorage hook a concentrated force development along the bent part of the hook may be assumed. A hook shall only be assumed effective if it has transverse reinforcement and is formed in accordance with Q408. If the hook is bent with an angle of 90°, the straight end after the bend shall be at least ten times the diameter of the bent bar. If the angle is 135°, the straight part may be reduced to five times the diameter of the bar.

For bars of steel grade B500A to C (see Q400), the concentrated force in the bend may be taken as 25% of the capacity of the bar, if the hook has an angle of 90°. If the angle is 135°, the force can be taken as 40%.

Anchorage for the remaining portion of the force in the bar shall be calculated by force development along the bar outside the bent part.

Tensile reinforcement of quality B500B or B500C with anchorage hook as described above, may be presumed to be anchored in the bent part of the bar provided the bar is bent with a mandrel of diameter equal to or less than 4·\( \phi \) and otherwise bent in accordance with Q400.

124 If the development length is not calculated in accordance with K106 through K108, the anchorage length of reinforcement in one layer in normal density concrete may be simplified be determined as follows:

a) For ribbed bar, the anchorage length shall be taken as 50·\( \phi \) for B500A to C. This applies provided the concrete cover is at least \( \phi \) and the spacing between the anchored bars is at least 8·\( \phi \). If the transverse reinforcement is located closest to the concrete surface and the concrete cover of the anchored reinforcement is at least 1.5·\( \phi \), the spacing shall be at least 5·\( \phi \).

b) For plain bars with end hooks, the anchorage length is taken as 40·\( \phi \) assuming that \( f_{sk} \leq 250 \text{ MPa} \).

c) For welded wire fabric, the anchorage length shall be at least so large that

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L. Partially loaded areas

L 100 General

101 Where a compression force \( F_{r} \) is transferred to a concrete member with nearly uniformly distributed compressive stresses over a limited loading area \( A_{1} \), increased compressive stress over the loaded area relative to \( f_{cd} \) may be allowed provided this area represents only a part of the surface (cross section) of the concrete member, and if the force can be assumed transferred further in the same direction and distributed over a larger distribution area, \( A_{2} \), in the concrete member.

102 The loaded area \( A_{1} \) used in the calculation and the assumed distribution area \( A_{2} \) shall be such that their centroids coincide with the applied force resultant. The side faces of the cut pyramid or cone which are formed between loaded area and distribution area shall not have an inclination larger than 1:2.

103 The cross-sectional dimensions of the distribution area shall not be assumed larger than the sum of the dimensions of the loaded surface measured in the same main direction and the concrete thickness measured parallel to the direction of the force.

104 If more than one load act simultaneously the respective distribution areas shall not be overlapping each other.

105 The compressive capacity for normal density concrete can be taken as

\[
F_{cd} = A_{1} f_{cd} \left( \frac{A_{2}}{A_{1}} \right)^{1/3}
\]

106 The compressive capacity for lightweight concrete can be taken as

\[
F_{cd} = A_{1} f_{cd} \left( \frac{A_{2}}{A_{1}} \right)^{1/4}
\]

107 The dimensions of the distribution area shall not be assumed greater than 4 times the dimensions of the loaded area measured in the same main direction, see Figure 15.
If the ratio between the larger and smaller dimension of the loaded area is less than 2, and the distribution area $A_2$ is assumed to be geometrically identical to the loaded area $A_1$, the compressive capacity for normal density concrete may be taken as

$$F_{cd} = A_1 \cdot f_{cd} \left( \frac{A_2}{A_1} \right)^{1/2} \leq 3 \cdot A_1 \cdot f_{cd}$$

The compressive capacity for lightweight aggregate concrete may be taken as

$$F_{cd} = A_1 \cdot f_{cd} \left( \frac{A_2}{A_1} \right)^{1/3} \leq 2 \cdot A_1 \cdot f_{cd}$$

see Figure 15

The concrete shall be sufficiently reinforced for transverse tensile forces.

In the two principal directions perpendicular to the direction of the compressive force reinforcement for the transverse forces shall be provided according to

$$0.25 \cdot F_f (1 - a_1/a_2) \quad \text{and} \quad 0.25 \cdot F_f (1 - b_1/b_2)$$

see Figure 15.

The transverse tensile reinforcement shall be placed such that the centroid of the reinforcement is located at a distance from the loaded area equal to half the length of the side of the distribution area in the same direction, but not larger than the distance to the distribution area. The reinforcement may be distributed over a width corresponding to the length of the side of the distribution area normal to the direction of the reinforcement and over a height that corresponds to half the side of the distribution area parallel to the direction of the reinforcement.

Additional reinforcement shall be provided, if additional transverse forces can develop caused by transverse expansion of soft supports (shims), fluid pressure or similar.

It shall be demonstrated that forces caused by bent reinforcement can be resisted. If no reinforcement is provided for transverse tension normal to the plane of the bent reinforcement, the spacing $s$ shall not be greater than twice the distance from the centre of the bar to the free surface.

In order to limit the contact pressure in the bend, the reinforcement shall not be bent around a mandrel diameter less than determined by the equations

$$D = \phi (\phi/s)^{1/2} \frac{\sigma_s}{f_{cd}}$$

for normal density concrete and

$$D = 1.5 \phi (\phi/s)^{1/2} \frac{\sigma_s}{f_{cd}}$$

for lightweight aggregate concrete

In this calculation, $s$ shall not exceed $4 \cdot \phi$.

For requirements to the mandrel diameter, see also Q400

It is not necessary to check that stirrups made in accordance with Q408 are in accordance with the provisions of this clause.

**M. Fatigue Limit State**

**M 100 General**

All stress fluctuations imposed during the life of the structure which are significant with respect to fatigue evaluation shall be taken into account when determining the long term distribution of stress range, (see Sec.5 D2000).

Statistical considerations for loads of a random nature are required for determination of the long term distribution of fatigue loading effects. Deterministic or spectral analysis may be used. The method of analysis shall be documented.

The effects of significant dynamic response shall be properly accounted for when determining stress ranges. Spe-
cial care is to be taken to adequately determine the stress ranges in structures or members excited in the resonance range. The amount of damping assumed is to be appropriate to the design.

104 The geometrical layout of the structural elements and reinforcement is to be such as to minimize the possibility of fatigue failure. Ductility is to be assured by confinement of the concrete by appropriate reinforcement.

105 Fatigue design may alternatively be undertaken utilizing methods based on fracture mechanics, or a combination of methods based on fatigue tests and cumulative damage analysis, methods based on fracture mechanics, or a combination of these. Such methods shall be appropriate and adequately documented.

106 For structures subject to multiple stress cycles, it shall be demonstrated that the structure will endure the expected stresses during the required design life.

107 Calculation of design life at varying stress amplitude can be based on cumulative linear damage theory. The stresses due to cyclic actions may be arranged in stress-blocks (action effect-blocks) each with constant amplitude and a corresponding number of stress cycles, \( n_i \).

A minimum of 8 blocks is recommended. The design criterion is

\[
\sum_{i=1}^{k} \frac{n_i}{N_i} < \eta
\]

where:

\( k \) = the number of stress-blocks
\( n_i \) = the number of cycles in stress-block \( i \)
\( N_i \) = the number of cycles with constant amplitude which causes fatigue failure
\( \eta \) = cumulative damage ratio

108 The characteristic fatigue strength or resistance (S-N curve) of a structural detail is to be applicable for the material, structural detail, state of stress considered and the surrounding environment. S-N curves are to take into account material thickness effects as relevant.

109 The cumulative damage ratio (\( \eta \)) to be used in the design is to depend on the access for inspection and repair. Cumulative damage ratios according to Table M1 are normally acceptable.

### Table M1 Cumulative Damage Ratios (\( \eta \))

<table>
<thead>
<tr>
<th>No access for inspection and repair</th>
<th>Below or in the splash zone(^1)</th>
<th>Above splash zone(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.33</td>
<td>0.5</td>
<td>1.0</td>
</tr>
</tbody>
</table>

\(^1\) In typical harsh environment (e.g. the North Sea or equivalent) structural details exposed to seawater in the splash zone are normally to be considered to have no access for inspection and repair, i.e. the cumulative damage ratios is to be reduced to 0.33.

\(^2\) For reinforcement, which cannot normally be inspected and repaired; the cumulative damage ratio for reinforcement above splash zone is reduced to 0.5.

110 The action effects shall be calculated according to the theory of elasticity.

111 The capacity may be assumed to be adequate, when calculated design life for the largest acting amplitude corresponds to at least \( 2.0 \times 10^6 \) cycles if the fatigue loading is caused by randomly variable actions such as wind, waves, traffic etc.

### M 200 Fatigue strength, design life

201 The design life of concrete subjected to cyclic stresses may be calculated from

\[
\log_{10} N = C_1 (1 - \frac{\sigma_{\text{max}}}{f_{\text{rd}}} ) / (1 - \frac{\sigma_{\text{min}}}{f_{\text{rd}}})
\]

where:

\( f_{\text{rd}} \) = the compressive strength for the type of failure in question
\( \sigma_{\text{max}} \) = the numerically largest compressive stress, calculated as the average value within each stress-block
\( \sigma_{\text{min}} \) = the numerically least compressive stress, calculated as the average value within each stress-block.

When \( \sigma_{\text{min}} \) is tension, it shall be taken as zero when calculating the design life.

The factor \( C_1 \) shall be taken as

- 12.0 for structures in air
- 10.0 for structures in water for those stress-blocks having stress variation in the compression-compression range
- 8.0 for structures in water for those stress-blocks having stress variation in the compression-tension range.

If the calculated design life \( \log N \) is larger than the value of \( x \) given by the expression

\[
X = C_1 / (1 - (\sigma_{\text{max}}/f_{\text{rd}}) + 0.1 \cdot C_1)
\]

The life may be increased further by multiplying the value of \( \log N \) by the factor \( C_2 \) where this is taken as

\[
C_2 = (1 + 0.2 \cdot (\log_{10} N - X)) > 1.0
\]

202 The design life of reinforcement subjected to cyclic stresses may be calculated based on

\[
\log_{10} N = C_3 - C_4 \log_{10} \Delta \sigma
\]

where:

\( \Delta \sigma \) is the stress variation of the reinforcement (MPa)
\( C_3 \) and \( C_4 \) are factors dependent on the reinforcement type, bending radius and corrosive environment.

The maximum stress \( \sigma_{\text{max}} \) in the reinforcement shall be less than \( \frac{f_{\text{rd}}}{\gamma_s} \), where \( \gamma_s \) is taken from Table B1.

203 For straight reinforcement bars in a concrete structure, which is exposed to moderate (NA) and mildly (LA) aggressive environment, the value of \( C_3 = 19.6 \) and \( C_4 = 6.0 \).

For reinforcement bent around a mantel of diameter less than \( 3 \cdot \phi \) and used in a structure which is exposed to moderate (NA) and mildly (LA) aggressive environment, the value of \( C_3 = 15.9 \) and \( C_4 = 4.8 \).

For intermediate bending diameters between \( 3 \cdot \phi \) and straight bars, interpolated values may be used.

Infinite fatigue life may be assumed if the calculated value of \( N \) is greater than \( 2 \times 10^8 \) cycles.

204 For straight reinforcement bars in a concrete structure, which is exposed to specially (SA) or severely (MA) aggressive environment, the influence of corrosion on the fatigue properties shall be assessed separately. Values of \( C_3 \) and \( C_4 \) for straight bars are suggested in Table M2.

Special assessment shall also be made for bent bars.

Reinforcement which is protected against corrosion using cathodic protection may be assessed for fatigue life using the values \( C_3 \) and \( C_4 \) in M203 above.

### Table M2

<table>
<thead>
<tr>
<th>Level of Stress Variations (MPa)</th>
<th>400 &gt; ( \Delta \sigma ) &gt; 235</th>
<th>235 &gt; ( \Delta \sigma ) &gt; 65</th>
<th>65 &gt; ( \Delta \sigma ) &gt; 40</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_3 )</td>
<td>15.7</td>
<td>13.35</td>
<td>16.97</td>
</tr>
<tr>
<td>( C_4 )</td>
<td>4.5</td>
<td>3.5</td>
<td>5.5</td>
</tr>
</tbody>
</table>

### M 300 Bending moment and axial force

301 Stresses in concrete and reinforcement shall be calculated based on a realistic stress-strain relationship. The effects of shrinkage and creep may be taken into account when calcu-
lating stresses.
For concrete subject to compression, \( f_{cd} \) is taken equal \( f_{cd} \).

302 If a more accurate calculation is not performed, stresses in concrete and reinforcement can be calculated with a linear stress distribution in the compression zone. The calculations may be based on a modulus of elasticity equal to 0.8 \( E_{ck} \) for the concrete.

In such a calculation the reference strength, \( f_{rd} \) of the concrete in compression can be taken as

\[
f_{rd} = \alpha f_{cd}
\]

The value of \( \alpha \) may be calculated as \( \alpha = 1.3 - 0.3 \beta > 1.0 \) where:

\[
\beta = \frac{\text{the ratio between the numerically smallest and largest stresses acting simultaneously in the local compressive zone. The distance between the points used when calculating } \beta \text{ shall not exceed 300 mm (0 < } \beta < 1.0).}
\]

M 400 Shear force

401 The design life at tensile failure of concrete without shear reinforcement can be calculated in accordance with M201.

\[
\begin{align*}
\sigma_{\text{max}} f_{rd} & \text{ shall be replaced by } V_{\text{max}} V_{cd} \\
\sigma_{\text{min}} f_{rd} & \text{ shall be replaced by } V_{\text{min}} V_{cd}
\end{align*}
\]

402 For those stress-blocks where the shear force changes sign, the denominator in the formula for \( \log N \) in M201 shall be replaced by

\[
1 + \frac{V_{\text{min}}}{V_{cd}}
\]

If the shear force changes sign the calculation shall, if necessary, be performed with both the positive and negative values for \( V_{\text{max}} \) and \( V_{\text{min}} \) respectively in the formulas above.

\( V_{cd} \) shall be calculated in accordance with F200.

The factor \( C_1 \) shall be taken as

12.0 for structures in air where the shear force does not change sign.
10.0 for structures in air where the shear force changes sign and for structures in water where the shear force does not change sign.
8.0 for structures in water where the shear force changes sign.

403 The design life for structures with shear reinforcement can be calculated in accordance with M201 by assuming the concrete at all load levels to transfer a portion of the acting shear force corresponding to the contribution of the concrete to the combined shear capacity of concrete and shear reinforcement. When calculating the shear contribution of the concrete, the tensile strength of the concrete shall be reduced to 0.5 \( f_{td} \).

Alternatively, the total shear force may be assumed carried by the shear reinforcement. The design life of the concrete at tensile shear failure shall be demonstrated in accordance with M100.

404 The design life of the shear reinforcement can be calculated in accordance with M202 by assuming the shear reinforcement at all load levels to transfer a portion of the acting shear force corresponding to the contribution of the shear reinforcement to the combined shear capacity of the shear reinforcement and the concrete calculated with a reduced tensile strength equal to 0.5 \( f_{td} \). The stresses in the shear reinforcement shall be calculated based on an assumed truss model with the compression struts inclined at 45°.

405 If the shear force changes sign, account of this shall be made when calculating the number of stress cycles in the shear reinforcement.

406 When demonstrating the compressive failure capacity the design life can be calculated in accordance with F208.

\[
\begin{align*}
\sigma_{\text{max}} f_{rd} & \text{ shall be replaced by } V_{\text{max}} V_{cd} \\
\sigma_{\text{min}} f_{rd} & \text{ shall be replaced by } V_{\text{min}} V_{cd}
\end{align*}
\]

For those stress-blocks where the shear force changes sign, use \( V_{\text{min}} = 0 \).

\( V_{cd} \) shall be calculated in accordance with F208.

The factor \( C_1 \) shall be entered with the values given in M402.

407 In addition to the checks required above, the expected design life of cross sections subjected to simultaneously acting axial forces shall be calculated from the principal compressive stresses at the mid-height of the cross section. The shear stresses in this case may be assumed constant over a height corresponding to the internal lever arm, which may be taken as 0.9d. The reference stress of the concrete, \( f_{cd} \) shall be taken as \( f_{cd} \).

M 500 Anchorage and splicing

501 Demonstration of the design life for force development can be performed in accordance with M201.

\[
\begin{align*}
\sigma_{\text{max}} f_{rd} & \text{ shall be replaced by } \tau_{\text{max}} f_{bd} \\
\sigma_{\text{min}} f_{rd} & \text{ shall be replaced by } \tau_{\text{min}} f_{bd}
\end{align*}
\]

The bond strength, \( f_{bd} \) shall be calculated in accordance with K116.

The bond stress, \( \tau_b \) shall be taken as

\[
\tau_b = 0.25 \frac{\phi}{h_b}
\]

502 For structures in air \( C_1 \) shall be 12.0, for structures in water \( C_1 \) shall be 10.0. If the bond stresses change sign, this reversible effect on fatigue life shall be especially considered when evaluating the fatigue life.

N. Accidental Limit State

N 100 General

101 Structural calculations for an accidental limit state shall document the capacity of the structure. The calculations can be performed according to the regulations of this clause and Subsecs. D, E, F, G, H, I, J, K, L or P.

102 The material coefficients are given in B607.

103 Strength and strain properties are as given in Sec.6 B600, C100, C200 and C300. The strain limits \( \varepsilon_{cu} \) and \( \varepsilon_{um} \) may however be given particular assessment.

104 Structures in Safety classes 2 and 3 (see Sec.2 A300) shall be designed in such a way that an accidental load will not cause extensive failure. Offshore structures are generally defined belonging to safety class 3.

The design may permit local damage and displacements exceeding those which are normally assumed by design in the ultimate limit state, and structural models and load transferring mechanisms which are normally not permitted may be assumed.

N 200 Explosion and impact

201 For explosion loads and impact type of loads, increased modulus of elasticity and material strength based on a documented relationship between strength and strain rate may be taken into account. The assumed strain rate in the structure shall be documented.

202 The structural calculations may take account of the load variation with time and the dynamic properties of the structure.

N 300 Fire

301 Required fire resistance is determined in one of the fol-
lowing ways:

— An offshore structure shall be designed to resist a fire in accordance with the requirements of DNV-OS-A101, if no other requirements for the actual structure are provided from National Building Code or other National Regulations.

— For structures where the National Building Regulations give requirements to fire resistance as a function of fire exposure, the fire loading is calculated, and the required fire resistance is determined in accordance with the Building Code.

— Necessary fire resistance can be determined based on calculated fire loading and fire duration or a temperature-time curve for those cases which are not covered by the National Building Code.

302 Structures can be demonstrated to have adequate fire resistance according to one of the following methods:

— calculation in accordance with N303
— use of other Internationally accepted methods
— testing in accordance with an International standard accepted.

The adequacy of the fire resistance shall be documented.

303 The temperature distribution in the structure is determined based on the actual temperature/time curve and the required fire resistance, taking the effects of insulation and other relevant factors into consideration.

The strength properties of the materials as a function of the temperature are as given in Sec.6 C104 for concrete and Sec.6 C311 for reinforcement. Special strength properties shall be applied for concrete exposed to temperatures down to cryogenic temperature.

The strain properties of the concrete are as given in Sec.5 D209. The strain properties of the reinforcement are as given in Sec.5 D210.

A stress-strain diagram similar to that applicable for the ultimate limit state, with the stress ordinate reduced, can be assumed for the concrete when calculating the capacity.

Displacements and forces caused by the temperature changes in the structure shall be taken into account in the design.

304 The structure shall be so detailed that it maintains the required load bearing ability for the required period. An appropriate geometrical form which reduces the risk of spalling of the concrete cover shall be sought. The reinforcement shall be so detailed that in the event of spalling of concrete cover at laps and anchorages, the reinforcement still has adequate capacity.

305 The temperature insulation ability and gas tightness of partitioning structures shall be demonstrated in the accidental limit state of fire.

O. Serviceability Limit State

O 100 General

101 When calculating action effects in the serviceability limit state, the mode of behaviour of the structure in this limit state shall govern the choice of analytical model.

The design resistance in SLS is normally related to criteria for:

— durability
— limitation of cracking
— tightness
— limitation of deflections and vibrations.

102 The properties of the materials under short - and long-term actions and the effect of shrinkage, temperature and imposed displacements, if any, shall be taken into account.

Cracking of concrete is to be limited so that it will not impair the function or durability of the structure. The crack size is controlled by ensuring that the predicted crack width by calculations is within the nominal characteristic crack width limits in Table O1.

103 When it is necessary to ensure tightness of compartments against leakage due to external/internal pressure difference, the concrete section is to be designed with a permanent boundary compression zone, see O500.

104 Concrete structures are to have at least, a minimum amount of reinforcement to provide adequate ability for crack distribution and resistance against minor load effects not accounted for in design.

105 The material coefficients ($\gamma_m$) for concrete and reinforcement are given in B607.

106 In the analysis and structural design it shall be ensured that displacements and cracks, spalling of concrete and other local failures are not of such a nature that they make the structure unfit for its purpose in the serviceability limit state, nor alter the assumptions made when designing in the other limit states.

O 200 Durability

201 For concrete structures of permanent character, dependent on the environmental conditions to which the structure is exposed, a material composition shall be selected in accordance with Sec.4.

202 The environment shall be classified in the following Environmental classes:

SA:  Specially aggressive environment: Structures exposed to strong chemical attack which will require additional protective measures. This may require specially mixed concrete, membranes or similar.

MA:  Severely aggressive environment: Structures in saline water, in the splash zone or exposed to sea spray, structures exposed to aggressive gases, salt or other chemical substances, and structures exposed to repeated freezing and thawing in a wet condition.

NA:  Moderately aggressive environment: Outdoor structures or indoor structures in humid environment and structures in fresh water.

LA:  Mildly aggressive environment: Indoor structures in dry climate without aggressiveness.

For structures of class SA, the requirements for material mixtures shall be considered in relation to the chosen protective measures. If the concrete may become exposed to the aggressive environment, at least the requirements for class MA shall be fulfilled.

203 In order to protect the reinforcement against corrosion and to ensure the structural performance, the reinforcement shall have a minimum concrete cover as given in Q200 and the nominal crackwidths calculated in accordance with O700 shall be limited as given in table O1.

<table>
<thead>
<tr>
<th>Environmental Class</th>
<th>Reinforcement sensitive to corrosion $w_g$</th>
<th>Reinforcement slightly sensitive to corrosion $w_g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA</td>
<td>Special consideration</td>
<td>Special considerations</td>
</tr>
<tr>
<td>MA</td>
<td>0.20 mm</td>
<td>0.30 mm</td>
</tr>
<tr>
<td>NA</td>
<td>0.20 mm</td>
<td>0.40 mm</td>
</tr>
<tr>
<td>LA</td>
<td>0.40 mm</td>
<td>-</td>
</tr>
</tbody>
</table>

204 Cold-worked prestressed reinforcement having a stress
exceeding 400 MPa, and reinforcement with diameter less than 5 mm, shall be considered as reinforcement sensitive to corrosion. Other types of reinforcement can be considered as slightly sensitive to corrosion.

205 For structures permanently submerged in saline water, the crack width requirements given for class NA in Table O1 apply. Exceptions are structures with water on one side and air on the opposite side, for which the requirements for class MA apply on the air side.

206 The crack width limitations given in Table O1 are related to the crack width at a distance from the reinforcement corresponding to the minimum concrete cover in accordance with Table Q1.

When the concrete cover is larger, the nominal crack width when comparing with the values in Table O1 may be taken as:

\[ w_k = w_{ck} \cdot \frac{c_1}{c_2} > 0.7 \cdot w_{ck} \]

where:

- \( w_{ck} \) = crack width calculated in accordance with O700.
- \( c_1 \) = minimum concrete cover, see Table Q1
- \( c_2 \) = actual nominal concrete cover

207 If reinforcement sensitive to corrosion is placed on the inside of slightly corrosion sensitive reinforcement and with larger concrete cover than the minimum requirement, the nominal crack width when comparing with the requirements for corrosion sensitive reinforcement in Table O1 may be taken as:

\[ w_k = w_{ck} \cdot \frac{\varepsilon_{s2}}{\varepsilon_{s1}} \]

where:

- \( \varepsilon_{s1} \) = tensile strain in reinforcement slightly sensitive to corrosion on the side with highest strain
- \( \varepsilon_{s2} \) = tensile strain at the level of the reinforcement sensitive to corrosion

208 For cross sections with reinforcement sensitive to corrosion the crack limitation requirements do also apply for cracks parallel to this reinforcement.

209 When calculating crackwidths for comparison with the values in Table O1, long-term actions shall be applied in combination with short-term actions. The short-term actions shall be chosen so that the crack width criterion will not be exceeded more than 100 times during the design life of the structure.

210 All load coefficient in this calculation shall be taken as 1.0.

211 If more accurate values are not known for short-term but frequently repeated actions such as wind, traffic and wave actions, 60% of the characteristic value of the action may be applied. For other variable actions that rarely reach their characteristic value, 100% of the long-term part of the actions in combination with 40% of the short-term part of the actions may be applied.

212 For short periods in the construction phase, the crack width limitation given in Table O1 may be exceeded by up to 100%, but not more than 0.60 mm in the classes where limiting values are specified, when the anticipated actions are applied.

213 The strain in the reinforcement shall not exceed the yield strain during short period loading in the construction phase or any permanent SLS load condition.

O 300 Displacements

301 It shall be demonstrated by calculations that the displacements are not harmful if the use of the structure or connected structural members imposes limits to the magnitude of the displacements.

302 Normally, the tensile strength of the concrete shall be ignored when calculating displacements. However, it may be taken into account that the concrete between the cracks will reduce the average strain of the reinforcement and thus increase the stiffness.

303 Action effects when calculating displacements shall be determined by use of actions and load factors in accordance with Sec.5 C100. Effect of prestressing forces shall be taken into account in accordance with Sec.5 D600.

When calculating long-term displacements, the variation of the variable actions with time may be taken into account.

O 400 Vibrations

401 If a structure and the actions are such that significant vibrations may take place, it shall be demonstrated that these are acceptable for the use of the structure.

O 500 Tightness against leakages of fluids

501 In structures where requirements to tightness against fluid leakages are specified, concrete with low permeability and suitable material composition shall be selected, see Sec.4:

- the acting tensile stresses and nominal crack widths shall be limited
- geometrical form and dimensions shall be chosen which permit a proper placing of the concrete.

502 Members subjected to an external/internal hydrostatic pressure difference shall be designed with a permanent compression zone not less than the larger of:

- 0.25·h
- values as given in Table O2

### Table O2 Depth of Compression Zone versus Pressure Difference

<table>
<thead>
<tr>
<th>Pressure Difference (kPa)</th>
<th>Depth of Compression Zone (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 150</td>
<td>100</td>
</tr>
<tr>
<td>&gt; 150</td>
<td>200</td>
</tr>
</tbody>
</table>

The above applies for the operating design condition using ULS combination b) (ref.: Sec.5 C105) except that a load coefficient of 0.5 is used instead of 1.3 for the environmental load (E).

503 Oil containment structures with an ambient internal oil pressure greater than or equal to the ambient external water pressure (including pressure fluctuations due to waves) shall be designed with a minimum membrane compressive stress equal to 0.5 MPa for the operation design condition using ULS combination b) (ref.: Sec.5 C105) except that a load coefficient of 0.5 is used instead of 1.3 for the environmental load (E). However, this does not apply if other constructional arrangements, e.g. special barriers, are used to prevent oil leakage.

504 In structures where requirements to tightness against leakages are specified, the reinforcement shall meet the requirements for minimum reinforcement for structures with special requirements to limitation of crackwidths, see R105 and R502.

O 600 Tightness against leakage of gas

601 Concrete is not gas tight and special measures shall be taken to ensure a gas tight concrete structures, when this is required.

O 700 Crackwidth calculation

701 Concrete may be considered as uncracked if the principal tensile stress \( \sigma_1 \) does not exceed \( f_{wr}/k_1 \).

With combined axial tensile force and bending moment the following condition applies:

\[ (k_w \sigma_{N} + \sigma_M) < k_w f_{wr}/k_1 \]

With combined axial compression force and bending moment the following condition applies:
In cases where the corrosion sensitive reinforcement is placed only in the compression zone, then the values of $k_1$ for “None corrosively 0.85·$f_{02}$, may be permitted provided it is documented

crackwidth is influenced by these parameters.

Stresses caused by temperatures, creep, shrinkage, deformations etc. shall be included in the evaluation provided the

Stresses in concrete can cover all properties of the structure, or only certain properties

e.g. a beam, a part of a structure (e.g. a beam support) or to a detail of a structure (e.g. a fixing device to a beam). The test
can cover all properties of the structure, or only certain proper-

Table O3 Values of Constant Parameter $k_1$

<table>
<thead>
<tr>
<th>Environmental Class</th>
<th>Corrosion sensitive Reinforcement</th>
<th>None Corrosion Sensitive Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>LA</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>NA</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>MA</td>
<td>2.0</td>
<td>1.5</td>
</tr>
</tbody>
</table>

In cases where the corrosion sensitive reinforcement is placed only in the compression zone, then the values of $k_1$ for “None Corrosion Sensitive Reinforcement” can be used.

For structures permanently submerged in saltwater, the values of $k_1$ for Environmental Class MA shall be used.

Stresses caused by temperatures, creep, shrinkage, deformations etc. shall be included in the evaluation provided the crackwidth is influenced by these parameters.

Table O4: Stress limitations for simplified documentation of satisfactory state of cracking

<table>
<thead>
<tr>
<th>Nominal characteristic crackwidth</th>
<th>Type of load effect</th>
<th>Stress in reinforcement (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Spacing between the bars or bundles of bars(mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100 mm</td>
</tr>
<tr>
<td>Wk = 0.4 mm</td>
<td>Bending</td>
<td>320 MPa</td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>300 MPa</td>
</tr>
<tr>
<td>Wk = 0.2 mm</td>
<td>Bending</td>
<td>170 MPa</td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>160 MPa</td>
</tr>
</tbody>
</table>

The listed stresses apply to cracks perpendicular to the direction of the reinforcement, and only when the amount of tensile reinforcement is no less than 0.005 $A_{c}$.

If the cross-sectional height exceeds 500 mm, then the reinforcement stresses for tension in Table O4 shall be used for simplified documentation.

704 In the calculations of stresses in reinforcement or crackwidth in structures exposed to water pressure of magnitude sufficient to influence the calculated stress level or crackwidth, then the impact of the water pressure in the crack shall be included in the calculation.

O 800 Limitation of stresses in prestressed structures

801 Stresses in prestressed reinforcement

The stresses in the prestressed reinforcement shall for no combination of actions exceed 0.8 $f_{y}$, alternatively 0.85·$f_{02}$.

During prestressing, however, stresses up to 0.85·$f_y$ alternatively 0.85·$f_{02}$, may be permitted provided it is documented that this does not harm the steel, and if the prestressing force is measured directly by accurate equipment.

802 Stresses in concrete

When a prestressing force acts within a concrete compression zone, the stress at the outer compressive fibre of the concrete shall not exceed the lesser of 0.6·$f_{ckj}$ or 0.5·$f_{ck}$ in the serviceability limit state.

The outer compressive fibre stress shall be calculated assuming a linear distribution of stresses over the cross section. $f_{ckj}$ shall be taken as the strength of the concrete at the time when

If a high predicted cracking load (cracking moment) is non conservative, then $f_{ck}$ shall be used in the calculations and $k_1$ shall be taken as 1.0.

702 The characteristic crackwidth of a reinforced concrete member exposed to tensile forces and shrinkage of concrete, can in general be calculated from:

$$ w_k = l_{sk} \cdot (\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) $$

where:

- $l_{sk}$ = the influence length of the crack, some slippage in the bond between reinforcement and concrete may occur.
- $\varepsilon_{sm}$ = the mean principal tensile strain in the reinforcement in the crack’s influence length at the outer layer of the reinforcement
- $\varepsilon_{cm}$ = mean stress dependent tensile strain in the concrete at the same layer and over the same length as $\varepsilon_{sm}$.
- $\varepsilon_{cs}$ = the free shrinkage strain of the concrete (negative value).

The crackwidths may be calculated using the methods outlined in Appendix F.

703 If no documentation of the characteristic crack widths is performed in accordance with O702, then the requirements for limitation of crackwidths may be considered as satisfied if the actual stresses in the reinforcement do not exceed the values in Table O4.

O 900 Freeze/thaw cycles

901 The general requirement to freeze/thaw resistance of concrete is given in Sec.4 D306. Where appropriate the freeze/thaw resistance of the concrete is to be evaluated. This evaluation is to take account of the humidity of the concrete and the number of freeze/thaw cycles the concrete is likely to be subjected to during its lifetime. Special attention is to be given to freeze/thaw of the concrete in the splash zone.

Special frost resistant concrete may be required based on this evaluation.

O 1000 Temperature effects

1001 Thermal stresses due to temperature effects shall be taken into account when relevant. Relevant material properties shall be used. Reference is made to Sec.5 D300.

P. Design by Testing

P 100 General

101 Concrete structures can be designed either by testing or by a combination of calculation and testing. This applies to all limit states defined in B201.

102 Testing can be applied to a complete structural member (e.g. a beam), a part of a structure (e.g. a beam support) or to a detail of a structure (e.g. a fixing device to a beam). The test can cover all properties of the structure, or only certain proper-
ties, which are relevant in the particular case.

Normally, the test shall be carried out on specimens of the same size as the object for which the properties shall be tested. If the test specimen is not of the true size, the model and the scale factors shall be evaluated separately.

The rules of the standard with regard to dimensions, including the rules for detailing of reinforcement in Q and for structural details in R shall also apply to structures and parts of structures dimensioned by testing. Deviations from these rules can be undertaken, provided it is demonstrated by the test that such deviations are justified.

P 200  The test specimen

201 When determining the dimensions of the test specimen, tolerances which exceed those given C400 worst case condition shall be taken into account. More stringent tolerances may be considered.

202 The test specimen may be produced with nominal dimensions if the specified tolerances are less than the requirements to C400. If the accepted deviations have been accounted for in a conservative way, the reduced material factors in Table B1 may be used. The tolerances may be considered incorporated, if the test specimen is produced in the same form as the component to be dimensioned by testing.

203 The effect of unintended eccentricity, inclination and curvature shall be taken into account as given in A301, D103, E106 - E108.

204 When determining the material strength in the test specimen, characteristic strengths equal to those prescribed for production of the component should be aimed at.

205 If the concrete strength is governing for the test result, the concrete used in the test specimen shall have a strength approximately equal to, but not higher than, the specified characteristic concrete strength for the component in question.

206 If there are changes regarding concrete mix, constituents or concrete supplier during the production process of the component, the compressive strength and the tensile strength shall be tested when the specimens are tested and when alterations are made.

207 The test results for the material strength taken during production of the components shall not be less than those taken from the test specimen, unless it can be proved that smaller values are justifiable.

208 If the reinforcement is considered to be governing for the test result, the same type of reinforcement shall be used as is intended for the structure to be dimensioned. The yield strength - or 0.2 limit - shall be determined. If the tested strength deviates from the prescribed strength of the reinforcement, this shall be taken into account when determining the capacity of the test specimen, on the basis of the tested yield strength and the nominal characteristic yield strength of the reinforcement used.

209 In order to determine the failure load for certain failure modes it may be necessary to prevent failures caused by other failure modes with possible lower failure load. In such cases it may be necessary to modify geometry, concrete strength or amount and strength of the reinforcement. If such means have been used it shall be clearly stated in the test report. It shall be assessed whether such modifications will influence the capacity for the failure mode which is tested.

P 300  Design actions

301 The design actions shall be determined with the same load coefficients used when the capacity is determined by calculation, normally in accordance with Sec.4 Table C1.

302 The design actions shall be selected so that it is representative for the anticipated actions on the structure, if necessary through simulation.
**Q. Rules for Detailing of Reinforcement**

**Q 100 Positioning**

**Q 101** Reinforcement shall be placed in such a way that concreting will not be obstructed and so that sufficient bond anchorage, corrosion protection and fire resistance will be achieved.

The positions of ribbed bars may be designed in accordance with the given minimum spacings without regard to the ribs, but the actual outer dimensions shall be taken into account when calculating clearance for placing of reinforcement and execution of the concreting.

The positioning of reinforcement shall be designed so that the given requirements to the concrete cover can be obtained in compliance with the specified tolerances.

**Q 102** Ribbed bars may be arranged in bundles. Bundles shall not consist of more than four bars including overlapping (see Q303). Normally, the bars shall be arranged so that the bundle has the least possible perimeter.

**Q 103** When using welded mesh fabric in accordance with approved International Standard, two layers may be placed directly against each other.

**Q 104** Ducts for prestressed reinforcement may be assembled in groups when this does not obstruct the concreting of the cross section or the direct transfer of forces to the concrete. At the anchorages, special requirements for placing will apply for the various tendon systems.

**Q 105** With respect to concreting, the free distance between reinforcement units in one layer where concrete has to pass through during casting shall be no less than $D_{100} + 5\,\text{mm}$.

In addition, the distance shall normally be no less than 40 mm in Environmental classes LA and NA, 45 mm in class MA, and no less than the outer diameter of bundles or ducts.

If reinforcement is placed in more than one layer, the free distance between the layers shall be no less than 25 mm in Environmental class LA and NA, 35 mm in class MA.

The concrete cover between vertical formed surfaces and horizontal reinforcement units shall normally be no less than the diameter of the reinforcement unit and no less than $D_{100} + 5\,\text{mm}$.

When concreting in water, the distance between reinforcement bars, bundles and layers shall be no less than 100 mm and the concrete cover no less than 70 mm.

**Q 106** With respect to the conditions during concreting of structures that are cast directly on bed-rock, hard and dry clay, or firm gravel, the free distance between the horizontal reinforcement and the ground shall be no less than 50 mm.

On other types of ground at least a 50 mm thick concrete layer with strength no less than 15 MPa or an equally stable base of another material shall be specified. If concrete is used as a base, the free distance between the reinforcement and the base shall be at least 30 mm.

When concreting in water, the horizontal reinforcement shall be placed at least 150 mm above the bottom.

**Q 200 Concrete cover**

**Q 201** The concrete cover shall not be less than $\phi$ for ribbed bars and bundled bars and $2\cdot\phi$ for pre/post tensioned reinforcement.

**Q 202** Based on requirements to corrosion protection the concrete cover shall not be less than the values given in Table Q1.

<table>
<thead>
<tr>
<th>Environmental Class</th>
<th>Reinforcement sensitive to corrosion</th>
<th>Reinforcement slightly sensitive to corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>SA</td>
<td>Special considerations</td>
<td>Special considerations</td>
</tr>
<tr>
<td>MA</td>
<td>50 mm</td>
<td>40 mm</td>
</tr>
<tr>
<td>NA</td>
<td>35 mm</td>
<td>25 mm</td>
</tr>
<tr>
<td>LA</td>
<td>25 mm</td>
<td>15 mm</td>
</tr>
</tbody>
</table>

In splash zone, the minimum concrete cover shall not be less than 50 mm.

The end surfaces of tensioned reinforcement in precast elements in very aggressive environment (MA) shall be protected. Adequate corrosion protection of the end anchorage system of post-tensioned bars shall be documented for the actual environmental class.

Post-tension bars shall be placed in tight pipes injected with grout, grease etc.

**Q 300 Splicing**

**Q 301** Reinforcement bars may be spliced by lapping, couplers or welding. Splices shall be shown on the drawings.

Splices shall be staggered and as far as possible also placed in moderately strained areas of the structure. Laps may be assumed as distributed if the distance from centre to centre of the splices is greater than the development length calculated in accordance with K. However, all splices may be located in the same section in the case of:

- laps in columns and walls if the laps are located at a transverse bracing (floor), and the area of the longitudinal reinforcement is taken as not more than 2% of the gross area of the concrete
- couplers with larger capacity than the characteristic capacity of the reinforcement.
— welded splices subjected to compression only
— laps in pillars in the ground, if the area of the longitudinal reinforcement is taken as not more than 2% of the gross area of the concrete
— laps, if the lap length calculated in accordance with K107 through K109 is increased by at least 25%, and a uniformly distributed transverse reinforcement with at least 70% cross-sectional area of the spliced bar is located outside the spliced reinforcement. The transverse reinforcement shall have a part of the reinforcement, not utilized when calculating \( f_{bd} \), which gives, when calculating bond strength, an additional contribution to \( f_{bd} \) of at least 25% of the bond stress utilized in the calculation
— reinforcement which is not utilized in the calculations.

302 At laps of tensile reinforcement, necessary development length shall at least be taken equal to the necessary development length calculated in accordance with Sub-sec. K. Plain bars shall in addition have end hooks.

303 Bars and bundles that are spliced by lapping shall be placed against each other.

Areas where transfer of forces between adjacent bars, which are not placed against each other are required, can be designed in accordance with I103 through I104.

Lapped reinforcement shall have a transverse reinforcement distributed along the lap length, and this shall have a total cross-sectional area of at least 70% of the cross-sectional area of one lapped bar.

If the lapped bar has a diameter greater or equal to 16 mm, then transverse reinforcement shall be provided equally spaced over the outer third part of the lapped joint.

When the equivalent diameter is larger than 36 mm for normal aggregate concrete and 32 mm for lightweight aggregate concrete, then the bars in bundles with up to three bars shall be lapped individually in such a way that there will be no more than four bars in any section. The lap length shall be calculated in accordance with K108.

Laps in tensile members shall be staggered and the laps shall be enclosed by closed stirrups with a total cross-sectional area at least equal to twice the area of the spliced bar and with spacing no larger than 10 times the diameter of one spliced bar.

### Q 400 Bending of bars

401 Bent reinforcement shall be designed with the following set of mandrel diameters (in mm) 16, 20, 25, 32, 40, 50, 63, 80, 89, 100, 125, 160, 200, 250, 320, 400, 500 and 630.

402 Reinforcement which will be straightened around mandrel diameters less than 1.5 times the diameter of the test mandrel used when demonstrating the bending properties of the steel, or at a lower temperature than the bending properties have been documented for. The minimum mandrel diameter is given in Table Q2 for reinforcement in accordance with EN 10025. For reinforcement in accordance with other International Standards like ISO6935, ASTM and ACI, bending criteria shall be in accordance with the applicable material standard. Use of mandrel diameters less than permissible diameters given in Table Q3 requires documentation in accordance with L111.

403 The temperature in the reinforcement shall be no less than -10°C during bending.

404 For normal bent reinforcement (EN 10025), the mandrel diameters given in Table Q3 may be used without documentation in accordance with L111. For stirrups and anchorage hooks, see Q408.

### Table Q2 Permitted mandrel diameter (mm) for bending of reinforcement which satisfies the requirements of the EN 10025

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Bar Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td>B500C(1)</td>
<td>16</td>
</tr>
<tr>
<td>B500B(1), b)</td>
<td>20</td>
</tr>
<tr>
<td>B500A</td>
<td>25</td>
</tr>
<tr>
<td>G250</td>
<td>20</td>
</tr>
</tbody>
</table>

(1) Warm rolled ribbed reinforcement produced with controlled cooling can be bent with temperatures down to 20°C below zero.

### Table Q3 Permissible mandrel diameter (mm) for bending of reinforcement without compliance to L111

<table>
<thead>
<tr>
<th>Tensile strength of Reinforcement (( f_{bd} )) MPa</th>
<th>Bar Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>100</td>
</tr>
<tr>
<td>250</td>
<td>50</td>
</tr>
</tbody>
</table>

405 Bent reinforcement which will be straightened or rebent shall not have been bent around a mandrel diameter less than 1.5 times the diameter of the test mandrel used when demonstrating the ageing properties of the steel.

### Table Q4 Permissible mandrel diameter (mm) for bending of reinforcement complying with EN 10025 which shall be rebent or straightened

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Bar Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td>B500C</td>
<td>32</td>
</tr>
<tr>
<td>B500B</td>
<td>63</td>
</tr>
<tr>
<td>B500A</td>
<td>50</td>
</tr>
<tr>
<td>G250</td>
<td>40</td>
</tr>
</tbody>
</table>

For reinforcement in accordance with EN 10025 the mandrel diameters given in Table Q4 can be used.

Reinforcement which will be straightened or rebent shall not have a temperature less than -10°C for bar diameters 12 mm and less. For larger dimensions the temperature shall not be below 0°C.

Reinforcement which will be straightened or rebent shall not
be used in structural members where the reinforcement will be subjected to fatigue.

406 Reinforcement bars of type "Tempcore" or similar shall not be heat treated when bending or straightening.

407 Stirrups and anchorage hooks shall be made of reinforcement of weldable quality.

408 Verification in accordance with L111 is not required for stirrups and anchorage hooks, provided the mandrel used has a diameter not larger than 100 mm, and a transverse bar with diameter neither less than the diameter of the bent bar nor less than 0.3 times the diameter of the mandrel used is located in the bend. Regardless of the level of stresses, such reinforcement shall always have a transverse bar in the bend.

The straight part following the bend of anchorage hooks may be placed parallel to the surface if the diameter of the reinforcement bar is not larger than 16 mm. If the diameter is larger, the straight part shall be bent into the cross section, in such a way that the concrete cover does not spill by straightening the hook when the reinforcement bar is tensioned. The bend shall at least be 135°.

409 Welded reinforcement bars with welded attachments can be bent around mandrel diameters in accordance with Q401 through Q408 provided the distance between the start of the bend and the welding point is no less than four times the diameter of the bar.

410 For structures subjected to predominantly static loads, the bar can be bent at the welding point with a mandrel diameter as given in Table Q3.

411 For structures subjected to fatigue loads, the diameter of bending for welded wire fabric shall be no less than one hundred times the diameter of wire if the weld is located on the outer periphery of the bend, or five hundred times the diameter of wire if the weld is located on the inside.

412 Prestressed reinforcement shall not be bent or placed with a sharper curvature than that giving a maximum stress in the steel - caused by curvature in combination with prestress - exceeding 95% of the yield stress or of the 0.2% proof stress. Where a sharper curvature is required, the steel shall be pre-bent before being placed in the structure. This is only permitted if it is demonstrated for the steel type and dimensions in question that such pre-bending is not harmful to the performance of the steel as prestressed reinforcement.

Q 500 Minimum area of reinforcement

501 Minimum reinforcement shall be provided so, that the reinforcement in addition to securing a minimum capacity also contributes to preventing large and harmful cracks. This is achieved by transferring the tensile force present when the concrete cracks to a well distributed reinforcement.

502 In each individual case, the actual structure and state of stresses shall be taken into consideration when determining the minimum reinforcement.

503 For Structures exposed to pressure from liquid or gas shall the numerical value of \( f_{sk} \) be replaced by \( (f_{sk} + 0.5 \cdot p_{w}) \) in the formulae for calculating the required amount of minimum reinforcement in accordance with R. Where \( p_{w} \) = liquid or gas pressure.

504 Through all construction joints a minimum reinforcement no less than the minimum reinforcement required for each of the parts concreted together shall normally be specified.

505 In structures in a severely aggressive environment and in structures where tightness is particularly important a well distributed reinforcement crossing all concreting joints shall be specified. This should have a cross section that is at least 25% larger than the required minimum reinforcement for the parts that are concreted together.

506 In structures with prestressed reinforcement without continuous bond, no less reinforcement of ribbed bars shall be specified than required as minimum reinforcement.

Minimum reinforcement parallel to the direction of prestressed reinforcement may nevertheless be omitted in areas with positive span moment provided no cracks develop in the concrete in the serviceability limit state.

507 In slabs the prestressing units shall not have larger spacing than six times the thickness of the slab.

R. Structural Detailing Rules

R 100 Slabs/plates

101 A structure or structural member shall be considered as a slab if the width of the cross section is larger than or equal to four times the thickness.

102 In general, the total depth of the cross-sectional, \( h \), shall be no less than \( L_{z}/135 \)

where \( L_{z} \) is the distance between zero moment points

103 For two-way slab systems, the lesser \( L_{z} \) for the two span directions shall apply, and for cantilever slabs;

\[ L_{z} = 2 \cdot L \]

Where \( L \) is the length of the cantilever.

104 Transverse to the main reinforcement and directly on this, a continuous minimum reinforcement shall be placed. The reinforcement shall have a total cross-sectional area equal to

\[ A_{c} \geq 0.25 \cdot k_{w} \cdot A_{o} \cdot f_{yk} / f_{sk} \]

where:

- \( k_{w} = 1.5 - \frac{h_{1}}{h} \geq 1.0 \)
- \( h \) = the total depth of the cross section
- \( h_{1} = 1.0 \, m \)
- \( f_{yk} = \) defined in Q503.

At inner supports this reinforcement may be distributed with one half in the upper face and one half in the lower face.

105 In structures where special requirements to limitation of crackwidths apply, the minimum reinforcement should be at least twice the value given above.

The spacing between the secondary reinforcement bars in the same layer shall not exceed three times the slab thickness nor exceed 500 mm.

106 In the span and over the support, a main reinforcement no less than the required minimum reinforcement shall be specified on the tension face. In the span and over the support, the spacing of the main reinforcement bars shall not exceed twice the slab thickness nor exceed 300 mm. When curtailing the main reinforcement, the spacing may be increased to four times the thickness or 600 mm.

107 A portion of the main reinforcement with a cross-sectional area no less than the requirement for minimum reinforcement shall be extended at least a length \( d \) beyond the calculated point of zero moment, where \( d \) is the distance from the centroid of the tensile reinforcement to the outer concrete fibre on the compression side. For reinforcement over the support the distance between support and point of zero moment shall not be assumed less than the distance calculated according to the theory of elasticity.

108 Of the maximum main reinforcement between supports the following portion shall be extended beyond the theoretical support:

- 30% at simple support
- 25% at fixed support or continuity

109 At simple end support the main reinforcement shall be...
anchored for a force which at least corresponds to the capacity of the required minimum reinforcement.

110 In two-ways slab systems, these rules apply for both directions of reinforcement.

111 At end supports a top reinforcement which at least is equal to the required minimum reinforcement shall normally be provided, even if no restraint is assumed in the calculations, unless the slab end support is actually fully free. For one-way slab systems, this top reinforcement may be omitted at end supports parallel to the main reinforcement.

112 As for inner supports the transverse reinforcement which is calculated in accordance with R105-R106 may be distributed with one half in the upper face and one half in the lower face.

113 Normally no stirrups or other types of shear reinforcement are required for slabs. To be taken into account in the shear capacity the shear reinforcement shall have a cross-sectional area at least corresponding to (in mm²/mm²)

\[ A_{\text{sk}} \geq 0.2 \cdot \frac{f_{\text{ik}}}{f_{\text{sk}}} \]

where:

- \( f_{\text{ik}} \) defined in Q503.

R 200 Flat slabs

201 Flat slabs are slabs with main reinforcement in two directions and supporting columns connected to the slab. The head of the column may be enlarged to a capital. The slab may be made with or without drop panel above the capital.

The slab shall have a minimum thickness of

\[ (l – 0.7 \cdot b_k) / 30 \geq 130 \text{ mm for slabs without drop panel} \]
\[ (l – 0.7 \cdot b_k) / 35 \geq 130 \text{ mm for slabs with drop panel} \]

1 is the distance between the centre lines of the columns.

- \( b_k \) is the width of the capital at the underside of the slab or the strengthening.
- \( b_k \) shall not be entered with a lower value than the width of the column in the span direction or with a larger value than the value corresponding to a 60° inclination of the face of the capital to the horizontal plane.

202 In each of the two main directions, the slab reinforcement shall have a total cross-sectional area at least equal to

\[ A_s \geq 0.25 \cdot k_w \cdot \frac{A_c \cdot f_{\text{tk}}}{f_{\text{sk}}} \]

\( k_w \) is in accordance with R105 – R106

\[ f_{\text{ik}} \text{ defined in Q503.} \]

203 At the middle of the span, the spacing of bars shall not exceed 300 mm.

204 Above columns in flat slabs with prestressed reinforcement without continuous bond, a non-prestressed reinforcement in the upper face shall be provided with an area no less than the required area in accordance with this clause, regardless of the state of stresses.

R 300 Beams

301 The cross-sectional depth \( h \) shall normally be no less than

\[ L_1 / 35 \]

\( L_1 \) is the distance between points of contraflexure. For cantilever beams, \( L_1 = 2 \cdot L \) and \( L \) is the length of the cantilever.

302 Rectangular beams should normally have a reinforcement at both the tension and compression face, at least equal to

\[ A_s \geq 0.35 \cdot k_w \cdot b \cdot h' \cdot \frac{f_{\text{ik}}}{f_{\text{sk}}} \]

where:

- \( k_w \) = as given in R105 – R106
- \( f_{\text{ik}} \) defined in Q503.

303 In beams with flanges, a minimum reinforcement shall be specified for the web as for rectangular beams.

Flanges subjected to tension shall be provided with additional reinforcement in accordance with the following formula:

\[ A_s \geq A_{\text{cf}} \cdot \frac{f_{\text{ik}}}{f_{\text{sk}}} \]

where:

- \( A_{\text{cf}} \) = the effective cross section area of the flange
- \( h_f \cdot b_{\text{eff}} \) = the part of the slab width which according to Sec.6 A400 is assumed to be effective when resisting tensile forces
- \( h_f \) = the thickness of the flange (the slab)
- \( f_{\text{ik}} \) = defined in Q503.

In beams where the neutral axis is located near the flange, this quantity may be reduced to

\[ A_s \geq 0.5 \cdot h_f \cdot b_{\text{eff}} \cdot \frac{f_{\text{ik}}}{f_{\text{sk}}} \]

In flanges subjected to compression, the requirement for minimum reinforcement is

\[ A_s \geq 0.25 \cdot A_{\text{cf}} \cdot \frac{f_{\text{ik}}}{f_{\text{sk}}} \]

304 In beams, the following fraction of the maximum main reinforcement in the span shall be extended beyond the theoretical support:

- 30% at simple support
- 25% at fixed support or continuity

In both cases at least 2 bars shall be extended.

At least 30% of the maximum required tensile reinforcement over supports shall either be extended a distance corresponding to the anchorage length beyond the point where calculated tension in the reinforcement is equal to zero, or be bent down as inclined shear reinforcement.

305 T-beams which are parallel to the main reinforcement of the slab shall have a transverse top reinforcement above the beam no less than half of the main reinforcement of the slab in the middle of the span. This top reinforcement shall be extended at least 0.3 times the span length of the slab to both sides of the beam.

306 Normally stirrups shall be provided along the entire length of a beam irrespective of the magnitude of the acting shear forces. This stirrup reinforcement shall have a cross-sectional area corresponding to

\[ A_s \geq 0.2 \cdot A_c \cdot \frac{f_{\text{ik}} \cdot \sin \alpha}{f_{\text{sk}}} \]

where:

- \( A_c \) = the concrete area of a longitudinal section of the beam web
- \( \alpha \) = the angle between stirrups and the longitudinal axis of the beam. The angle shall not be taken less than 45°
- \( f_{\text{ik}} \) = defined in Q503.

The tensile strength \( f_{\text{ik}} \) shall not have a lower value than 2.55 MPa. The distance between the stirrups shall neither exceed 0.6 h' nor 500 mm whatever is the smaller. The stirrups shall enclose all tensile reinforcement bars, if necessary by means of spliced stirrups. In beams with flanged cross section transverse reinforcement outside the longitudinal reinforcement may be assumed to enclose the longitudinal reinforcement. A longitudinal reinforcement bar shall be placed in all the corners of the stirrups and in any anchorage hooks. The diameter of this longitudinal bar shall be no less than the diameter of the stirrup.

If the depth of the beam exceeds 1 200 mm, an additional longitudinal surface reinforcement on the faces of the beam web shall be provided. This reinforcement shall be no less than the required minimum stirrup reinforcement.

In prestressed concrete, the distance between the stirrups may be up to 0.8 h' if the capacity is sufficient without shear rein-
forcement, but no larger than 500 mm. In those parts of pre-tressed beams which have compression in the entire cross section in the ultimate limit state, minimum stirrup area may be reduced to 70% of the above requirements.

In wide beams, the distance between stirrups or legs of stirrups measured perpendicularly to the longitudinal axis shall not exceed the depth of the beam.

307 Requirements to minimum stirrup reinforcement may be waived for ribbed slabs with ribs in one or two directions, monolithically connected to a top slab. The following requirements shall be satisfied:

— the width of the ribs shall be at least 60 mm and the depth shall not exceed 3 times the minimum width
— clear distance between ribs shall not exceed 500 mm
— the thickness of the top slab shall be at least 50 mm and shall have reinforcement at least equal to the required minimum reinforcement for slabs.

For ribbed slabs that do not satisfy these requirements the rules for beams shall apply.

308 Compression reinforcement bars shall be braced by stirrups with spacing not exceeding 15 times the diameter of the compression reinforcement bar.

R 400 Columns

401 The dimensions of columns shall be no less than:

— 40 000 mm² as gross cross-sectional area
— 150 mm as minimum sectional dimension for reinforced columns
— 200 mm as minimum sectional dimension for un-reinforced columns

402 Reinforced columns shall not have less total cross-sectional area of longitudinal reinforcement than the larger of:

\[ 0.01 \cdot A_c + 0.2 \cdot A_c \cdot f_{ck} / f_{sk} \]

403 The minimum reinforcement shall be symmetrical. The diameter of longitudinal reinforcement shall be no less than 10 mm. If the column has a larger cross section than structurally required the minimum reinforcement may be determined by the structurally required cross section.

404 If the longitudinal reinforcement in the column is not extended into the structure below, splicing bars shall be extended up into the column with a total area at least equal to the required reinforcement for the column.

405 If bars at the top of a column are bent towards the centre to allow extension into a column with a smaller section located above, the longitudinal inclination shall not exceed 1.6, and the point of bend shall be located minimum 100 mm above the column top.

406 If a larger area of longitudinal reinforcement than 2% of the cross-sectional area of the column is utilized, lapped splicing at transverse bracings shall be limited to a fraction corresponding to 2% of the area of the column. Spliced and continuous bars shall be symmetrically distributed over the cross section of the column.

407 The position of the longitudinal reinforcement shall be secured by stirrups enclosing the reinforcement at a spacing not exceeding 15 times the diameter of the longitudinal reinforcement. In addition the longitudinal reinforcement shall be secured at any points of the bend. Required compressive reinforcement shall not be located further away from corner of supporting transverse reinforcement, stirrup or hook than 15 times the diameter of the supporting bar.

408 If concrete of grade C65 or higher is used, the spacing of the links shall be reduced to 10 times the diameter of the longitudinal reinforcement, and the stirrups shall be ribbed bars with diameter at least equal to 10 mm.

409 In spiral reinforced columns the spiral shall be bent mechanically and shall have circular form in sections perpendicular to the direction of the force. The ascent per winding shall not exceed 1:7 of the core diameter. The clear distance between spiral windings shall not exceed 60 mm nor be less than 35 mm. The spiral reinforcement shall extend through the entire length of the column and is only permitted to be omitted where the column is embedded in a reinforced concrete slab on all sides. Splicing of spiral reinforcement between floors of concrete shall be performed as welded splices. When terminating a spiral, the spiral bar shall be bent into the core and shall there be given an anchorage length at least equal to 25 times the diameter of the bar. Plain bars shall in addition be terminated by a hook. The base for a spiral reinforced column shall be made strong enough to resist the increased stress in the core section. If the force transfer is not secured in another way, a sufficiently large transition spiral of height at least equal to the core diameter of the columns shall be placed in the column base.

R 500 Walls

501 Reinforced walls shall have horizontal reinforcement with cross-sectional area corresponding to:

\[ A_h \geq 0.6 \cdot A_c \cdot f_{ck} / f_{sk} \]

for horizontal reinforcement in external walls

\[ A_h \geq 0.3 \cdot A_c \cdot f_{ck} / f_{sk} \]

for internal walls, horizontal and vertical reinforcement

\[ A_h \geq 0.6 \cdot A_c \cdot f_{ck} / f_{sk} \]

for reinforcement in shell type structures in both directions.

where \( f_{sk} \) is defined in Q503.

502 In structures where strong limitations of the crack widths is required, the horizontal reinforcement should be at least twice the values given above. The horizontal reinforcement may be reduced if the wall is free to change its length in the horizontal direction and if it can be demonstrated by calculations that the chosen reinforcement can resist the forces caused by loads, shrinkage and temperature changes with acceptable crack widths. The spacing between horizontal bars in same layer shall not exceed 300 mm.

503 The spacing between vertical bars in the same layer shall not exceed 300 mm. At openings in walls, in addition to the minimum reinforcement given above at least 2 ribbed bars of 12 mm diameter shall be provided parallel to the edges or diagonally at the corners, and the anchorage lengths to both sides shall be at least 40 times the diameter of the bar.

504 In walls which primary is exposed to bending caused by local pressure load, the requirements regarding minimum reinforcement in plates in accordance with R100 shall apply.

R 600 Reinforced foundations

601 Foundations shall have thickness no less than 10 times the diameter of the reinforcement bar or 200 mm, whichever is the smaller.

602 Tensile reinforcement at the bottom of a column foundation may be uniformly distributed over the full width if the width does not exceed 5 times the diameter of the column measured in the same direction. If the width of the foundation is larger, 2/3 of the tension reinforcement shall be located within the middle half of the foundation unless a more correct distribution is verified.

603 Foundations shall be considered as beams or slabs with respect to minimum reinforcement, reference is made to 100 - 300.

R 700 Prestressed structures

701 The structures shall be designed, formed and constructed so that the deformations required according to the calculations are possible when applying the prestressing forces. The influence of creep shall be considered when necessary.
702 At the anchorages, the concrete dimensions shall be sufficient to ensure that a satisfactory introduction and transfer of the anchorage forces is obtained. The documentation shall be based on calculations or tests for the anchorage in question.

703 Directly inside anchorages for prestressed reinforcement, extra reinforcement in the shape of a welded wire fabric perpendicular to the direction of the force or a circular reinforcement should be provided. If the stress in the contact surface between anchorage member and concrete exceeds $f_{ak}$, this shall be applied. The quantity of this extra reinforcement shall be documented by tests or calculations for the type of anchorage in question.

S. Corrosion Control

S 100 General

101 Requirements to corrosion protection arrangement and equipments are generally given in DNV-OS-C101 Sec.10. Special evaluations relevant for offshore concrete structures are given herein.

102 Fixed and floating concrete structures associated with production of oil and gas comprises permanent structural components in Carbon-steel that require corrosion protection, both topside and in shafts. In addition, shafts and caissons may contain mechanical systems such as piping for topside supply of seawater and for ballast, crude oil storage and export. These piping systems are exposed to corrosive environments both internally and externally. Riser and J-tubes may be routed within or outside shafts. Drill shafts contain conductors and support structures with large surface areas that are also to be protected from corrosion. Internal corrosion control of risers, tubing and piping systems containing fluids other than seawater is, however, not covered by this Standard.

103 Concrete rebars and prestressing tendons are adequately protected by the concrete itself, (i.e. provided adequate coverage and type/quality of the aggregate). However, rebar portions freely exposed to seawater in case of concrete defects, and embedment plates, penetration sleeves and various supports which are freely exposed to seawater or the marine atmosphere will normally require corrosion protection.

S 200 Corrosion zones and environmental parameters

201 A fixed concrete structure will encounter different types of marine corrosion environments. These may be divided into corrosion zones as given in Table S1.

<table>
<thead>
<tr>
<th>Table S1 Corrosion zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>External zones</td>
</tr>
<tr>
<td>External atmospheric zone</td>
</tr>
<tr>
<td>Splash zone</td>
</tr>
<tr>
<td>External submerged zone</td>
</tr>
<tr>
<td>Buried zone</td>
</tr>
</tbody>
</table>

202 The splash zone is the external part of the structure being intermittently wetted by tidal and wave action. Intermediate zones include shafts and caissons that are intermittently wetted by seawater during tidal changes and dampened wave action, or during movement of crude oil/ballast water interface level. The external/internal atmospheric zones and the submerged zones extend above and below the splash/internal zones respectively. The buried zone includes parts of the structure buried in seabed sediments or covered by disposed solids externally or internally.

203 The corrosivity of the corrosion zones varies as a function of geographical location; temperature being the primary environmental parameter in all zones. In the atmospheric zones, the frequency and duration of wetting ("time-of-wetness") is a major factor affecting corrosion. In the external atmospheric zone, the corrosive conditions are typically most severe in areas sheltered from direct rainfall and sunlight but freely exposed to sea-spray and condensation that facilitates accumulation of sea salts and moisture with a resulting high time-of-wetness. A combination of high ambient temperature and "time-of-wetness" creates the most corrosive conditions.

204 In the atmospheric zones and the splash/intermediate zones, corrosion is primarily governed by atmospheric oxygen. In the external submerged zone and the lower part of the splash zone, corrosion is mostly affected by a relatively thick layer of marine growth. Depending on the type of growth and the local conditions, the net effect might be either to enhance or retard corrosion attack. In the buried and internal submerged zones (i.e. seawater flooded compartments), oxygen in the seawater is mostly depleted by bacterial activity. Similarly, steel surfaces in these zones, and in the external submerged zone, are mostly affected by biological growth that retards or fully prevents access of oxygen by diffusive mass transfer. Although this could retard corrosion, corrosive metabolises from bacteria can offer an alternative corrosion mechanism.

205 Corrosion governed by biologic activity (mostly bacteria) is referred to as MIC (microbiologically influenced corrosion). For most external surfaces exposed in the submerged and buried zones, as well as internal surfaces of piping for seawater and ballast water, corrosion is primarily related to MIC.

S 300 Forms of corrosion and associated corrosion rates

301 Corrosion damage to uncoated C-steel in the atmospheric zone and in the splash/intermediate zones associated with oxygen attack is typically more or less uniform. In the splash zone and the most corrosive conditions for the external atmospheric zone (i.e. high time of wetness and high ambient temperature), corrosion rates can amount to 0.3 mm per year, and for internally heated surfaces in the splash zone even much higher (up to of the order of 3 mm per year. In more typical conditions for the external atmospheric zone and for internal atmospheric zones, the steady-state corrosion rate for C-steel (i.e. as uniform attack) is normally around 0.1 mm per year or lower. In the submerged and buried zones, corrosion is mostly governed by MIC causing colonies of corrosion pits. Welds are often preferentially attacked. Corrosion as uniform attack is unlikely to significantly exceed about 0.1 mm per year but the rate of pitting may be much higher; 1 mm per year and even more under conditions favouring high bacterial activity (e.g. ambient temperature of 20°C to 40°C and access to organic material, including crude oil).

302 In most cases, the static load carrying capacity of large structural components is not jeopardized by MIC due to its localized form. The same applies to the pressure containing capacity of piping systems. However, MIC can readily cause leakage in piping by penetrating pits, or initiate fatigue cracking of components subject to cyclic loading.

303 Galvanic interaction (i.e. metallic plus electrolytic coupling) of Carbon-steel to e.g. stainless steel or copper base alloys may enhance the corrosion rates given in F301. On external surfaces in the submerged and buried zones, galvanic corrosion is efficiently prevented by cathodic protection. In the atmospheric and intermediate zones, and internally in piping systems, galvanic corrosion shall be prevented by avoiding metallic or electrolytic contact of non-compatible materials.

304 Very high strength steels ($f_{ak} > 1 200$ MPa) and certain high strength aluminium, nickel and copper alloys are sensitive to stress corrosion cracking in marine atmospheres. If susceptible materials shall be used, cracking should be prevented by use of suitable coatings.

S 400 Cathodic protection

401 For details of design of cathodic protection systems, see DNV-OS-C101 Sec.10 Č. “Cathodic Protection".
SECTION 7
CONSTRUCTION

A. General

A 100 Application

101 This Chapter apply to the fabrication and construction of reinforced and prestressed concrete structures and structural parts or assemblies in concrete.

102 Fabrication and construction of assemblies not adequately covered by this Standard shall be specially considered.

A 200 Codes and standards

201 Codes and Standards other than those stated within this Standard may be accepted as an alternative, or as a supplement, to these Standards. The basis for such acceptance is stated in Sec. 1.

A 300 Scope

301 The requirements of this section apply to material testing, formwork, reinforcement, concrete production, concrete coating, prestressing systems and repairs during construction of concrete structures.

B. Definitions

B 100 Terms

101 In the context of these Rules the term “fabrication and construction” is intended to cover fabrication and construction workings from initial fabrication to end of design life of the installation or component thereof, as applicable.

102 The term Site used within the context of these Rules is to mean the place of construction of the concrete structure (placing of reinforcement, formwork assembly and pouring of concrete into the formworks or assembling of precast concrete units).

C. Documentation

C 100 General

101 As the basis for fabrication and construction activities the following documentation, as applicable, is to be approved:

— drawings showing structural arrangement and dimensions with specifications and data defining all relevant material properties
— relevant fabrication and construction specifications
— details of welded attachments/connections
— drawings and description of the reinforcement and prestressing system
— requirements to extent, qualification and results of fabrication and construction, inspection, testing and examination procedures
— specifications for the corrosion protection systems
— any limitations/tolerances applicable as a result of design assumptions.

102 Assumptions made during the design of the structure influencing the fabrication and construction activities shall be documented.

103 Relevant documentation from the fabrication and construction required for safe operation of the structure is to be readily available on the Installation.

Such documentation is to give sufficient information to evaluate damages and subsequent possible repairs and modifications.

D. Quality Control - Inspection, Testing and Corrective Actions

D 100 General

101 Supervision and inspection shall ensure that the works are completed in accordance with this Standard and the provisions of the project specification.

102 Quality assurance and quality control. A quality management system based on the requirements of EN ISO 9001 shall be applied to the following phases:

— organisation
— design and procurement
— equipment, shop manufacture
— equipment, storage and transport
— construction, (i.e. earthworks, construction, towing, installation, backfilling, civil works and structural steelwork, storage tanks, pressure vessels, separators, furnaces, boilers, pumps, above ground piping including supports, underground piping, instrumentation, electricity, cathodic protection, paint work, thermal insulation, fire proofing etc). The content in brackets will vary dependent on the actual structure/plant under construction.

A specific quality control programme including inspection and tests shall be set up to monitor the quality throughout the different phases of the design, fabrication and construction.

D 200 Inspection Classes

201 In order to differentiate the requirements for inspection according to the type and use of the structure, this Standard defines three inspection classes:

IC 1: Simplified inspection
IC 2: Normal inspection
IC 3: Extended inspection.

202 The inspection class to be used shall be stated in the project specification.

203 Inspection class may refer to the complete structure, to certain members of the structure or to certain operations of execution.

204 In general, inspection class 3, ”Extended inspection”, applies for Offshore Concrete Structures. Inspection class 1 ‘Simplified inspection’ shall not be used for concrete works of structural importance.

D 300 Inspection of materials and products

301 The inspection of the properties of the materials and
products to be used in the works shall be as given in Table D1.

### Table D1 - Inspection of materials and products

<table>
<thead>
<tr>
<th>Subject</th>
<th>Inspection Class 1</th>
<th>Inspection Class 2</th>
<th>Inspection Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Simplified</td>
<td>Normal</td>
<td>Extended</td>
</tr>
<tr>
<td>Materials for formwork</td>
<td>Not required</td>
<td>In accordance with project specification</td>
<td></td>
</tr>
<tr>
<td>Reinforcing steel</td>
<td>In accordance with ISO 6935 and relevant national standards</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prestressing steel</td>
<td>Not applicable</td>
<td>In accordance with ISO 6934</td>
<td></td>
</tr>
<tr>
<td>Fresh concrete: ready mixed or site mixed</td>
<td>In accordance with this Standard</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other items 1)</td>
<td>In accordance with project specification and this standard</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Precast elements</td>
<td>In accordance with this Standard</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inspection report</td>
<td>Not required</td>
<td>In accordance with this Standard</td>
<td></td>
</tr>
</tbody>
</table>

1) Could be items such as embedded steel components.

### D 400 Inspection of execution

#### 401 General

Inspection of execution according to this Standard shall be carried out as given in Table D2 unless otherwise stated in the project specification.

### Table D2 - Inspection of execution

<table>
<thead>
<tr>
<th>Subject</th>
<th>Inspection Class 1</th>
<th>Inspection Class 2</th>
<th>Inspection Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scaffolding, formwork and falsework</td>
<td>Random checking</td>
<td>Major scaffolding and formwork to be inspected before concreting</td>
<td>All scaffolding and formwork shall be inspected before concreting</td>
</tr>
<tr>
<td>Ordinary reinforcement</td>
<td>Random checking</td>
<td>Major reinforcement shall be inspected before concreting</td>
<td>All reinforcement shall be inspected before concreting</td>
</tr>
<tr>
<td>Prestressing reinforcement</td>
<td>N/A</td>
<td>All prestressing components shall be inspected before concreting, threading, stressing. Materials to be identified by appropriate documentation</td>
<td></td>
</tr>
<tr>
<td>Embedded items</td>
<td>According to project specification</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Erection of precast elements</td>
<td>N/A</td>
<td>Prior to and at completion of erection</td>
<td></td>
</tr>
<tr>
<td>Site transport and casting of concrete</td>
<td>Occasional checks</td>
<td>Basic and random inspection</td>
<td>Detailed inspection of entire process</td>
</tr>
<tr>
<td>Curing and finishing of concrete</td>
<td>Occasional checks</td>
<td>Occasional checks</td>
<td>Regular inspection</td>
</tr>
<tr>
<td>Stressing and grouting of prestressing reinforcement</td>
<td>N/A</td>
<td>Detailed inspection of entire process, including evacuation of stressing records prior to cutting permission</td>
<td></td>
</tr>
<tr>
<td>As-built geometry</td>
<td>N/A</td>
<td>According to project specification</td>
<td></td>
</tr>
<tr>
<td>Documentation of inspection</td>
<td>N/A</td>
<td>Required</td>
<td></td>
</tr>
</tbody>
</table>

#### 402 Inspection of falsework and formwork

Before casting operations start, inspections according to the relevant inspection class shall include:

- geometry of formwork
- stability of formwork and falsework and their foundations
- tightness of formwork and its parts
- removal of detritus such as saw dust, snow and/or ice and remains of tie wire and debris from the formwork etc. from the section to be cast
- treatment of the faces of the construction joints
- wetting of formwork and/or base
- preparation of the surface of the formwork
- openings and blockouts.

The structure shall be checked after formwork removal to ensure that temporary inserts have been removed.

#### 403 Inspection of reinforcement

Before casting operations start, inspections according to the relevant inspection class, shall confirm that:

Reinforcement is not contaminated by oil, grease, paint or other deleterious substances;

- The reinforcement shown on the drawings is in place, at the specified spacing
- The cover is in accordance with the specifications
- Reinforcement is properly tied and secured against displacement during concreting
- Space between bars is sufficient to place and compact the concrete.

After concreting, the starter bars at construction joints shall be checked to ensure that they are correctly located.

#### 404 Inspection of prestressing works

Before casting operations start, inspections shall verify:

- the position of the tendons, sheaths, vents, drains, anchorages and couplers in respect of design drawings. (including the concrete cover and the spacing of tendons)
- the fixture of the tendons and sheath, also against buoyancy, and the stability of their supports
- that the sheath, vents, anchorages, couplers and their sealing are tight and undamaged
- that the tendons, anchorages and/or couplers are not corroded
- the cleanliness of the sheath, anchorages and couplers.

Prior to tensioning or prior to releasing the pretension force, the actual concrete strength shall be checked against the strength required.

The relevant documents and equipment according to the tensioning programme shall be available on site.

The calibration of the jacks shall be checked. Calibration shall also be performed during the stressing period if relevant.
Before grouting starts, the inspection shall include:
- preparation/qualification tests for grout
- the results of any trial grouting on representative ducts
- ducts open for grout through their full length and free of harmful materials, e.g. water and ice
- vents prepared and identified
- materials are batched and sufficient to allow for overflow.

During grouting, the inspection shall include:
- conformity of the fresh grout tests, e.g. fluidity and segregation
- the characteristics of the equipment and of the grout
- the actual pressures during grouting
- order of blowing and washing operations
- precautions to keep ducts clear
- order of grouting operations
- actions in the event of incidents and harmful climatic conditions
- the location and details of any re-injection.

405 Inspection of the concreting operations

The inspection and testing of concreting operations shall be planned, performed and documented in accordance with the inspection class as shown in Table D3.

The inspection class for concreting operations shall be IC 3. Extended inspection, unless otherwise specified in the project specification.

Different structural parts in a project may be allocated to different inspection classes depending on the complexity and the importance in the completed structure.

The planning for all construction stages is to ensure that there is adequate time for the concrete to harden sufficiently to support the loads imposed.

Due consideration is to be given to access and time required for adequate survey and inspection as the construction proceeds.

406 Inspection of precast concrete elements

When precast concrete elements are used, inspection shall include:
- visual inspection of the element at arrival at site;
- delivery documentation
- conditions of the element prior to installation
- conditions at the place of installation, e.g. supports
- conditions of element, any protruding rebars, connection details, position of the element, etc., prior to jointing and execution of other completion works.

407 Actions in the event of a non-conformity

Where inspection reveals a non-conformity, appropriate action shall be taken to ensure that the structure remains fit for its intended purpose. As part of this, the following should be investigated:
- Implications of the non-conformity on the execution and the work procedures being applied
- Implications of the non-conformity on the structure, safety and functional ability
- Measures necessary to make the element acceptable
- Necessity of rejection and replacement of non-conforming elements.

Documentation of the procedure and materials to be used shall be approved before repair or corrections are made.

E. Construction Planning

101 Prior to Construction, procedures for execution and control of all construction activities shall be prepared in order to ensure that the required quality is obtained and documented.

102 Procedures detailing the construction sequences, testing and inspection activities shall be prepared. Sufficient delivery of materials and storage capacity is to be ensured to accommodate the anticipated demand for any continuous period of casting.

103 The planning for all construction stages is to ensure that there is adequate time for the concrete to harden sufficiently to support the loads imposed.

104 Due consideration is to be given to access and time required for adequate survey and inspection as the construction proceeds.

105 Constructional operations concerning transportation and installation operations shall be detailed in special procedures prepared in accordance with the requirements given in Sec.3.

F. Materials and Material Testing

F 100 General

101 Constituent materials, reinforcement and prestressing systems used in construction, as well as fresh and hardened concrete and grout are to satisfy the relevant requirements given in Sec.4.

102 Testing of materials is to be performed prior to and during construction to confirm quality of the materials and to ensure that the specified properties are obtained.

103 Testing of materials is to be performed in accordance with the requirements of Sec.4. The testing is to be conducted with calibrated and tested instruments and equipment.

104 Testing at independent, recognized laboratories may be required.

105 Records of all performed testing shall be kept for later inclusion in the Construction Records.

F 200 Constituent Materials

201 Storage and handling of constituent materials shall be in accordance with recognized good practice. The materials shall be protected from detrimental influences from the environment.

202 Cement is to be delivered with mill certificate in accordance with Sec.4. Different batches of cement are, as far as practicable, to be stored in different silos, such that the results of the on site testing can be referred to specific batches.

203 Testing of cement on site is to be performed on a random basis during the construction period. The frequency of the sampling is to be specified based on experience and is to be approved by client/verification authority prior to start of construction. The sampling is to be representative for the delivered cement. An increased frequency of sampling may be required in the following cases:

a) Change of supplier.

b) Change of type/grade.
c) Change of requirements to concrete properties.
d) Unsatisfactory test results.
e) Unsatisfactory storage conditions.
f) Other information or events that may justify an increased sampling.

204 Testing of cement is at least to be performed to establish the following properties:
- fineness
- initial and final set
- oxide composition
- mortar strength.

Testing is to be performed as specified in Sec.4, and the test results are to satisfy the requirements in Sec.4. Cement failing to meet the requirements, is not to be used.

205 Aggregates shall be tested upon delivery at site. If different sources of aggregates are used the properties shall be established for each source. The following properties shall be established:
- particle size distribution (grading) including silt content
- content of organic matter
- density and specific gravity
- strength in standard mix of concrete and mortar
- petrographical composition and properties that may affect the durability of the concrete.
- water content.

206 Aggregates delivered to the site shall be stored separately and such that the aggregates are protected from accumulation of water and other harmful influences of the environment, and have markings identifying their contents.

207 Testing of aggregates is to be performed on a regular basis during the construction period. The frequency of the sampling is to be specified based on the quality and consistency of the supply as well the concrete production volume, and is to be approved prior to start of construction. An increase in the test frequency may be required when tests are not giving satisfactory results, upon “a change of supplier” or if changes in the uniformity of the supply are observed.

208 The water source(s) shall be investigated for the suitability and dependability of the water supply. The water is not to contain organic impurities, detrimental salts or other matter that may have harmful or adverse effects on fresh or hardened concrete as well as reinforcement. The supply is to be sufficient and dependable enough to ensure adequate supply during any foreseen extensive production period.

209 The quality of mixing water is to be documented by testing at intervals adjusted in each case to type of water supply (public of other) as agreed between the Client and the Society.

210 Admixtures delivered to a site for mix shall be furnished with test reports confirming the specified properties. Handling and storage of admixtures shall be in accordance with the supplier’s recommendations.

211 The effect of the admixtures on concrete shall be tested at intervals on site in terms of the following properties:
- consistency, e.g. at 5 and 30 minutes after mixing
- water requirement for a given consistence
- shrinkage/swelling
- strength in compression and tension (bending) at 7, 28 and 91 days.

F 300 Reinforcement and prestressing system components

301 All reinforcement is to be delivered to the construction site with appropriate certificates confirming compliance with the specified requirements (see Sec.4). The steel is to be adequately marked for identification upon arrival. The marking is to be maintained to establish traceability until actual use in the structure.

302 Reinforcement is to be stored in a manner which prevents harmful corrosion and erasure of marking. Reinforcement of different grades and dimensions shall be stored separately.

303 Components of the prestressing system shall be delivered with appropriate certificates confirming compliance with the specified requirements. (see Sec.4). The marking is to be maintained to establish traceability until actual use in the structure.

304 Components for prestressing systems, including cables, shall be stored in a dry environment without any danger of harmful corrosion. They shall be given additional protection with water soluble, protective oil. The oil is to be documented not to adversely affect the bond to the grout or alternately shall be cleaned prior to use.

305 Regular spot checks shall be performed on site to ensure:
- proper traceability, marking and storage of reinforcement and components of prestressing system
- that bending of bars is performed within approved diameters and temperatures.

306 Procedures for welding of reinforcement steel and welder’s qualification are documented in accordance with the requirements of Sec.4.

All welds shall be 100% visual examined. Samples of welding shall be taken and tested at regular intervals. Comprehensive documentation may be required by the client/verification authorities for critical welds.

307 Testing of mechanical splices in reinforcement are to comprise:
- Prior to construction; 3 tensile tests of the splices
- During construction; tensile tests of 1% of all splices performed.

308 Testing of prestressing steel prior to its use is to be performed at regular intervals. The intervals shall be part of the procedure and the result of the testing shall be documented.

309 Testing of components for the prestressing system and testing and calibration of stressing equipment may be required and shall be documented.

F 400 Production testing

401 Prior to start of construction, the properties of the intended concrete mix shall be verified by testing of samples from series of trial mixes. The testing and test method is to be in accordance with the requirements of Sec.4.

402 The following properties shall be documented:
- mix proportions and the resulting consistence, bleeding and air content
- compressive strength
- setting times and strength development
- modulus of elasticity in compression
- permeability of hardened concrete
- durability in accordance with the approved specification requirements
- effect of admixtures.

403 During production, the concrete is to be tested regularly for strength, air content, consistency, temperature and density,
as given in Table F1.

<table>
<thead>
<tr>
<th>Table F1 Frequency of Production Testing of Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>Strength</td>
</tr>
<tr>
<td>Air content, Temperature and consistency</td>
</tr>
<tr>
<td>Density</td>
</tr>
</tbody>
</table>

Each sample for strength testing, taken from one batch at the form after transportation, is to comprise of at least 4 test specimens unambiguously marked for identification. The collection, curing and testing is to be performed in accordance with an approved specification.

404 Until the uniformity of a concrete has been demonstrated, higher rates of testing may be required. During continuous production, rates of testing may be reduced as agreed with parties involved.

405 Records shall be kept of all testing and are to identify mix designs against test results in addition to stating date and time of sampling.

406 The properties of a grout shall be tested at regular intervals during the production of grout. The frequency of production testing of grout is as a minimum to be as given in Table F2.

<table>
<thead>
<tr>
<th>Table F2 Frequency of Production Testing of Grout</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>Strength</td>
</tr>
<tr>
<td>Initial and final setting time</td>
</tr>
<tr>
<td>Expansion and bleeding</td>
</tr>
<tr>
<td>Viscosity</td>
</tr>
<tr>
<td>Density</td>
</tr>
</tbody>
</table>

Until uniform quality of the grout has been demonstrated, higher frequencies of testing may be required.

407 Testing of grout is to be performed on specimens taken from samples collected during grout production. The collection, curing and testing is to be performed in accordance with an approved specification.

408 Samples for testing of fresh and hardened grout are, whenever possible, to be collected from evacuation points of the compartments being grouted and the samples taken from the emerging, surplus grout.

409 Records shall be kept of all testing and are to identify mix designs against test results in addition to stating the day, time and sections/parts grouted.

F 500 Testing of concrete in the structure

501 The quality of the concrete in the structure may be required verified by tests of sawn, drilled or in-situ cast cores from the structure, or by non-destructive examination. The extent, location and methods of such testing is to be agreed upon by client/verification authority in each case. Increased examination of concrete in the structure is to be considered if one of the following conditions occur:

— standard strength test specimens indicate abnormally low strength
— the concrete has visible signs of inferior quality
— the concrete has been subjected to chemical attack or fire
— the concrete during curing has been exposed to freezing or premature drying out
— inadequate compaction, curing or other unfavourable conditions are observed or suspected.

502 The procedures to be followed together with calibration methods and criteria for non-destructive examination shall be approved in each case.

503 When test results are compared, a relationship is to be established between the results from standard specimens tested in accordance with the approved specification and the results of the additional testing of the concrete in the structure.

G Formwork

G 100 Design, materials and erection

101 Falsework and formwork, including their supports and foundations, shall be designed and constructed so that they are:

— capable of resisting any actions expected during the construction process
— stiff enough to ensure that the tolerances specified for the structure are satisfied and the integrity of the structural member is not affected.

Form, function, appearance and durability of the permanent structure shall not be impaired due to falsework and formwork or their removal.

102 Formwork is to have sufficient strength, stiffness and dimensional stability to withstand the loadings from casting, compaction and vibration of fresh concrete. In addition the support conditions for the formwork and possible live and environmental loads prior to, during and after the casting shall be considered.

103 For special and critical casting operations it may be required to submit design calculations for the formwork for advance approval.

104 Special care is to be taken when designing formwork for concrete with long setting time where large heights of fresh concrete may exert significant loading on the formwork.

105 Slipforming operations shall be described in a slipforming procedure. The procedure shall contain structural design, jacking arrangement, power supply, method for dimensional control, criteria for lifting and emergency procedures in case of stoppage.

106 Feasibility tests on site may be required for complicated slipform operations.

107 Slipforms with variable dimensions shall be specially considered.

108 Materials for formwork are to accommodate the requirements to strength, stiffness and low water absorption. Formwork is to be erected by experienced personnel working in
accordance with detailed drawings. Wooden spacers are not to be used.

109 Any material that leads to the fulfilment of the criteria given for the structure may be used for formwork and falsework. The materials shall comply with relevant product standards where such exist. Properties of the specific materials, such as shrinkage, shall be taken into account if they can affect the end product.

110 The materials employed shall be consistent with any special requirements for the surface finish or later surface treatment.

111 The method statement shall describe the method of erection and dismantling of temporary structures. The method statement shall specify the requirements for handling, adjusting, intentional precambering, loading, unkeying, striking and dismantling.

112 Deformations of formwork during and after concreting shall be limited to prevent deleterious cracking in the young concrete. This may be achieved by limiting the deformations and by organizing the casting operations in a manner such as to avoid harmful deformations.

113 Formwork shall keep the concrete in its required shape until it is hardened.

114 Formwork and the joints between boards or panels shall be sufficiently tight against loss of water and fines.

115 Formwork that absorbs moisture or facilitate evaporation shall be suitably wetted to minimize water loss from the concrete, unless the formwork was designed specifically for that purpose.

116 The internal surface of the formwork shall be clean. When slipforming is used, the form panels shall be thoroughly cleaned and a grease-like mould-release agent shall be applied prior to assembling of the form.

G 200 Slipform systems

201 When using the slipforming method, the design and erection of the form and the preparation of the works shall take into account the difficulties controlling the geometry and the stiffness of the entire working platform. The entire slipform structure shall be designed with the appropriate stiffness and strength. Due account shall be taken of friction against hardening concrete, weight of materials stored on the form, systems for adjusting geometry such as diameter, wall thickness, as well as climatic conditions to be expected during the slipforming period.

202 The lifting capacity of the jacks shall be adequate. The climbing rods shall be sufficiently strong not to buckle. Normally, the climbing rods are left totally encased within the concrete. If the climbing rods shall be removed, the holes thus left in the concrete shall be properly filled with grout via grouting inlets at the bottom or by injection hoses threaded in from the top. The grout consumption shall be monitored to confirm complete filling.

203 The materials applied in the form may be either steel or wood, and shall comply with the requirements for formwork materials. The form shall have a height and batter consistent with the concrete to be used. The slipforming rate and other conditions affecting the hardening process of the concrete shall be such as to reduce or eliminate the tendency for lifting cracks.

204 The slipform shall have a hanging platform below the main form, giving access for application of curing as well as inspection and, if necessary, light repair of the hardening concrete when appearing from under the slipform.

205 The concrete cover to the reinforcement shall be kept within the tolerances using sufficiently long and stiff steel guides between the reinforcement and the form, adequately distributed around the form.

206 There shall be contingency plans prepared for unintended situations such as break-down in concrete supply, problems with the lifting devices, hardening of the concrete, etc.

G 300 Jumpforming systems

301 Jumpforming systems, when used, shall have adequate strength and stiffness for all loads exerted during the erection and casting period. There shall be a robust system for support of the forms in the previously cast concrete. Inserts for support shall be approved for the actual application.

302 The jumpform, when installed, shall allow the necessary preparation and cleaning of construction joints. The jumpform system shall accommodate the necessary walkways and access platforms to ensure that the concreting works can be performed in an appropriate manner.

G 400 Inserts in formwork, recesses and blockouts

401 Temporary inserts to keep the formwork in place bars, ducts and similar items to be cast within the section and embedded components, e.g. anchor plates, anchor bolts, etc. shall:

— be fixed robustly enough to ensure that they will keep their prescribed position during placing and concreting
— not introduce unacceptable loading on the structure
— not react harmfully with the concrete, the reinforcement or prestressing steel
— not produce unacceptable surface blemishes
— not impair functional performance, tightness and durability of the structural member
— not prevent adequate placing and compaction of the fresh concrete.

402 Any embedded item shall have sufficient strength and stiffness to preserve its shape during the concreting operation and be free of contaminants that would affect them, the concrete or the reinforcement.

403 Recesses used for temporary works shall be filled and finished with a material of similar quality as the surrounding concrete, unless it is otherwise specified. Block-outs and temporary holes shall generally cast with normal concrete. Their surfaces shall be keyed or slanted and prepared as construction joints.

G 500 Removal of formwork and falsework

501 Falsework and formwork shall not be removed until the concrete has gained sufficient strength to:

— resist damage to surfaces that may arise during the striking
— take the actions imposed on the concrete member at that stage
— avoid deflections beyond the specified tolerances due to elastic and inelastic (creep) behaviour of the concrete.

502 Striking shall be made in a manner that will not subject the structure to overload or damage.

503 Propping or re-propping may be used to reduce the effects of the initial loading, subsequent loading and/or to avoid excessive deflections. Propping may be required in order to achieve to intended structural behaviour of members cast in two or more casting operations.

504 If formwork is part of the curing system, the time of its removal shall take into account the requirements J300.

G 600 Surface treatment and final preparation

601 At completion of formwork erection and during slipforming operations it is to be ensured that the formwork is free of all foreign matter, that casting joints are prepared and treated as specified and that the formwork is given appropriate
surface treatment.

602 Formwork with permanent low-adhesion coating may be used. Form release agents used shall be satisfactorily documented not to be detrimental to the bond between reinforcement and concrete.

603 The surface treatment and final preparation of formwork is to be described in a special procedure.

604 Release agents shall neither be harmful to the concrete nor shall they be applied in a manner that will affect the concrete, the reinforcement or the bond between the two.

Release agents shall have a detrimental effect on the surface finish, or subsequent coatings if any. Release agents shall be applied in accordance with the manufacturer's specification.

605 Dimensional control during and after completion of the formwork is, as a minimum, to include:

— geometry and dimensions of cross sections.
— overall geometry, including deviation from theoretical shape and out of alignment.

H. Reinforcement and Embedded Steel

H 100 Reinforcement

101 Reinforcement is to be of the type, grade and dimensions given in the approved specification/ drawings (see also requirements in Sec.4) and is to be placed with the spacing, splices and concrete covers stated in the same documents.

102 The surface of the reinforcement is to be free of substances that may be harmful to the steel or the bond between steel and concrete at the time of installation and is to be protected from such substances until casting of concrete commences.

103 Reinforcement is normally to be cold bent to the required shape in one operation. Hot- or rebending is only allowed upon special agreement. Bending shall be done at a uniform rate.

104 Bending of reinforcement with temperature below 0°C shall only be performed on steel of given quality specified in Sec.5.

105 Welding of reinforcement is to be carried out by qualified welders working in accordance with approved procedures. The welds shall be non-destructively examined to the extent given in the approved specification. Production tests of such welds shall be considered for special wels of importance. The Production tests and quality of the welding procedures shall be documented.

106 Welding is only permitted on reinforcing steel that is classified as weldable in the relevant product standard, according to ISO 6935 or international standards.

107 Welding shall be used and performed in accordance with specifications by design, and shall conform to special requirements in International Standards as relevant.

108 Welding should not be executed at or near bends in a bar, unless specifically approved by the design.

109 Welding of galvanized or epoxy-coated reinforcement is only permitted when a procedure for repair is specified and approved.

110 For bars, wires, welded reinforcement and fabric bent after welding the diameter of the mandrel used should be as specified by design and in accordance with the standard relating to the type of reinforcement. Under no condition shall reinforcement be bent over a mandrel with diameter which is not at least 1.5 times greater than a test mandrel used to document by bending tests what that steel and bar diameter can take without cracking or damage.

111 In-place bending of steel in the formwork may be allowed if it can be demonstrated that the prescribed bending radius is obtained, and the work can be performed without misplacing the reinforcement.

112 The straightening of bent bars is prohibited unless the bars are originally bent over a mandrel with a diameter at least 1.5 times greater than a test mandrel used to document what that particular steel and bar diameter can take and be straightened without damage, a procedure for such work shall be prepared and approved.

113 Reinforcement delivered on coil shall be handled using the appropriate equipment, straightening shall be performed according to approved procedures, and all required mechanical properties maintained.

114 Prefabricated reinforcement assemblies, cages and elements shall be adequately stiff and strong to be kept in shape during transport, storage, placing and concreting, they shall be accurate so that they meet all the requirements regarding placing tolerances for reinforcement.

115 Deformed, high bond bars may be bundled in contact to ensure adequate concrete penetration into areas with congested reinforcement. Special attention is to be given to the possibility of water channels along the bars in such cases. For structures required to be watertight, no more than 4 bars, including the splices (see Sec.6 Q303), are allowed to be in the same bundle at any section.

116 The reinforcement is to be supported and fixed in a manner which prevents; accidental movement during completion of the formwork, and the casting, compaction and vibration of the concrete.

117 The specified concrete cover is to be ensured by securely fixed, sturdy spacers. Wooden spacers are prohibited.

118 Attention is to be paid to the execution and detailing of reinforcement at construction joints and the areas around pre-stressing anchorages.

119 Joints on bars shall be done by laps or couplers. Only couplers whose effectiveness is tested and approved may be used. Butt-welds may be permitted to a limited extent but only when subject to prequalification testing with non-destructive examination and visual quality inspection of all welds during execution. The welds shall be identified on design drawings.

120 The length and position of lapped joints and the position of couplers shall be in accordance with design drawings and the project specification. Staggering of such joints shall be considered in design. For details see Sec.6.

121 The reinforcement shall be placed according to the design drawings and fixed within the tolerances for fixing of reinforcement in this Standard, and secured so that its final position is within the tolerances given in this Standard. For details see Sec.5.

122 Assembly of reinforcement should be done by tie wire. Spot or tack welding is not allowed for the assembling of reinforcement unless permitted by national standards and the project specification, and due account has been taken of the risk of fatigue failure of the main rebar at the weld.

123 The specified cover to the reinforcement shall be maintained by the use of suitable chairs and spacers. Spacers in contact with the concrete surface in corrosive atmosphere shall be made from concrete of at least the same quality as the structure. Detailed requirements to concrete cover are given in Sec.6 Q100 and Q200.

124 In areas of congested reinforcement, measures shall be taken to ascertain that the concrete can flow and fill all voids without segregation and can be adequately compacted.

H 200 Prestressing ducts and anchorages

201 The prestressing assembly, e.g. all components of the
tendons, shall be assembled in accordance with suppliers' specifications or approval documents, and as shown in the approved for construction drawings.

202 The surfaces of ducts and anchorages shall be free of substances that may be harmful to the material or to the bond, and shall be protected from such substances until casting of concrete commences. All components of the entire post-tensioning assembly or system consisting e.g. of post-tensioning steel, ducts, sheaths, anchorage devices, couplers as well as prefabricated tendons and tendons fabricated on site shall be protected from harmful influences during transport and storage and also whilst placed in the structure prior to permanent protection. The ducts and anchorages shall be examined for mechanical damage and corrosion before installation.

203 Approval documents, identification documents and certification of tests on materials and/or tendons shall be available on site. Each item or component shall be clearly identified and traceable.

204 Documentation of post-tensioning steel of different deliver-ies shall be made in the as-built records.

205 Cutting shall be done by an appropriate method in a way that is not harmful.

206 Prestressing steel shall not be subject to welding. Steel in the vicinity of post-tensioning steel shall not be subject to oxygen cutting or welding except when sufficient precaution have been taken to avoid damage to post-tensioning steel and ducts.

207 The post-tensioning assembly shall be placed in compliance with the project/suppliers specification and in accordance with the relevant construction drawings. The tendon and all components shall be placed and secured in a manner that maintains their location within the permissible tolerances for position, angular deviation, straightness and/or curvature. Tendons shall not sag or have kinks of any kind. The ducts and anchorages shall be installed and fixed to prevent accidental movement during completion of the formwork and the casting, compac-tion and vibration of concrete.

208 The straight entry into anchorages and couplers as well as the coaxiality of tendon and anchorage shall be as specified by the supplier's specifications or system approval documents.

209 Care is to be taken during the installation and fixation of ducts to avoid undulations that may cause air and water pockets away from the high point vents during grouting.

210 Vents and drains on the sheaths shall be provided at both ends, and at all points where air or water can accumulate. In the case of sheaths of considerable length, inlets, vents and drains might be necessary at intermediate positions. As alternative to drains, other documented methods of removing water may be considered.

211 Inlets, vents and drains shall be properly marked to identify the cable.

212 The sheaths and their joints shall be sealed against ingress of water and the ends shall be capped to avoid, rain, dirt and debris of any kind. They shall be secured to withstand the effects of placing and compacting of the concrete.

213 Sheaths shall be checked after pouring of concrete to ensure sufficient passage for the tendons.

214 Sheaths shall be cleared of any water immediately prior to tendon threading.

H 300 Embedded steel

301 Embedded steel in the form of penetrations, surface embedments, etc. shall be of type and dimensions and shall be placed as shown on approved drawings.

302 The surfaces of embedments shall be free of substances that may be harmful to the material or the bond, and shall be protected from such substances until casting of concrete commences. The embedments shall be examined for mechanical damage and corrosion before installation.

303 Embedments shall be securely fixed at their location to prevent any accidental movement during succeeding construction stages.

304 Due consideration is to be given, where relevant, to heat transfer into the concrete during welding, and the correspond-ing effects on concrete quality, anchoring bond as well as the quality of the welding.

305 Adequate sealing is to be provided around embedments to prevent ingress of seawater to the reinforcement. Materials (waterstops or similar) and procedures for the sealing shall be in accordance with the approved specification. Temporary embedments shall be protected against corrosion, unless it can be demonstrated that their corrosion will not cause concrete spalling endangering the reinforcement.

H 400 Inspection and survey

401 During and after installation of reinforcement, ducts, anchorages and embedments, survey and inspection is to be performed. The survey and inspection is as a minimum to include:

- dimensions, type, grade, spacing and concrete cover for reinforcement.
- type, dimensions and location of ducts and anchorages.
- compliance with installation/operation procedures.

I. Production of Concrete and Grout

I 100 General

101 All the required properties for the concrete to achieve its service functions shall be identified. The properties of the fresh and hardened concrete shall take account of the method of execution of the concrete works, e.g. placing, compaction, formwork striking and curing.

102 Prior to any concreting, the concrete shall be documented by pretesting to meet all the requirements specified. Testing may be performed based on laboratory trial mixes, but should preferably also include a full-scale test from the batch plant to be used. Documented experience from earlier use of similar concrete produced on a similar plant with the same constituent materials may be regarded as valid pretesting. The quality control procedures shall be available at site. The procedures shall include the possible corrective actions to be taken in the event of nonconformity with the project specification or agreed procedures. For details see Sec.4.

103 The various mix designs shall be approved for their intended applications and the mix proportions recorded, again see Sec.4. Each approved mix design is to be allocated an identification symbol and the mix designs shall be related to the part of the structure or construction phase where they are intended to be used.

104 The lay-out of, and mixing procedures to be used at the mixing plant, shall be described and approved prior to start of construction. The description is to contain:

- description of plant lay-out and equipment
- qualification of personnel
- mixing time for wet and dry mixing
- methods of weighing and required tolerances
- method for monitoring fresh mix consistency.

105 The constituent materials shall be weighed, volumetric batching is not to be used unless adequate accuracy is documented regularly. The quantity of water used in the mixes is to be adjusted according to the water content of the aggregates.

106 In special cases, it may be required to maintain the tem-
perature of the fresh mix at certain levels. Cooling of constituent materials or addition of ice may be sufficient to bring about the desired cooling of the fresh mix. Conversely heating of constituent materials, such as steaming of frozen aggregates may be applicable. The usefulness of the methods and their influence on the properties of the mix design is to be investigated, documented and approved before such methods are used.

107 Survey and inspection is to be performed during production of concrete and grout, and is as a minimum to include:
- compliance with mix design and mixing procedures
- compliance with sampling and test intervals.

J. Transport, Casting, Compaction and Curing of Concrete

J 100 Transport

101 Transport of concrete from the mixing plant to the place of casting is to be performed in a manner that provides optimum quality concrete at the place of casting. Segregation in the fresh concrete is to be avoided, and in cases where early setting may represent a problem the maximum time allowed between emergence from the mixer and completed casting is to be specified and approved.

102 Rotating truck mixers shall be used for transport from the mixing plant, and pumping or buckets be used for placing the concrete in the forms. The use of bucket transporters is to be avoided, except for very short distances. Other methods of transportation may be considered by the Society.

103 Concrete shall be inspected at the point of placing and, in the case of ready-mixed concrete, also at the point of delivery. Samples for acceptance testing shall be taken at the point of placing, in the case of ready-mixed concrete samples for identity testing shall be taken at the point of delivery.

104 Detrimental changes of the fresh concrete, such as segregation, bleeding, paste loss or any other changes shall be minimized during loading, transport and unloading as well as during conveyance or pumping on site.

105 Concrete may be cooled or heated either during mixing, during transport to site or at site if documented acceptable by pretesting. The temperature of the fresh concrete shall be within the specified or declared limits.

106 The maximum amount of water that may be added to the concrete during the transport is to be specified and shall be in accordance with the pretesting documentation.

107 When pumping is used for the casting of large sections, a sufficient number of back-up units is to be provided.

J 200 Casting and compaction

201 A procedure for the casting process is to be prepared and submitted for approval by client/verification authority. The procedure is, as a minimum, to specify:
- inspection requirements prior to casting
- maximum thickness of each new layer of concrete. Maximum thickness of concrete that may remain not set
- maximum temperature to be allowed in the concrete during curing
- maximum/minimum temperature of the fresh mix at the place of casting
- extent of vibration and revibration
- contingency measures in case of form stop, blockage, equipment failure etc.

202 Before casting commences examination of the formwork, reinforcement, ducts, anchorages and embedments is to be completed with acceptable results. Immediately before placing of the concrete the formwork is to be examined for debris and foreign matters detrimental to concrete quality. The form shall be free of detritus, ice, snow and standing water.

203 Construction joints shall be prepared and roughened in accordance with project specifications. In monolithic structures an adequately roughened surface may be obtained by the application of a surface retarder on the fresh concrete, and later cleaning by water jetting. Construction joints shall be clean, free of laitance and thoroughly saturated with water, but surface dry. Construction joints in contact with the section to be cast shall have a temperature that does not result in the adjoining concrete freezing. Particular care is to be exercised in the preparation of construction joints in sections of the structure that are to remain watertight in temporary or operational phases.

204 During casting care is to be exercised when placing the concrete in the forms so that accidental displacement of reinforcement, embedments etc. will not occur.

205 The concrete shall be placed and compacted in order to ensure that all reinforcement and cast-in items are properly embedded in compacted concrete and that the concrete achieves its intended strength and durability. Vibration and compaction is to ensure thorough compaction, penetration of concrete into voids and homogeneous concrete. Direct contact between vibrators and reinforcement is to be avoided.

206 Appropriate procedures shall be used where cross-sections are changed, in narrow locations, at box outs, at dense reinforcement arrangements and at construction joints. Settlement cracking over reinforcement in top surface shall be avoided by revibration.

207 Casting of sections exceeding one metre in thickness, and very large pours require preparation of special procedures. Necessary precautions to be specified in the procedures, may include:
- artificial cooling of the fresh mix
- cooling of the concrete during curing
- insulation of the concrete to ensure an even temperature distribution during the first weeks of cooling
- special formwork for the casting operation.

208 The rate of placing and compaction shall be high enough to avoid cold joints and low enough to prevent excessive settlements or overloading of the formwork and falsework. The concrete shall be placed in layers of a thickness that is compatible with the capacity of the vibrators used. The concrete of the new layer should be vibrated systematically and include revibration of the top of the previous layer in order to avoid weak or inhomogeneous zones in the concrete. The vibration shall be applied until the expulsion of entrapped air has practically ceased, but not so as to cause segregation or a weak surface layer.

209 Concrete shall be placed in such a manner as to avoid segregation. Free fall of concrete from a height of more than 2 m shall not occur unless the mix is demonstrated to allow this without segregation. Concrete shall be compacted by means of high frequency vibrators. Contact between internal vibrators and reinforcement or formwork shall be avoided as much as possible. Vibrators shall not be used for horizontal transportation (spreading) of concrete.

NOTE: Alternative methods to the use of vibrators in order to obtain an adequately compacted concrete may be permitted provided this can be documented for the relevant type of conditions by trial casting.

210 Low temperature concreting may require special procedures to ensure that the concrete reaches adequate maturity. Necessary precautions to be specified in the procedures, may include:
- heating the concrete mix
— use of accelerators in the concrete mix
— heated and/or insulated formwork.

211 Hot weather concreting is to be performed carefully and the references to the maximum temperature of the concrete during curing shall be followed, to avoid excessive dehydration of the concrete. If the ambient temperature is forecasted to be above 30°C at the time of casting or in the curing period, precautions shall be planned to protect the concrete against damaging effects of high temperatures.

212 During placing and compaction, the concrete shall be protected against adverse solar radiation and wind, freezing, water, rain and snow. Surface water shall be removed during concreting if the planned protection fails.

213 For underwater concreting special procedures shall be prepared and the adequacy documented.

214 Records shall be kept during the casting operations. Each batch is to be recorded with regard to all specified and relevant information e.g. mix identification, contents of constituent materials, weights, mixing time, date and time of mixing, temperatures of the mix, part of the structure, reference to test samples taken, etc.

215 During casting of concrete survey and inspection is to be performed to ensure compliance with the approved procedure.

216 Special concreting methods shall be stated in the project specification or agreed.

217 Special execution methods shall not be permitted if they may have an adverse effect on the structure or its durability. Special execution methods might be required in cases where concrete with lightweight or heavyweight aggregates are used and in the case of under-water concreting. In such cases, procedures for the execution shall be prepared and approved prior to the start of the work. Trials might be required as part of the documentation and approval of the methods to be used.

218 Concrete for slipforming shall have an appropriate setting time. Slipforming shall be performed with adequate equipment and methods for transportation to the form and distribution at the form. The methods employed shall ensure that the specified cover to the reinforcement, the concrete quality and the surface finish are achieved.

J 300 Curing

301 Concreting procedures are to ensure adequate curing in order to obtain maximum durability, minimize plastic shrinkage, losses in strength and durability and to avoid cracking. The curing period is normally not to be less than two weeks. The duration of curing may be further estimated based on testing of strength or alternatively by the maturity of the concrete on the basis of either the surface temperature of the concrete or the ambient temperature. The maturity calculation should be based on an appropriate maturity function proven for the type of cement or combination of cement and addition used.

302 During curing the concrete surface is, as far as practicable, to be kept wet with fresh water. Care is to be taken to avoid rapid lowering of concrete temperature (thermal shock) caused by applying cold water on hot concrete surfaces. Seawater shall not be used for curing. Fresh concrete shall not be permitted submerged in seawater until an adequate strength of the surface concrete is obtained. If there is any doubt about the ability/capacity to keep the concrete surfaces permanently wet for the whole of the curing period, or where there is danger of thermal shock, a heavy duty curing membrane is to be used.

303 Whenever there is a possibility that the concrete temperature may fall below the freezing point during curing, adequate insulation is to be provided.

304 On completion of compaction and finishing operations on the concrete, the surface shall be cured without delay. If needed to prevent plastic shrinkage cracking on free surfaces, temporary curing shall be applied prior to finishing.

305 Curing compounds are not permitted on construction joints, on surfaces where bonding of other materials is required, unless they are fully removed prior to the subsequent operation, or they are proven to have no detrimental effects to bond.

306 Early age thermal cracking resulting from thermal gradients or restraints from adjoining members and previously cast concrete shall be minimized. In general, a differential in temperature across a section should not be allowed to exceed 10°C per 100 mm.

307 The concrete temperature shall not fall below 0°C until the concrete has reached a compressive strength of at least 5 MPa and also is adequate for all actions in frozen and thawed condition until the specified full strength is gained. Curing by methods using water shall not be done if freezing conditions are likely. In freezing conditions, concrete slabs and other elements that may become saturated shall be protected from the ingress of external water for at least seven days.

308 The peak temperature of the concrete within an element shall not exceed 70°C unless data are provided documenting that higher temperatures will have no significant adverse effect.

309 The set concrete shall be protected from vibrations and impacts that can damage the concrete or its bond to reinforcement.

310 The surface shall be protected from damage by heavy rain, flowing water or other mechanical influences.

J 400 Completion

401 Formwork is not to be removed until the concrete has gained the strength required to support itself and withstand other relevant loads imposed by the environment or construction activities.

402 After removal of the formwork tie-rods, spacer bars, etc. shall be broken off at a level corresponding to the concrete cover, and the holes patched with cement mortar.

403 The concrete surface is to be examined and areas subject to repair marked out. If any areas show visible signs of inferior quality, the area is to be marked for possible testing of concrete quality.

K. Completion of Prestressing Systems

K 100 Threading and stressing of tendons

101 Before threading of tendons is commenced, the anchorages and ducts shall be examined for possible damages, attacks of corrosion, blockage of ducts by concrete, the integrity of the ducts and watertightness. All ducts shall be cleared by compressed air or similar means prior to threading of tendons.

102 Tendons shall be examined for damages, corrosion, dimension and identification before they are threaded.

103 Stressing of tendons is to be carried out according to the system manufacturer's or other approved procedure which as a minimum is to specify:

— the sequence of stressing for multiple cables
— the number of stressing steps
— elongation versus load
— amount of over stressing to compensate for creep
— requirements to equipment.

104 Stressing of tendons is to be carried out by personnel with documented qualification, e.g. previous experience or adequate training.

105 On completion of stressing operations, protruding ends
of tendons shall be protected.

106 The final stress in each tendon is to be recorded.

107 During threading and stressing of tendons, survey and inspection is to be performed to ensure compliance with the approved procedure.

K 200 Tensioning of tendons

201 Tensioning shall be done in accordance with an approved method statement giving the tensioning programme and sequence. The jacking force/pressure and elongation at each stage/step in the stressing operation until full force is obtained shall be recorded in a log. The obtained pressures and elongations at each stage/step shall be compared to pre-calculated theoretical values. The results of the tensioning program and its conformity or non-conformity to the requirements shall be recorded. All observations of problems during the execution of the prestressing works shall also be recorded.

202 Stressing devices shall be as permitted for the prestressing system. The valid calibration records for the force measuring devices shall be available on the site before the tensioning starts.

203 Application and/or transfer of prestressing forces to a structure may only be at a concrete strength that meets the requirements as specified by design, and under no condition shall it be less than the minimumcompressive strength stated in the approval documents of the prestressing system. Special attention in this respect shall be paid to the anchorage areas.

K 300 Pre-tensioning

301 Pre-tensioning is normally carried out under manufacturer condition and the tendons are stressed prior to casting the concrete. If, during stressing, the calculated elongation cannot be achieved within a range of

±3% for a group of tendons, or
±5% for a single tendon within the group for the specified tensioning force,

action shall be taken in accordance with the method statement either to the tensioning program or to the design.

302 The release of prestressing force in the rig/bed shall be done in a careful manner in order not to affect the bond in the anchorage zone of the tendon in a negative manner.

303 If the fresh concrete cannot be cast in due time after tensioning, temporary protective measures shall be taken which will not affect the bond or have detrimental effect on the steel and/or the concrete.

304 Pre-tensioning will normally not be used as prestressing method for large offshore structures. However, if the offshore structure is assembled by precast elements, pre-tensioning may be applied.

K 400 Post-tensioning

401 Tensioning shall not take place at temperatures below +5°C within the structure unless special arrangements can assure the corrosion protection of non-grouted tendons. Tensioning is prohibited at temperatures below -10°C.

402 If, during the stressing operation, the calculated elongation cannot be achieved within a range of

±5% for a group of tendons, or
±10% for a single tendon within the group for the specified tensioning force,

action shall be taken in accordance with the method statement either to the tensioning programme or to the design.

403 In the case of deviations from the planned performance during tensioning, tendon-ends shall not be cut off and grouting is not permitted. Works that can impair re-tensioning shall not be carried out. No tendons shall be cut if the obtained elongations deviate from the theoretical by more than 5%, without design approval. Further work shall be postponed until the tendon has been approved, or further action decided.

NOTE: In case of deviations between theoretical and obtained results, tests to confirm friction factors and E-modulus of the tendon assembly might be necessary.

404 The prestressing tendons shall be protected from corrosion in the period from threading to presstressing. This period should normally not be allowed to exceed one week. Should the period from threading to casting exceed one week, then the condition of the tendons shall be specially evaluated for harmful conditions and special precautions may be required to protect the tendons.

K 500 Protective measures, grouting, greasing, concreting

501 Tendons placed in sheaths or rigid ducts in the concrete, couplers and anchorage devices shall be protected against detrimental corrosion. This protection shall be ensured by filling all voids with a suitable grouting/injection material such as cement grout, grease or wax. Anchorages and end caps shall be protected as well as the tendons.

502 In case of post-tensioning with required bond, cement grouting of sheaths shall comply with recognized international or national standards. Grouting/injection shall follow as soon as possible after tensioning of the tendons, normally within one week. If a delay is likely to permit corrosion, protective measures should be considered in accordance with national regulations or recommendations by the supplier.

503 A method statement shall be provided for the preparation and execution of the grouting/injection, all important data/observations from the grouting shall be reported in a log, e.g. volume consumed compared to theoretical volume, temperature of the structure and mix proportions and problems/stops.

504 Grouting devices shall be as permitted for the prestressing system.

K 600 Unbonded tendons

601 Anchorages areas of unbonded tendons or single strands, their sheaths and end-caps shall be filled by non-corrosive grease or wax. End caps shall be encased in concrete tied to the main structure by reinforcement.

602 Sheathed unbonded tendons shall be adequately sealed against penetration of moisture at their ends.

K 700 Grouting of ducts

701 Grouting with cement-based grouts shall only be done at temperatures in the structure in the range +5°C to +25°C. The temperature of the grout shall not be less than +10°C nor above +25°C. If a frost resistant grout is used, grouting at lower temperatures than +5°C may be permitted.

702 If the temperature in the structure is above +25°C grouting may be permitted provided special precautions valid in the place of grouting can ensure a successful grouting.

703 Grouting shall be carried out at a continuous and steady rate from the lowest inlet until the emerging grout has the appropriate quality, not affect by evacuated water or preservation oil from the duct. In vertical ducts, the grouting pressure shall be given particular attention. Normally the grout pressure inside the duct should not be allowed to exceed 2 MPa, unless permitted by the design. Non-retarded grout and grout with an expanding admixture shall be used within 30 min after mixing.

704 In vertical or inclined ducts or ducts of particularly large diameter, post-injection might be necessary in order to remove bleed water or voids. Post-injection shall be performed before the grout is stiffened. If voids are detected at inlets or outlets after the grout is stiffened, post-grouting shall be carried out, if required, by vacuum grouting.
A grouting procedure is to be prepared and submitted for approval. The procedure is, as a minimum, to contain the following information:

- requirements to fresh grout properties; bleeding, viscosity, density, etc.
- requirements to hardened grout
- batching and mixing requirements
- means of transportation of fresh grout
- requirements to pumps and other equipment
- grouting pressure
- holding time
- number and placing of vents
- particulars of difficult operations such as grouting of long, vertical ducts
- grout quality sampling points and procedure
- contingency measures in case of equipment failure, blockages, etc.

Prior to start of operation it is to be ensured that the grouting system is operable, and that air and surplus grout may be evacuated from the ducts at a rate exceeding the filling rate. Means shall be provided to observe the emergence of grout from the various vent points of the ducts.

The grouting operation is to be conducted with strict adherence to the approved procedure. In case of vacuum-injection, the free volume in the ducts shall be measured. The amount of grout injected shall be comparable with this volume. Vacuum grouting procedures, particularly in the case of vertical tendons, should be prequalified by trials of relevant geometry.

Provision for vacuum grouting or reinjection shall be made in case of discovery of a blockage in the duct. Ducts shall under no circumstances be left empty and ungrouted without specific approval by design.

After completion of grouting, unintended loss of grout from the ducts shall be prevented by sealing them under pressure of minimum 0.5 MPa for a minimum of one minute.

If grouting of a duct is interrupted, corrective actions, such as washing out all fresh grout, shall be taken. No ducts shall be left with incomplete filling of grout.

Records shall be kept during the grouting operation. Each batch is to be recorded with regard to the specified and relevant information e.g. mix identification, constituent materials, weights, mixing time, date and time of mixing, volume, duct being grouted, reference to test samples taken, etc.

During the grouting operation, survey and inspection is to be performed to ensure compliance with the approved procedure.

Adequate records related to the construction of the structure shall be prepared. Construction records shall be compiled in parallel with the construction process. Compiled records shall be systematic and fully traceable. Such records are to reflect all testing, alterations, additions, corrections and revisions made during the construction period in order to provide information required during the in-service life of the structure.

As a minimum the construction records are to contain:

- quality assurance/quality control manual
- relevant material certification and test reports
- summary of reports of testing of constituent materials, additives and reinforcement
- summary reports of production testing of concrete and grout with reference to location in the structure
- summary report of testing of concrete in the structure
- summary reports from stressing of prestressing system, including final stresses
- summary of repair work, including location references
- documentation of welding and structural steel work
- dimensional control reports of final geometry of cross sections, overall geometry (including deviation from theoret-

M. Corrosion protection

N. Site Records and As-built Documentation

L. Repairs
icai shall minimize the risk of possible damage
effectiveness and stability of temporary and final supports.

302
assembly drawings and the erection program.

tion of the elements shall be performed in accordance with the
shall also define the arrangement of the supports and possible
from the specified requirements
information with regard to storage, handling, installation,
testing and operation of items shipped with the structure.

O. Precast Concrete Elements

O 100 General

101 This clause specifies requirements for the construction operations involving precast elements, whether produced in a factory or a temporary facility at or outside the site, and applies to all operations from the time the elements are available on the site, until the completion of the work and final acceptance.

102 Precast elements, when used in offshore concrete structures, their manufacture and design are covered by this Standard, and shall meet all requirements to materials, strength and durability as if they were cast in situ

103 When precast elements are used, these shall be designed for all temporary conditions as well as the structural performance in the overall structure. This shall at least cover:

— joints, with any bearing devices, other connections, additional reinforcement and local grouting
— completion work (in situ casting, toppings and reinforce ment)
— load and arrangement conditions due to transient situations during execution of the in situ works
— differential time dependent behaviour for precast and in situ concrete.

104 Precast elements shall be clearly marked and identified with its intended position in the final structure. As built information and records of conformity testing and control shall be available.

105 A complete erection work program with the sequence of all on-site operations shall be prepared, based on the lifting and installation instructions and the assembly drawings. Erection shall not be started until the erection program is approved.

O 200 Handling and storage

201 Instructions shall be available giving the procedures for the handling, storage and protection of the precast elements.

202 A lifting scheme defining the suspension points and forces, the arrangement of the lifting system and any special auxiliary provision shall be available. The total mass and centre of gravity for the elements shall be given.

203 Storage instructions for the element shall define the storage position and the permissible support points, the maximum height of the stack, the protective measures and, where necessary, any provisions required to maintain stability.

O 300 Placing and adjustment

301 Requirements for the placing and adjustment of the precast elements shall be given in the erection program, which shall also define the arrangement of the supports and possible temporary stability provisions. Access and work positions for lifting and guiding of the elements shall be defined. The erection of the elements shall be performed in accordance with the assembly drawings and the erection program.

302 Construction measures shall be applied which ensure the effectiveness and stability of temporary and final supports. These measures shall minimize the risk of possible damage and of inadequate performance.

303 During installation, the correct position of the elements, the dimensional accuracy of the supports, the conditions of the element and the joints, and the overall arrangement of the structure shall be checked and any necessary adjustments shall be made.

O 400 Jointing and completion works

401 The completion works are executed on the basis of the requirements given in the erection program and taking climatic conditions into account.

402 The execution of the structural joints shall be made in accordance with the project specifications. Joints that shall be concreted shall have a minimum size to ensure a proper filling. The faces shall normally meet the requirements to construction joints.

403 Connectors of any type shall be undamaged, correctly placed and properly executed to ensure an effective structural behaviour.

404 Steel inserts of any type, used for joint connections, shall be properly protected against corrosion and fire by an appropriate choice of materials or covering.

405 Welded structural connections shall be made with weldable materials and shall be inspected. Threaded and glued connections shall be executed according to the specific technology of the materials used.

P. Geometrical Tolerances

P 100 General

101 Design tolerances are specified in Sec.6, C400 with design consequences in Sec.6, B607. The design assumption is based on an alternative approach, either:

— design and construct in accordance with the tolerances in Sec.6, C400 with high material factors provided in Sec.6, B607; or
— design and construct the for any tolerances, the maximum positive and negative tolerances have to be included in design in the most design critical way, and the construction work has to confirm compliance with the set of tolerances.

102 This clause defines the types of geometrical deviations relevant to offshore structures. See P300, P400 and P500. The list is provided as guidelines, and the designer shall fill in the required tolerances to be used in construction. The tolerances shall be marked on the drawings issued for construction.

103 In general, tolerances on dimensions are specified in order to ensure that:

— geometry is such as to allow parts fit together as intended
— geometrical parameters used in design are satisfactorily accurate
— the structural safety of the structural member is ensured
— construction work is performed with a satisfactorily accurate workmanship
— weights are sufficiently accurate for floating stability considerations.

104 All these factors shall be considered when tolerances are specified. Tolerances assumed in design (See Sec.6 C400) may be greater than the tolerances actually found to be acceptable for other reasons.

105 Changes in dimensions following temperature effects, concrete shrinkage, post-tensioning and application of loading, including those resulting from different construction sequences, are not part of the construction tolerances. When deemed important, these changes shall be considered separately.
P 200 Reference system

201 A system for setting out tolerances and the position points, which mark the intended position for the location of individual components, shall be in accordance with ISO 4463-1.

202 Deviations of supports and components shall be measured relative to their position points. If a position point is not established, deviation shall be measured relative to the secondary system. A tolerance of position in plane refers to the secondary lines in plane. A tolerance of position in height refers to the secondary lines in height.

P 300 Member tolerances (Guidelines)

301 Requirements may be given for the following type of tolerances as relevant:

a) skirts:
   - deviation from intended centre for circular skirts
   - deviation from intended position for individual points along a skirt
   - deviation from best fit circle for circular skirts
   - deviation from intended elevation for tip and top of skirt
   - deviation from intended plumb over given heights.

b) slabs and beams:
   - deviation from intended elevation for centre plane
   - deviation from intended planeness measured over given lengths (2 m and 5 m)
   - deviation from intended slope.

c) walls, columns and shafts:
   - deviation from intended position of centre plane or horizontal centre line
   - deviation from intended planeness - horizontal direction
   - deviation from intended planeness - vertical direction
   - deviation from intended plumb over given heights.

d) domes:
   - deviation of best fit dome centre from intended centre, horizontal and vertical directions
   - deviation of individual points from best fit inner dome
   - deviation of individual points from best fit exterior dome.

e) circular members:
   - deviation of best fit cylinder centre from intended centre line
   - deviation of best fit inner radius from intended inner radius
   - deviation of individual points from best fit inner circle over given lengths
   - deviation of individual points from best fit exterior circle over given lengths
   - deviation from intended plumb over given height.

f) shaft/deck connections:
   - deviation of best fit centre from intended centre of shaft
   - deviation in distances between best fit centres of shafts
   - position of temporary supports horizontal and vertical
   - position of anchor bolts horizontal plane and verticality.

P 400 Cross-sectional tolerances (Guidelines)

401 Requirements may be given for the following type of tolerances:

a) thickness:
   - individual measured points
   - overall average for area.

b) reinforcement position:
   - tolerance on concrete cover
   - tolerance on distance between rebar layers same face
   - tolerance on distance between rebar layers opposite faces
   - tolerances on spacing of rebars in same layer
   - tolerances on laplengths.

c) prestressing:
   - tolerance on position of prestressing anchors
   - position of ducts/straightness at anchors
   - position of ducts in intermediate positions
   - tolerances on radius for curved parts of tendons.

P 500 Embedments and penetrations (Guidelines)

501 Requirements may be given for the following type specified of tolerances as relevant. Tolerances shall be for items individually or for groups, as appropriate:

a) embedment plates:
   - deviation in plane parallel to concrete surface
   - deviation in plane normal to concrete surface
   - rotation in plane of plate (degrees).

b) attachments to embedments:
   - deviation from intended position (global or local system)


c) penetrations:
   - sleeves deviation from intended position of centre
   - sleeves deviation from intended direction
   - manholes deviation from intended position and dimension
   - blackouts deviations from intended position and dimensions.
SECTION 8
IN-SERVICE INSPECTION, MAINTENANCE AND CONDITIONAL MONITORING

A. General

A 100 Application

101 The purpose of this section is to specify requirements and recommendations for in-service inspection, maintenance and condition monitoring of offshore concrete structures, and to indicate how these requirements and recommendations can be achieved. Alternative methods may also fulfil the intent of these provisions and can be demonstrated and documented to provide the same level of safety and confidence.

102 Requirements for In-service Inspection, maintenance and Condition Monitoring for Concrete Offshore Structures in general are given under this Sub-heading A “General”.

A 200 Scope

201 The in-service inspection, maintenance and condition monitoring programme shall be established as part of the design process considering safety, environmental consequences and total life cycle costs.

The overall objective for the inspection, maintenance and condition monitoring activities is to ensure that the structure is suitable for its intended purpose throughout its lifetime.

The condition monitoring activities should include the latest developments, knowledge and experience available. Special attention should be paid to deterioration mechanisms for the relevant materials and structural components:

- time-dependent effects
- mechanical/chemical attacks
- corrosion, loading
- seabed conditions
- stability
- scour protection and damage from accidents.

As appropriate, the condition monitoring activities should reflect the need for repair works and maintenance.

Maintenance shall be carried out according to a plan based on the expected life of the structure or component or when the specified inspection or monitoring efforts detect unpredicted happenings.

A 300 Personnel qualifications

301 Personnel involved in inspection planning and condition assessment shall have relevant competence with respect to marine concrete design, concrete materials technology, concrete construction and specific experience in the application of inspection techniques and the use of inspection instrumentation and equipment. Because each offshore structure is unique, inspectors shall familiarize themselves with the primary design and operational aspects before conducting an inspection.

302 Inspectors shall have adequate training appropriate for supervisors, divers, ROV-operators as specified in accordance with national requirements where applicable.

A 400 Planning

401 The planning of in-service inspection, maintenance and condition monitoring activities shall be based on the:

- function of each structural element
- exposure to damage
- vulnerability to damage
- accessibility for inspection.

402 The condition of the loadbearing structure shall be documented by periodic examinations and, where required, supplemented by instrumentation-based systems. A programme for planning and implementation of inspection and condition monitoring including requirements for periodic inspections shall be prepared. The programme for inspection and condition monitoring shall cover the whole structure and comprise the use of instrumentation data.

403 If values for loads, load effects, erosion or foundation behaviour are highly uncertain, the installation shall be equipped with instrumentation for measurement of environmental condition, dynamic motion, strain, etc. to confirm the applicability of governing design assumptions. Significant changes to equipment and storage/ballast operations should be identified and recorded.

404 Continuous monitoring shall be carried out to detect and give warnings regarding damage and serious defects, which significantly reduce the stability and load carrying capacity. Significant events are those that within a relatively short period of time can cause structural failure or those that represent significant risk to people or the environment or those having large economical consequences. Forecasting the occurrence of these events is needed to allow sufficient lead-time for corrective action (e.g. repair) or abandonment.

405 The structure should also be monitored to detect small damages and defects, which can develop to a critical situation. Particular emphasis should be placed on identifying the likelihood of small failures, which can lead to progressive collapse. The type and extent of monitoring on this level should be handled as a risk minimization problem, which includes the probability of damage/defect occurrence, detection probability, monitoring costs and cost savings by repairing the damage/defect at an early stage.

A 500 Programme for inspection and condition monitoring

501 The first programme for inspection and condition monitoring should provide an initial assessment, as described in A602 of the condition of the structure, i.e. the assessment should have an extent and duration which, as far as possible, provides a total description of the condition of the structure (design verification). The programme for in-service inspection, maintenance and condition monitoring shall be based on information gained through preceding programmes and new knowledge regarding the application of new analysis techniques and methods within condition monitoring and maintenance. As such, the programme shall be subjected to periodic review, and possible revision as new techniques, methods or data become available. The intervals may also be altered on the same basis.

A 600 Inspection and condition monitoring milestones and intervals

601 Accumulated historical inspection data, experiences gained from similar structures together with thorough knowledge based on concrete design and technology, i.e. deterioration processes etc., form the basis for defining necessary inspection and condition monitoring intervals. The extent of work effort on inspection and condition monitoring shall be sufficient to provide a proper basis for assessing structural integrity and thereby the safety for the personnel involved, with respect to defined acceptable risks and consequences of failure.
An early inspection to verify that the structure has no obvious defects shall be carried out soon after installation. The inspection activities and the assessment shall be carried out during the first year of operation. This initial inspection shall be comprehensive and thorough, and shall address all major structural elements.

During in-service, more information will become available and the knowledge about the initial condition can be updated.

Inspection and condition monitoring of the structure shall be carried out regularly in accordance with the programme for inspection and condition monitoring established.

Assessment of the condition shall be carried out following the inspection activities. A summary evaluation shall be prepared at the end of each programme for inspection and condition monitoring period as outlined in A700. The data gathered from each periodic inspection shall be compared to data gathered from previous inspections. Evaluations shall consider not only new information, but also data trends that might indicate time-dependent deterioration processes.

Inspection and condition monitoring should be conducted after direct exposure to a design environmental event (e.g., wave, earthquake, etc.). Special inspection following a design environmental event shall encompass the critical areas of the structure. Special inspections following accidental events may, in certain circumstances, be limited to the local area of damage. Inspection should also be conducted after severe accidental loading (e.g., boat collision, failing object, etc.)

In the event of change of use, lifetime extension, modifications, deferred abandonment, damages or deterioration of the structure or a notable change in the reliability data on which the inspection scheme is based, measures should be taken to maintain the structural integrity appropriate to the circumstances. The programme shall be reviewed to determine the applicability to the changed conditions and shall be subjected to modification as required. Risk to the environment shall be included.

A removal programme, an assessment of the structural integrity may be carried out prior to removal. The need to complete this assessment, and the extent of the assessment and inspection required, will depend heavily on the period, which has elapsed since the last periodic or special inspection. As a minimum, however, this assessment needs only consider safety of personnel.

### A 700 Documentation

The efficiency and integrity of the inspection and condition monitoring activities is dependent on the validity, timeliness, extent and accuracy of the available inspection data.

To facilitate periodic inspection as specified in the programme for inspection and condition monitoring, the following documents/information shall be recorded:

- Data from the design, construction and installation phase (Summary Report).
- Basic information about each inspection performed (e.g., basic scope of work, important results, available reports and documentation).

Up-to-date summary inspections shall be retained by the owner/operator. Such records shall describe the following:

- tools/techniques employed
- actual scope of work (including any field changes)
- inspection data collected including photographs, measurements, videotapes, etc.
- inspection findings, including thorough descriptions and documentation of any anomalies discovered.

Any repairs and in-service evaluations of the structure shall be documented and retained by the owner/operator.

### A 800 Important items related to inspection and condition monitoring

#### 801 Inspection of concrete offshore installations normally includes a survey of the different parts of the structure, including the atmospheric zone, the splash and the tidal zones and the large amounts of immersed concrete. It is generally recognized that the splash zone is the most vulnerable to corrosion. The submerged zone is also recognized as important because most of the structure is underwater.

#### 802 Inspection activities, therefore, will most often seek to identify symptoms and tell-tale signs made evident on the surface originating from the defect, i.e. often at a relatively advanced stage of defect progression. In many cases, it is assumed that signs of damage will be obvious before the integrity of the structure is impaired, but it should not be assumed that this always is the case.

#### 803 Essential elements of a successful condition monitoring programme include the following:

- It is focused on areas of high damage probability and areas critical to safety
- It is well documented
- It is completed at the specified intervals, as a minimum
- It is repetitive, to enhance training of assigned personnel.

#### Guidance note:

It is also important to differentiate between the extent of assessment and frequency for inspection for different structural elements. The function of each structural element will play a role in establishing the extent and frequency of assessment. The exposure or vulnerability to damage, of each element, shall be considered when establishing priorities for assessment. The accessibility for assessment may also be highly variable. The atmospheric zone provides the least difficult access, while the submerged zone the most. However, the splash zone may provide the most severe environmental exposure and a greater likelihood of accidental impact for many concrete marine structures. Therefore the condition monitoring plan shall consider the function of each structural element and provides further consideration of element access and exposure. Focusing on critical structural elements located in high exposure areas of the structure lead to efficiency in monitoring.

### 804 Inspection and condition monitoring of the Atmospheric Zone should focus on detecting possible damage or defects caused by:

- structural design and construction imperfections
- environmental loads
- mechanical loads
- static and dynamic operational loads
- altered operational conditions
- chloride ingress
- geometric anomalies, such as construction joints, penetrations, embedments
- subsidence
- impact loads.

#### Typical defects will be:

- deformation/structural imperfections
- cracks
- reinforcement corrosion
- damaged coatings
- freeze/thaw damage
- spalls and delaminations
- local impact damage.

#### 805 In addition to the aspects listed for the atmospheric zone, the inspection and condition monitoring of the Splash Zone...
should focus on:

- effects due to alternating wetting and drying of the surface
- marine growth.

806 In addition to the aspects listed for the Atmospheric and Splash zones, the inspection and condition monitoring of the Submerged Zone should focus on:

- scouring of the seabed under or in the immediate vicinity of the installation or build-up of seabed substance/sediments
- build-up of substance/sediments if such build-up covers significant parts of the structure
- current conditions
- movement in bottom sediments
- mechanical loads
- tension cable anchor points
- debris
- settlement
- cathodic protection system (anodes).

807 The inspection of the internal parts shall focus especially on:

- detecting any leakage
- biological activity
- temperature, composition of seawater and pH values in connection with oil storage
- detecting any reinforcement corrosion
- concrete cracking.

The presence of bacterial activity, such as sulphate reducing bacteria (SRB), and pH shall be evaluated, considering the quality and thickness of the concrete cover. Necessary action against possible harmful effect of bacterial activity shall be evaluated.

808 Concrete durability is an important aspect concerning structural integrity and shall be assessed during the lifetime of the structure. Important factors to assess are:

- those factors that are important but are unlikely to change significantly with time, such as permeability and cover to reinforcement
- those factors that will change with time and need to be assessed regularly, such as chloride profiles, chemical attacks, abrasion depth, freeze/thaw deterioration and sulphate attack, especially in petroleum storage area.

809 Chloride profiles should be measured in order to establish the rate of chloride ingress through the concrete cover. Either total chloride ion content or water-soluble chloride content should be measured. However, the method chosen should be consistent throughout the life of the structure. These profiles can be used for estimating the time to initiation of reinforcement corrosion attack in the structure.

A 900 Corrosion protection

901 Periodic examination with measurements shall be carried out to verify that the cathodic protection system is functioning within its design parameters and to establish the extent of material depletion.

902 As far as cathodic protection (or impressed current) is utilized for the protection of steel crucial to the structural integrity of the concrete, the sustained adequate potential shall be monitored. Examination shall be concentrated in areas with high or cyclic stress utilization, which need to be monitored and checked against the design basis. Heavy unexpected usage of anodes should be investigated.

903 Inspection of coatings and linings is normally performed by visual inspection and has the objective to assess needs for maintenance (i.e. repairs). A close visual examination will also disclose any areas where coating degradation has allowed corrosion to develop to a degree requiring repair or replacement of structural or piping components.

904 Inspection of corrosion control based on use of corrosion resistant materials can be integrated with visual inspection of the structural or mechanical components associated with such materials.

Guidance note:

One of the main objectives of an inspection is to detect any corrosion of the reinforcement. Several techniques have been developed for the detection of corrosion in the reinforcement in land-based structures. These are mainly based on electro potential mapping, for which there is an ASTM standard. Since the corrosion process is the result of an electrochemical cell measurement of the electro potential of the reinforcement can provide some indication of corrosion activity. These techniques are useful for detecting potential corrosion in and above the splash zone but have limited application underwater because of the low resistance of seawater.

905 It has been established that under many circumstances underwater corrosion of the reinforcement does not lead to spalling, and rust staining. The corrosion products are of a different form and can be washed away from cracks, leaving no evidence on the surface of the concrete of buried corrosion of the reinforcement. However when the reinforcement is adequately cathodically protected any corrosion should be prevented. In cases where cathodic protection of the reinforcement can be limited, the absence of spalling and rust staining at cracks in the concrete cover should not be taken as evidence for no corrosion.

A 1000 Inspection and condition monitoring types

1001 The extent and choice of methods may vary depending on the location and function of the actual structure/structural part. In the choice of inspection methods due consideration shall be taken to reduce the risk associated with the inspection activity itself. The main techniques for use underwater depend on visual inspection, either by divers or by ROVS. In some cases, it is necessary to clean off marine growth to examine potential defects in more detail.

1002 The methods shall be chosen with a focus on discovering serious damage or defects on the structures. The methods shall reveal results suitable for detection and characteristic description of any damage/defect. Areas with limited accessibility should preferably be monitored through instrumentation.

1003 The following type of inspection shall be considered:

a) Global visual inspection

Global visual inspection is an examination of the total structure to detect obvious or extensive damage such as impact damage, wide cracks, settlements, tilting, etc. The inspection can be performed at a distance, without direct access to the inspected areas, for instance by use of buoys. Prior cleaning of inspection item is not needed. The inspection should include a survey to determine if the structure is suffering from uniform or differential settlement.

b) Close visual inspection

Close visual inspection is a visual examination of specific surface area, structural part or total structure to detect incipient or minor damage. The inspection method requires direct access to the inspected area. Prior cleaning of the inspected item might be needed.

c) Non-destructive inspection/testing

Non-destructive inspection/testing is a close inspection by electrical, electrochemical or other methods to detect hidden damage. The inspection method requires direct access to the inspected area. Prior cleaning of the inspection item is normally required.
d) Destructive testing

**Destructive testing** is an examination by destructive methods such as core drilling, to detect hidden damage or to assess the mechanical strength or parameters influencing concrete durability.

e) Instrumentation based condition monitoring (IBCM).

In areas with limited accessibility, or for monitoring of load effects, corrosion development, etc., additional information can be provided by use of **instrumentation based condition monitoring**. The instrumentation can be temporary or permanent. Sensors shall preferably be fitted during fabrication. The sensors will be such as strain gauges, pressure sensors, accelerometers, corrosion probes, etc.

**1004** The structure may be instrumental in order to record data relevant to pore pressure, earth pressure, settlements, subsidence, dynamic motions, strain, inclination, reinforcement corrosion, temperature in oil storage, etc.

**1005** In the case where the structure is equipped with active systems which are important to the structural integrity, e.g. pore pressure, water pressure under the base, drawdown (reduced water level internally in the structure to increase the external hydrostatic prestressing of the structural member) in case of storms, etc., these monitoring systems shall be inspected regularly.

**A 1100 Marking**

**1101** A marking system shall be established to facilitate ease of identification of significant items for later inspection. The extent of marking should take account of the nature of the deterioration to which the structure is likely to be subjected and of the regions in which defects are most prone to occur and of parts of the structure known to become, or have been, highly utilized. Marking should also be considered for areas suspected to be damaged and with known significant repairs. The identification system should preferably be devised during the design phase. In choosing a marking system, consideration should be given to using materials less prone to attract marine growth and fouling.

**A 1200 Guidance for inspection of special areas**

**1201** Poor quality concrete, or concrete containing construction imperfections, should be identified during the initial condition assessment, and monitored for subsequent deterioration. Surface imperfections of particular importance include poorly consolidated concrete and rock pockets, spalls, delaminations and surface corrosion staining.

**1202** The emphasis for the monitoring will be to detect and monitor damage caused by overstressing, abrasion, impact damage, and environmental exposure.

**1203** Overstressing is often evidenced by cracking, spalling, concrete crushing, and permanent distortion of structural members. Not all cracking is the result of structural overload. Some cracking can be the result of creep, restrained drying shrinkage, plastic drying shrinkage, finishing, thermal fluctuations, and thermal gradients through the member thickness. Creep and restrained shrinkage cracks commonly penetrate completely through a structural member, but are not the result of overload. Plastic drying shrinkage and finishing cracks commonly do not penetrate completely through a member and are also not load related.

**1204** Non-characteristic cracking pattern. Whenever possible, inspectors should be familiar with characteristic cracking patterns that are associated with loading. A second distinction that should be made is whether the observed cracks are “active” or “passive”. Active cracks are those that change in width and length as loads or deformation occur. Passive cracks are benign in that they do not increase in severity with time. Sec.5 provides guidance on critical crackwidths that signal concern for the ingress of chloride ions and the resulting corrosion of embedded reinforced steel. Active cracks and load or deformation-induced cracks should be investigated regardless of crackwidth. The investigation should identify the cause or causes, the changes with time, and the likely effect on the structure.

**1205** Concrete crushing, spalling and delamination also require careful determination of cause. Crushing is generally associated with either flexural overload, axial compression or impact. Delamination and spalling can be either load related or caused by severe corrosion of embedded reinforced steel. The appropriate repair method for these distress types will vary considerably depending upon the actual distress cause.

**1206** The interface being the main load transfer point between the steel super-structure and the concrete support should preferably be examined for structural integrity annually. The examination should include the load transfer mechanism (flexible joints, rubber bearings, bolts and cover) and the associated ring beam.

The concrete interface should be inspected for evidence of overstress and corrosion of embedded reinforcement steel. Corrosion potential surveys can be used to detect ongoing corrosion that is not visible by visual inspection alone.

**1207** Construction joints in the concrete structure represent potential structural discontinuities. Water leakage and reinforcement corrosion are possible negative effects. Construction joints should be located remote from locations of high stress and high fatigue cycling. However, achieving these recommendations is not always possible. As a minimum, the monitoring program should identify construction joints located in high stress areas, and monitor the performance with respect to evidence of:

- leakage
- corrosion staining
- local spalling at joint faces, which indicate relative movement at the joint
- evidence of poorly placed and compacted concrete, such as rock pockets and delaminations
- joint cracking or separation.

**1208** Penetrations are, by their nature, areas of discontinuity and are prone to water ingress and spalling at the steel/concrete interface. Penetrations added to the structure during the operational phase are particular susceptible to leakage resulting from difficulties in achieving high quality consolidation of the concrete in the immediate vicinity of the added penetration. All penetrations in the splash and submerged zones will require frequent inspections.

**1209** Vertical intersections between different structural parts. A representative sample, chosen to coincide with the highest stress/fatigue utilization as obtained from analysis, should be inspected. Areas with known defects should be considered for more frequent examination. The significance of cracks, in these areas, on the structural integrity is substantial and emphasises the need for frequent crack monitoring for dynamic movement and length and width increases.

**1210** Embedment plates may constitute a path for galvanic corrosion to the underlying steel reinforcement. Main concerns are corrosion and spalling around the plates. Galvanic corrosion is especially severe where dissimilar metals are in a marine environment and may lead to deterioration of the reinforcing steel, which is in contact with the embedments.

**1211** Repair areas and areas of inferior construction. These areas need to be individually assessed on the extent and method of repair and their criticality. Particular concern may be associated with areas that provide a permeable path through which salt-water flow can take place. Continuous flow of saline and oxygenated water can cause corrosion of the reinforcement and washout of cementitious paste with an ensuing weakening effect of the reinforced concrete matrix. In such
areas, adequate emphasis needs to be placed on the detection of local loss of reinforcement section due to chloride induced (black) corrosion. Attention should be placed on the surface and the perimeter of patched areas for evidence of shrinkage cracking and loss of bond to the parent concrete surface.

1212 The splash zone can experience damage from impact of supply vessels, etc. and can also deteriorate from ice formation with ensuing spalling in surface cavities where concrete has been poorly compacted.

Even where high quality concrete was placed originally, the splash zone is susceptible to early deterioration as a result of ice abrasion and freeze-thaw cycling. Both distress mechanisms result in loss of surface concrete, with subsequent loss of cover over the reinforcement steel. For structures designed for lateral loads resulting from the movement of pack ice relative to the structure, the heavily abraded concrete surface can cause an increase in applied global lateral loads. Repairs to these surfaces should be made as soon as possible to prevent further deterioration and structural overload.

1213 Debris. Drill cuttings can build up on the cell tops and/or against the side of the structure and should be assessed for:

- lateral pressures exerted by the cuttings
- whether they cause an obstruction to inspection.

Removal of drill cuttings needs to be assessed accordingly. Debris can cause structural damage through impact, abrasion, or by accelerating the depletion of cathodic protection systems. Also, it poses a danger to diving activities and precludes examination if allowed to accumulate. Particular vigil needs to be maintained for impact damage, covered by debris.

1214 Scour is the loss of foundation supporting soil material and can be induced by current acceleration round the base of the structure or by "pumping" effects caused by wave induced dynamic rocking motion. It can lead to partial loss of base support and ensuring unfavourable redistribution of loads.

1215 Differential hydrostatic pressure (drawdown). Structural damage, or equipment failure, can lead to ingress of water and affect the hydrostatic differential pressure (see A1005). This might call for special inspection before and during drawdown.

1216 Temperature of oil to storage. Continuous records of the temperature of the oil sent to storage should be examined for compliance with design limits.

In cases where differential temperatures have exceeded design limits, following an analysis of the additional loading, special inspections might be required.

1217 Sulphate reducing bacteria (SRB). SRBs occur in anaerobic conditions where organic material is present (such as hydrocarbons). The bacteria produce as their natural waste H$_2$S (Hydrogen sulphide) which, in large enough amounts, will cause a lowering of pH value of the cement paste in the concrete. Favoured conditions for SRB growth might be present in un-aerated water in for example the water filled portion of shafts and cells. An acidic environment can cause concrete softening and corrosion of reinforcement. An inspection of the concrete surface, which is likely to be affected by SRB activity, is difficult to undertake. Some guidance can be obtained by adequate monitoring of SRB activity and pH levels.

1218 Post-tensioning. Tendons are usually contained within ducts, which are grouted. Inspection of tendons is therefore very difficult using conventional inspection techniques.

Guidance note:
Some problems with inadequate protection of tendons have been found through water leakage at anchorage points in dry shafts. Partial loss of prestress in tendons is generally recognised as local concrete cracking resulting from redistribution of stress and should be investigated upon discovery. Total loss of prestress can result in member collapse. Design documents should be reviewed to establish the arrangement and distribution of cracking that could be expected to result from partial loss of prestress. This information should be documented with the inspection records and made available to the inspection team.

Post-tensioning anchorage zones are commonly areas of complex stress patterns. Because of this, considerable additional reinforcement steel is used to control cracking. In many cases, the reinforcing steel is very congested, and this condition can lead to poor compaction of concrete immediately adjacent to the anchorage. Also, the anchorages for the post-tensioning tendons are generally terminated in prestressing pockets in the structure, and the recess is fully grouted after tensioning and before launch. Experience has also shown that the anchorage zones are prone to distress in the form of localized cracking and spalling of anchorage pocket grout materials. These conditions expose the critical tendon anchors to the marine environment, causing corrosion of the anchor and additional spalling and delamination of concrete and grout in the anchorage zone. Regular visual inspection of the anchorages is recommended. Should evidence exist for potential distress, a more detailed visual inspection supplemented by impact sounding for delaminations should be completed to determine if the anchorage is distressed. The visual inspection should focus on corrosion staining, cracking, and large accumulations of efflorescence deposit.

---end-of-Guidance-note---
A. General

A 100 Environmental Loads

101 Wind, wave, tide and current are important sources of environmental loads (E) on many structures located offshore. In addition, depending on location, seismic or ice loads or both can be significant environmental loads.

102 Loads from wind, wave and current occur by various mechanisms. The most important sources of load are:

- viscous or drag effects, generally of most importance for relatively slender bodies
- inviscid effects due to inertia and wave diffraction. These are generally of most importance in terms of global effects for relatively large volume bodies.

103 For fixed concrete structures, static analyses can be adequate. The possibility that dynamic analysis is required on local components or on the global platform shall be investigated. In the specific case of wave loading, the possibility that non-linear effects can lead to loads at frequencies either above or below the frequency range in the wave spectrum both during temporary floating conditions and at the permanent location shall be investigated. Potential dynamic effects on local or global loads from wave, wind and current sources shall also be investigated.

104 The influence of the structure on the instantaneous water surface elevation shall be investigated. Possible direct impact of green-water on a deck or shafts shall also be investigated. Total water surface elevation depends on storm surge and tide, the crest height of incident waves, and the interaction of the incident waves with the structure or other adjacent structures.

105 Environmental loads due to wind, wave and current relate particularly to the ultimate limit state requirements. In addition, these loads can contribute to the fatigue, serviceability, and accidental limit states. Environmental loads due to wind, wave and current shall also be considered in temporary configurations of the structure during construction, tow, and installation.

106 The estimation of loads due to wind, wave and current requires an appropriate description of the physical environment in the form of seastate magnitude and direction, associated wind magnitude and direction, and relevant current descriptions in terms of current velocity profiles through the depth and associated directional information. The derivation of wind, wave and current combinations required for calculation of loads is described in DNV-OS-101 Sec.3.

107 Procedures for the estimation of seismic loads are provided in DNV-OS-C101 Sec.3.

108 The computation of ice loads is highly specialized and location dependent and is not covered in detail by this Standard. There is an extensive relevant body of literature available for the computation of ice loads that should be consulted for guidance. Ice loads shall be computed by skilled personnel with appropriate knowledge in the physical ice environment in the location under consideration and with appropriate experience in developing loads based on this environment and the load return periods in accordance with DNV-OS-C101 Sec.3.

A 200 Extreme wave loads

201 Wave loads from extreme conditions shall be determined by means of an appropriate analysis procedure supplemented, if required, by a model test program. Global loads on the structure shall be determined. In addition, local loads on various appurtenances, attachments and components shall be determined.

202 The appropriate analysis procedure to compute wave loads generally depends on the ratio of wavelength to a characteristic dimension of the structure, such as the diameter of a column or shafts. For ratios less than approximately 5, a procedure such as diffraction analysis shall be applied that accounts for the interaction of the structure with the incident wave-field. For higher ratios, a slender body theory such as Morison theory may be considered. Where drag forces are important in this regime, both methods should be applied in combination. In some cases, such as in the computation of local loads on various external attachments to a structure, both procedures can be required.

The length of the structure relative to wave length is also of importance for floating structures, as cancellation or reinforcement effects may occur if the wave length corresponds with the length or multiple length of the structure.

203 Model testing shall be considered to supplement analytical results, particularly in cases where it is anticipated that non-linear effects will be significant, or where previous experience is not directly applicable because of the configuration of the structure.

A 300 Diffraction analysis

301 Global loads on large volume bodies shall generally be estimated by applying a validated diffraction analysis procedure. In addition, local kinematics, required in the design of various appurtenances, shall be evaluated including incident, diffraction and (if necessary) radiation effects.

302 The fundamental assumption is that the fluid is inviscid and that the oscillatory motions of both the waves and of the structure are sufficiently small to permit the assumption of linearity. The hydrodynamic interaction between waves and a prescribed structure can be predicted based on linearized three-dimensional potential theory.

303 Analytical procedures shall be implemented generally through well-verified computer programs typically based on source/sink (Green's Function) panel methods or similar procedures. Alternative procedures including classical analytical or semi-analytical methods and the finite element procedure may be considered in specialized cases. Programs should be validated by appropriate methods.

304 Diffraction analysis using panel methods shall be executed with an adequate grid density to provide a solution with the required accuracy. The grid density shall be sufficient to adequately capture fluctuations in parameters such as pressure. In zones where the geometry changes abruptly (corners, edges), denser grids shall be employed. Also, in the vicinity of the free surface, grid densities will generally be increased. Grid densities shall be related to the wave period in order to provide an adequate description of fluctuations over the wavelength. Six panels per wavelength are usually sufficient on a smooth surface. In general, convergence tests with grids of variable density shall be carried out to confirm the adequacy of any proposed panel model.

305 Diffraction models shall be combined with Morison models in the assessment of various relatively slender attachments to large volume structures. Diffraction methods provide local fluid velocity and acceleration required in the Morison model. Morison theory may be applied to compute resulting loads.

306 The proximity of additional relatively large volume
structures shall be included in assessing loads. Disturbances in the wave field around two or more structures may interact and this interaction shall be accounted for in the analysis.

307 Structures with significantly varying cross-section near the waterline, with the likely wave affected zone, call for additional consideration. Non-wall sided structures are not consistent with the underlying assumptions of linear diffraction theory and both local and global loads and load effects can be significantly non-linear relative to the magnitude of the sea-state. Linear diffraction theory assumes wall-sided geometry at the waterline.

308 The calculation of wave forces on surface piercing structures that will be overtopped by the progressing wave need special attention and validation of the computing technique is necessary.

309 Careful consideration shall be given to potential pressure fluctuations on the base of a platform during the passage of a wave field. If the foundation conditions are such that pressure fluctuations are expected to occur on the base, such pressure fluctuations shall be included in the analysis.

310 Diffraction analysis programs may be used to produce coefficients required in the evaluation of various non-linear effects typically involving sum frequency or difference frequency effects.

A 400 Additional requirements for dynamic analysis under wave load

401 In cases where the structure can respond dynamically, such as in the permanent configuration (fixed or floating), during wave load or earthquakes or in temporary floating conditions, additional parameters associated with the motions of the structure shall be determined. Typically, these additional effects shall be captured in terms of inertia and damping terms in the dynamic analysis.

402 Ringing can control the extreme dynamic response of particular types of concrete gravity structure. A ringing response resembles that generated by an impulse excitation of a linear oscillator: it features a rapid build up and slow decay of energy at the resonant period of the structure. In high sea-states, ringing may be excited by non-linear (second, third and higher order) processes in the wave loading that are only a small part of the total applied environmental load on a structure.

403 The effects of motions in the permanent configuration such as those occurring in an earthquake, floating structures or in temporary phases of fixed installations during construction, tow or installation, on internal fluids such as ballast water in tanks, shall be evaluated. Such sloshing in tanks generally affects the pressures, particularly near the free surface of the fluid.

A 500 Model testing

501 The necessity of model tests to determine extreme wave loads shall be determined on a case-by-case basis. Generally, model tests shall be considered when it is required to:

— verify analytical procedures. Model tests should be executed to confirm the results of analytical procedures, particularly in cases with structures of unusual shape, structures in shallow water with steep extreme waves, or in any other case where known limitations of analytical procedures are present

— Complement analytical procedures. Model tests should be executed where various effects such as ringing, wave run up, potential occurrence of deck slamming or in cases where the higher order terms neglected in analytical procedures may be important. These effects cannot usually be assessed in the basic analytical procedure.

502 Froude scaling is considered to be appropriate for typical gravity driven processes like waves acting alone on large volume fixed structures. The influence of viscosity and Reynolds number effects shall be considered in any decision to apply Froude scaling.

503 Where possible, model test loads shall be validated by comparison with analytical solutions or the results of prior appropriate test programs.

504 Appropriate data shall be recorded in model tests to facilitate computation of wave loads. Data in the form of time history recordings may include:

— the local instantaneous air/water surface elevation at various locations
— local particle kinematics
— global loads such as base shear, vertical load or overturning moment as well as local loads as pressure distribution acting on individual components
— structural response such as displacements and accelerations, particularly if dynamic response occurs.

505 Model test data shall be converted to full scale by appropriate factors consistent with the physical scaling procedures applied in the test program.

506 It shall be recognized that, analogous with analytical procedures, model test results have inherent limitations. These limitations shall be considered in assessing the validity of the resulting loads. The primary sources of inherent limitation include:

— surface tension effects. These are not generally allowed for in model test program definition and may be significant particularly where large-scale factors are applied
— viscous effects. The Reynolds number is not generally accurately scaled and these effects are important where viscosity is significant such as in the prediction of drag or damping effects
— air/water mixing and entrainment. Various loads that depend on this type of factor such as slamming forces will not in general be accurately scaled in typical Froude scale based model tests.

507 The influence of different effects on loads determined in model tests shall be assessed and steps taken in the testing program to reduce or minimize them. Such effects might be:

— wave reflections from the ends of model test basins
— scattering of waves from large volume structures and reflection of spurious scattered waves from model basin sidewalls interfering with target design wave conditions
— break down of wave trains representing the target design wave due to various instabilities leading to an inaccurate realisation of design wave conditions
— difficulties in the inclusion of wind or currents in association with wave fields.

A 600 Current load

601 Currents through the depth, including directionality, shall be combined with the design wave conditions. The characteristic current load shall be determined in accordance with DNV-OS-C101 Sec.3.

602 The disturbance in the incident current field due to the presence of the fixed structure shall be accounted for.

603 Current loads on platforms shall be determined using recognized procedures. Typical methods are based on the use of empirical coefficients accounting for area, shape, shielding, etc. Such empirical coefficients shall be validated. Model tests or analytical procedures or both shall be considered to validate computed current loads.

604 Numerical procedures based on Computational Fluid Dynamics (CFD) may be considered in the evaluation of current loads or other effects associated with current. These pro-
 Procedures are based on a numerical solution of the exact equations of the motion of viscous fluids (the Navier Stokes equations). Only well validated implementations of the CFD procedure shall be used in the computation of current effects. The method can provide a more economic and reliable procedure for predicting drag forces than physical modelling techniques.

605 Disturbances in the incident current field lead to modifications in the local current velocity in the vicinity of the structure. Loads on local attachments to the structure shall be computed based on the modified current field. The possibility of Vortex Induced Vibrations (VIV) on various attachments shall be investigated.

606 The presence of water motions in the vicinity of the base of a structure can lead to scour and sediment transport around the base. The potential for such transport shall be investigated. Typical procedures require the computation of fluid velocity using either CFD or model test results. These velocities are generally combined with empirical procedures to predict scouring.

607 If found necessary scour protection should be provided around the base of the structure. See DNV-OS-C101 Sec.11.

A 700 Wind loads

701 Wind loads may be determined in accordance with DNV-OS-C101 Sec.3 E700.

702 Wind forces on a Offshore Concrete Structure (OSC) will consist of two parts:

- wind forces on topside structure
- wind forces on concrete structure above sea level.

703 The wind load on the exposed part of the Offshore Concrete Structure is normally small compared to the wind forces on the topside and to wave load effects. A simplified method of applying the wind load effect to the concrete structure is by using the wind forces derived for the topside structure. These forces will contribute to the overall global loads like the overturning moment and horizontal base shear in addition to increased forces in vertical direction of the concrete shafts.

704 Global mean wind loads on the exposed part of an concrete structure shall be determined, based on the appropriate design wind velocity, in combination with recognized calculation procedures. In a typical case global wind load may be estimated by simplified procedures such as a block method. In this type of procedure wind loads may be based on calculations that include empirical coefficients for simple shapes for which data is available, an appropriate exposed area, and the square of the wind velocity normal to the exposed area. Local wind loads shall generally require inclusion of a gust factor or similar considerations to account for more local variations of wind velocities.

705 Global dynamic effects of wind load shall be investigated if relevant. As an example, a structure and its mooring system in a temporary condition during the construction, towing or installation phases can be susceptible to wind dynamics. An appropriate description of wind dynamics such as a wind spectrum shall be included in wind load estimation.

706 In addition to wind, wave and current loads present at the offshore site, these loads shall also be systematically evaluated where relevant during construction, tow and installation/removal conditions. The complete design life cycle of the structure, from initial construction to removal, shall be considered and appropriate governing design combinations of wind, wave and current shall be assessed in any phase.
APPENDIX B
STRUCTURAL ANALYSES – MODELLING (GUIDELINES)

A. General

A 100 Physical representation

101 Dimensions used in structural analysis calculations shall represent the structure as accurately as necessary to produce reliable estimates of load effects. Changes in significant dimensions as a result of design changes shall be monitored both during and after the completion of an analysis. Where this impacts on the accuracy of the analysis, the changes shall be incorporated by reanalysis of the structure under investigation.

102 It is acceptable to consider nominal sizes and dimensions of the concrete cross-section in structural analysis, provided that tolerances are within the limits set out for the construction and appropriate material partial safety factors are used.

103 Where as-built dimensions differ from nominal sizes by more than the permissible tolerances, the effect of this dimensional mismatch shall be incorporated into the analysis. The effect of tolerances shall also be incorporated into the analysis where load effects and hence the structural design are particularly susceptible to the magnitude (imperfection bending in walls, implosion of shafts, etc.).

104 Concrete cover to nominal reinforcement and positioning of prestressing cables may be provided where these are defined explicitly in detailed local analysis. Again, this is subject to construction tolerances being within the specified limits and appropriate material partial safety factors being applied to component material properties.

105 The effects of wear and corrosion shall be accounted for in the analysis where significant and where adequate measures are not provided to limit such effects.

106 It will normally be sufficient to consider centre-line dimensions as the support spacing for beams, panels, etc. Under certain circumstances, however, face-to-face dimensions may be permitted with suitable justification. The effect of eccentricities at connections shall be considered when evaluating local bending moments and stability of the supporting structure.

107 Material properties used in the analyses of a new design shall reflect the materials specified for construction. For existing structures, material properties may be based on statistical observations of material strength taken during construction or derived from core samples extracted from the concrete.

108 It is normally acceptable to simulate the concrete by equivalent linear elastic properties in most limit states. Unless a different value can be justified, the secant modulus of plain concrete at zero strain may be used as the modulus of reinforced concrete in such an analysis. The value used shall be in accordance with the concrete design rules in use. For loads that result in very high strain rates, the increase in concrete modulus of elasticity should be considered in the analyses of the corresponding load effects.

109 Age effects on the concrete may be included, if sufficiently documented by applicable tests. Effects of load duration and resultant creep of the concrete shall also be considered, where significant. Where loads may occur over a significant period in the life of the structure, the least favourable instance shall be considered in determining age effects.

110 Accurate evaluation of concrete stiffness is particularly important for natural frequency or dynamic analysis, and for simulations that incorporate significant steel components, such as the topsides or conductor framing. Consideration shall be given to possible extreme values of concrete stiffness in such analyses. The aggregate type may influence the stiffness of the concrete and this effect shall be allocated for in the analyses.

111 Non-linear analysis techniques are often applied to local components of the structure. It is typical to discretely model concrete, reinforcement and prestressing tendons in such simulations. Where this is the case, each material shall be represented by appropriate stress-strain behaviour, using recognized constitutive models.

112 The density of reinforced concrete shall be calculated based on nominal sizes using the specified aggregate density, mix design and level of reinforcement, with due allowance for design growth. For existing structures, such densities shall be adjusted on the basis of detailed weight reports, if available. Variation in effective density through the structure shall be considered, if significant.

113 Unless another value is shown to be more appropriate, a Poisson’s ratio of \( v = 0.2 \) shall be assumed for uncracked concrete. For cracked concrete, a value of \( v = 0 \) may be used. A coefficient of thermal expansion of \( 1.0 \times 10^{-5} \degree/\cm \) shall also be used for concrete and steel in lieu of other information. Where the design of the concrete structure is particularly sensitive to these parameters, they shall be specifically determined by the materials in use. Special considerations are required for concrete exposed to cryogenic temperature.

114 The representation of a fixed structure foundation will differ depending on the type of analysis being undertaken. For static analysis, reactive pressures applied to soil contact surfaces shall be sufficient, but for dynamic analysis or where soil/structure interaction is significant, an elastic or inelastic representation of foundation will normally be required to provide suitable stiffness. Seismic analysis is typically very dependent on soil properties, particularly at the ductility level earthquake. Further details of foundation modelling requirements for specific analysis types may be found in Appendix C.

115 Reactions on the structure from its foundation/anchorages shall be based on general principles of soil mechanics in accordance with DNV-OS-C101 Sec.11. Sufficient reactive loads shall be applied to resist each direction of motion of the structure (settlement, rocking, sliding, etc.). The development of hydraulic pressures in the soil that act in all directions should be considered where appropriate. Consideration shall be given to potential variation of support pressures across the base of a fixed concrete structure.

116 The calculations used shall reflect the uncertainties inherent in foundation engineering. Upper and lower bounds and varied patterns of foundation reaction shall be incorporated and an appropriate range of reactive loads shall be assessed. In particular, the sensitivity of structural response to different assumptions concerning the distribution of reaction between the base and any skirts shall be determined.

117 Consideration shall also be given to the unevenness of the seabed, which can potentially cause high local reactions. Foundation unevenness may be considered as a deformational load in subsequent design checks in accordance with Sec.5 B900. Other than this, foundation pressures shall be considered as reactive loads, their magnitude being sufficient to react all other factored loads.

118 Upper limits of soil resistance should be considered during analysis of platform removal.

119 The analyses shall include intermediate conditions, such as skirt penetration and initial contact as well as the fully grouted condition, if significant. Disturbance of the seabed due to the installation procedure should be considered in calculat-
ing subsequent foundation pressures.

120 Where it significantly affects the design of components, soil interaction on conductors shall also be incorporated in the analysis, particularly with regard to local analysis of conductor support structures.

121 Other than direct support from foundation soils, a component may be supported by:
- external water pressure, while floating
- other components of the structure
- anchor supports
- any combination of the above, and foundation soils.

122 The load of water pressure in support of a fixed concrete structure while floating or a floating concrete structure shall be evaluated by suitable hydrostatic or hydrodynamic analysis and shall be applied to appropriate external surfaces of the structure.

123 Representative boundary conditions shall be applied to the analysis of a component extracted from the global structure. These boundary conditions shall include possible settlement or movement of these supports, based on a previous analysis of the surrounding structure.

124 In the absence of such data, suitable idealized restraints should be applied to the boundary of the component to represent the behaviour of surrounding structure. Where there is uncertainty about the effective stiffness at the boundaries of the component, a range of possible values shall be considered.

125 Force, stiffness or displacement boundary conditions may be applied as supports to a component. Where there is uncertainty as to which will produce the most realistic stresses, a range of different boundary conditions shall be adopted and the worst load effects chosen for design.

126 Where components of the structure are not fully restrained in all directions, such as conductors within guides and bearing surfaces for deck and bridge structures, allowance shall be made in the analysis for movement at such interfaces.

A 200 Loads

201 Loads shall be determined by recognized methods, taking into account the variation of loads in time and space. Such loads shall be included in the structural analysis in a realistic manner representing the magnitude, direction and time variance of such loads.

202 Permanent and live loads shall be based on the most likely anticipated values at the time of the analysis. Consideration shall be given to minimum anticipated values as well as maximum loading. The former governs some aspects of the design of gravity-based structures.

203 Hydrostatic pressures shall be based on the specified range of fluid surface elevations and densities. Hydrostatic pressures on floating structures during operation, transportation, installation and removal stages shall include the effects of pitch and roll of the structure due to intentional trim, wind heel, wave load or damage instability. The above also apply to fixed structures under transportation, installation and removal phases.

204 Prestressing effects shall be applied to the model as external forces at anchorages and bends, or as internal strain compatible effects. In both cases, due allowance shall be made for all likely losses in prestressing force. Where approximated by external reactions, relaxation in tendon forces due to the effect of other loads on the state of strain in the concrete shall be considered.

205 Thermal effects are normally simulated by temperatures applied to the surface and through the thickness of the structure. Sufficient temperature conditions shall be considered to produce maximum temperature differentials across individual sections and between adjacent components. The temperatures shall be determined with due regard to thermal boundary conditions and material conductivity. Thermal insulation effects due to insulating concrete or drill cuttings shall be considered, if present.

206 Wave, current and wind loads shall include the influence of such loads on the motion of the structure while floating. In cases where dynamic response of the structure may be of importance, such response shall be considered in determining extreme load effects. Pseudo-static or dynamic analyses shall be used.

207 Uncertainties in topsides centre of gravity, built-in forces and deformations from transfer of topsides from barges to the concrete structure shall be represented by a range of likely values, the structure being checked for the most critical extreme value.

208 Structures designed to contain cryogenic gas (LNG) shall additionally be designed in accordance with the provisions made in DNV-OS-C503.

A 300 Mass simulation

301 A suitable representation of the mass of the structure shall be prepared for the dynamic analysis, motion prediction and mass-acceleration loads while floating. The mass simulation shall include relevant quantities from at least the following list:
- all structural components, both steel and concrete, primary and secondary
- the mass of all intended equipment, consistent with the stage being considered
- the estimated mass of temporary items, such as storage, lay-down, etc
- masses of any fluids contained within the structure, including equipment and piping contents, oil storage, LNG storage, flooding, etc
- the mass of solid ballast within the structure
- snow and ice accumulation on the structure, if significant
- drill cuttings or other deposits on the structure
- the mass of marine growth and external water moving with the structure
- added water mass
- added soil mass.

302 The magnitudes of masses within the structure shall be distributed as accurately as necessary to determine all significant modes of vibration (including torsional modes) (when required) or mass-acceleration effects for the structural analysis being performed. Particular attention shall be paid to the height of topsides equipment or modules above the structural steelwork.

303 It is normally necessary to consider only the maximum mass associated with a given analysis condition for the structure. For dynamic analyses, however, this may not produce the worst response in particular with respect to torsional modes and a range of values of mass and centre of gravity may have to be considered. For fatigue analysis, the variation in load history shall be considered. If appropriate, an average value over the life of the structure may be used. In such cases, it is reasonable to consider a practical level of supply and operation of the platform.

NOTE Calculation of the added mass of external or entrained water moving with the structure shall be based on best available published information or suitable hydrodynamic analysis. In lieu of such analysis, this mass may be taken as the fullmass of displaced water by small-submerged members, reducing to 40% of the mass of displaced water by larger structural members. Added mass effects may be ignored along the axial length of prismatic members, such as the shafts.
A 400 Damping

401 Damping arises from a number of sources including structural damping, material damping, radiation damping, hydrodynamic damping and frictional damping between moving parts. Its magnitude is dependent on the type of analysis being performed. In the absence of substantiating values obtained from existing platform measurements or other reliable sources, a value not greater than 3% of critical damping may be used.
APPENDIX C
STRUCTURAL ANALYSES (GUIDELINES)

A. General

A 100 Linear elastic static analysis

101 It is generally acceptable for the behaviour of a structure or component to be based on linear elastic static analysis unless there is a likelihood of significant dynamic or non-linear response to a given type of loading. In such cases, dynamic or non-linear analysis approaches shall be required, as defined in A200 – A400.

102 Static analysis is always permissible where all actions on the component being considered are substantially invariant with time. Where actions are periodic or impulsive in nature, the magnitude of dynamic response shall be evaluated in accordance A200 and static analysis shall only be permitted when dynamic effects are small.

103 Reinforced concrete is typically non-linear in its behaviour, but it is generally acceptable to determine global load paths and sectional forces for ultimate, serviceability and fatigue limit states based on an appropriate linear elastic analysis, subject to the restrictions presented below. Non-linear analysis shall generally be required for accidental limit states, ductility level earthquakes and local analysis.

104 Linear stiffness is acceptable provided that the magnitudes of all actions on the structure are not sufficient to cause significant redistribution of stresses due to localised yielding or cracking. Response to deformatonal loads, in particular, is very susceptible to the level of non-linearity in the structure and shall be carefully assessed for applicability once the level of cracking in the structure is determined.

105 Reduction of the stiffness of components should be considered if it can be shown that, due to excessive cracking, for example, more accurate load paths might be determined by such modelling. Such reduced stiffness shall be supported by appropriate calculations or by non-linear analysis.

106 A linear analysis preserves equilibrium between external applied loads and internal reaction forces. Linear solutions are thus always equilibrium states. The equations of a linear system need to be solved only once and the solution results may be scaled to any load level. A solution is hence always obtained, irrespective of the load levels. Linear analysis can be carried out for many independent load cases at the time. The independent load cases may be superimposed into combined cases without new solution of the equation system.

NOTE: Practise has shown that the use of a system representing all actions as unit loadcases that afterwards can be scaled in magnitude and added to represent complete load combinations i.e. loading scenarios is very effective.

A 200 Dynamic analysis

201 Fixed structures with natural periods of the global structure greater than 2.5s can be susceptible to dynamic response due to wave action during in-service conditions, at least for fatigue assessment. Structures in shallow water or subject to extreme wave conditions may exhibit significant dynamic response at lower periods due to the higher frequency content of shallow water or particularly steep waves.

202 Other load conditions to which the structure may be subjected, such as sea tow, wind turbulence, vibration, impact and explosion can also impose dynamic forces of significant magnitude close to fundamental periods of the structure or its components. Structures that respond to a given set of actions by resonant vibration at one or more natural periods shall be assessed by dynamic analysis techniques.

203 Earthquakes are a particularly severe form of oscillatory loading that shall always require detailed dynamic analysis if the zone of seismic activity produces significant ground motions.

204 Where dynamic effects can be significant, dynamic response can be evaluated on the basis of a simplified representation of the structure or by the calculation of natural periods and the evaluation of dynamic amplification factors. In evaluating dynamic amplification factors for wave loading, consideration shall be given to higher frequency components of wave and wind action that occur due to drag loading, sharp crested shallow water waves, finite wave effects, ringing, etc.

205 Where substantial dynamic response of the structure is predicted, having magnitude at critical sections exceeding that predicted by static only analysis, detailed dynamic analysis shall be required. Dynamic analysis shall also be required where more than one fundamental mode of the structure is significantly excited by the applied actions, as is the case for seismic response.

206 Where dynamic effects are relatively insignificant, a pseudo-static analysis of the structure or its components may be performed, including dynamic effects in accordance with Appendix C A300.

207 Where dynamic response is likely to be significant; full dynamic analysis shall be performed to quantify such effects. Appropriate mass and damping simulations shall be applied to the structure to enable the natural modes of vibration to be determined with accuracy.

208 Dynamic analysis will normally require a linearized simulation of the soil stiffness for in-service conditions. This stiffness shall be determined with due allowance for the expected level of loading on the foundation. Specific requirements for seismic analysis are presented in Appendix D.

209 Actions applied to the structure or component shall include all frequency content likely to cause dynamic response in the structure. The relative phasing between different actions shall be rigorously applied.

210 Harmonic or spectral analysis methods are suitable for most forms of periodic or random cyclic loading. Where significant dynamic response is coupled with non-linear loading, or non-linear behaviour of the structure, component or foundation, then transient dynamic analysis shall be required.

211 Where modal superposition analysis is being performed, sufficient modes to accurately simulate structural response shall be included; otherwise, a form of static improvement shall be applied to ensure that static effects are accurately simulated.

212 For impulse actions, such as ship impacts, slam loads and blast loading, dynamic amplification effects may be quantified by the response of single- or multi-degree of freedom systems representing the stiffness and mass of the components being analysed. Transient dynamic analysis will normally be required.

A 300 Pseudo-static analysis

301 In this context, pseudo-static analysis refers to any analysis where dynamic actions are represented approximately by a factor on static loads or by equivalent quasi-static actions. The former approach is appropriate where static and dynamic action effects give an essentially similar response pattern within the structure, but differ in magnitude.

302 For the former approach, dynamic amplification factors shall be used to factor static only response. Such factors will,
in general, vary throughout the structure to reflect the differing magnitudes of static and dynamic response. For platform columns or shafts, appropriate local values of bending moment should be used. Base shear, overturning moment and soil pressure are representative responses for the platform base.

303 For the latter approach, additional actions shall be applied to the structure to represent dynamic mass-acceleration and inertial effects. All actions applied in a pseudo-static analysis may be considered constant over time except in the case of non-linear response, where knowledge of the load history may be significant and loading should be applied to the simulation in appropriate steps.

304 Factored dynamic results shall be combined with factored static effects due to gravity, etc. in accordance with the limit states being checked. Load partial safety factors for dynamic loads should be consistent with the loading that causes the dynamic response, normally environmental. The most detrimental magnitude and direction of dynamic loading shall be considered in design combinations.

A 400 Non-linear analysis

401 Non-linear behaviour shall be considered in structural analysis when determining action effects in the following cases:

— where significant regions of cracking occur in a structure such that global load paths are affected
— where such regions of cracking affect the magnitude of actions (temperature loads, uneven seabed effects, dynamic response, etc.)
— where the component depends upon significant non-linear material behaviour to resist a given set of loads, such as in response to accidents or ductility level seismic events
— for slender members in compression, where deflection effects are significant (imperfection bending of buckling).

402 A non-linear analysis is able to simulate effects of geometrical or material nonlinearities in the structure or a structural component. These effects increase as the loading increases and require an application of the loading in steps with solution of the equations a multiple of times. The load must be applied in steps or increments, and at each loading step, iterations for equilibrium must be carried out.

403 Non-linear solutions can not be superimposed. This implies that a non-linear analysis must be carried out for every load case or load combination, for which a solution is requested.

404 Non-linear analysis of the global structure or significant components may be based on a relatively simple simulation model. Where linear elastic elements or members are included in this simulation, it shall be demonstrated that these components remain linear throughout the applied actions. Appropriate stress-strain or load deflection characteristics shall be assigned to other components. Deflection effects shall be incorporated if significant.

405 Non-linear analysis of components to determine their ultimate strength shall normally be performed on relatively simple simulations of the structure or on small components, such as connections. Complex non-linear analysis of such D-regions using finite element methods should not be used without prior calibration of the method against experimental results of relevance. Material properties used in non-linear analysis should be reduced by appropriate material partial safety factors, in accordance with Sec.5. Where components of the structure rely upon nonlinear or ductile behaviour to resist extreme actions, such components shall be detailed to permit such behaviour, in accordance with Sec.6.
APPENDIX D
SEISMIC ANALYSIS (GUIDELINES)

A. General

A 100 Seismic analysis

101 Two levels of seismic loading on an offshore concrete structure shall be considered:
   — strength level earthquake (SLE), which shall be assessed as a ULS condition
   — ductility level earthquake (DLE), for which ductile behaviour of the structure assuming extensive plasticity is permissible provided the structure survive.

LNG containment structures shall be designed in accordance with DNV-OS-C503. The LNG storage tank shall be designed for both the SLE and DLE earthquakes. Systems which are vital for the plant system shall remain operational for both SLE and DLE.

102 If ductile response of specific components of the structure under the SLE event is predicted or considered in the analysis, such components shall be designed for ductile behaviour, in accordance with Sec.6. Expected best estimate of stress strain parameters associated with ductile behaviour may be adopted in the analyses. Due consideration shall be given to the effects of overstrength with respect to the transfer of forces into adjoining members, and for the design of those failure modes of such members that are not ductile, such as shear failure. For those cases where the structure can be designed to the DLE event applying normal ULS criteria, no special detailing for ductility is required.

103 Seismic events may be represented by input response spectra or by time histories of significant ground motion, in accordance with DNV-OS-C101 Sec.3 E800. Where the global response of the structure is essentially linear, a dynamic spectral analysis shall normally be appropriate. Where non-linear response of the structure is significant, transient dynamic analysis shall be performed.

104 Seismic response of a structure is highly dependent on the natural periods of the structure over a range of modes. This relies upon accurate assessments of the platform mass and stiffness, and a best estimate of soil stiffness. Such parameters shall be carefully assessed and, if necessary, the sensitivity of the structure to changes in these parameters shall be evaluated.

105 Interaction of the structure with its foundation is particularly significant for seismic analysis. The foundation shall be simulated with sufficient accuracy in global structural analysis to ensure accurate assessment of natural periods of vibration and a suitable distribution of soil loads into the structure.

106 Two principal types of seismic analyses are suggested for fixed concrete structures:
   — direct soil-structure analysis
   — impedance function/substructure analysis.

107 For direct soil-structure analysis, the caisson may be modelled as a rigid structure connected to a flexible simulation of the foundation. In substructure analysis, the caisson may be considered as a rigid circular disk for the computation of impedance functions.

108 Consideration shall be given to the range of likely values of soil stiffness in the analysis. In particular, the possible degradation of soil properties during high-level seismic events, such as the ductility level earthquake, shall be considered. Appropriate non-linear or reduced soil stiffness properties shall be used.

109 Soil properties, particularly shear wave velocity, dynamic shear modulus and internal damping are dependent on the shear strains used. These values should be adjusted for the expected strains appropriate to the seismic excitation and the variation in vertical effective stress and voids ratio due to the presence of the structure.

110 The simulation shall include a representation of the mass of the structure, in accordance with Appendix B A300. Enclosed fluids can be included as a lumped mass where the height of water column is short.

111 Unless a detailed evaluation is made, critical internal damping of not more than 5% shall be used to simulate structural and hydrodynamic damping for seismic analysis. Any increased value shall be subject to justification based on expected response. Values of soil damping shall be determined based on the soil type present.

112 For the strength level earthquake, linear dynamic global structural analysis may be performed using the response spectrum method (CQCM) and directionally using a square root sum of squares (SRSS) approach. Alternative methods are permitted based on site-specific data. These spectra may be combined in the vertical direction unless a lesser value can be justified based on site-specific data. These spectra may be combined together modal using the complete quadratic combination method (CQCM) and directionally using a square root sum square (SRSS) approach. Alternative methods are permitted with suitable justification that all seismic action effects are included.

113 Secondary spectra may be developed for the analysis of components such as deck or conductor frames to evaluate the response of substructures, appurtenances and equipment not modelled for the global analysis. Alternatively, the design of local components may be based on equivalent pseudo-static analysis of such components, based on maximum vertical and horizontal accelerations obtained from the global seismic analysis.

114 One design spectrum may be used equally in each principal horizontal direction, combined with 2/3 of this spectrum in the vertical direction unless a lesser value can be justified based on site-specific data. These spectra may be combined together modal using the complete quadratic combination method (CQCM) and directionally using a square root sum square (SRSS) approach. Alternative methods are permitted with suitable justification that all seismic action effects are included.

115 For the ductility level earthquake, non-linear seismic
analysis may be performed using a time history or transient approach. Unless time histories are available by scaling or by other means, they may be developed numerically from the design spectra. Multiple time histories are required to represent the random nature of seismic ground motions. At least three sets of tri-axial statistically independent time histories shall be developed and used, and the maximum response parameters shall be adopted in the design. The requirement for statistically independent time histories may be considered satisfied if the correlation coefficient between any pair of time histories is less than 0.3. The time step for integration shall be selected to ensure accuracy and stability of the non-linear dynamic solution, normally it should not be more than $1/(12f)$.

118 The computer model for ductility level earthquake analysis shall include discrete models of all primary components of the structure using either linear elastic or material non-linear simulations. Deflection effects shall be evaluated and gravitational loads included in the analysis to ensure that second order effects are simulated with sufficient accuracy.

119 The action effects on components that are simulated as linear elastic in either the SLE or the DLE analyses shall be evaluated and used to confirm that these components satisfy ULS criteria. Components that demonstrate ductile response shall be so designed, and assessed against acceptance criteria relevant for the actual limit state with respect to all relevant response parameters.
APPENDIX E
USE OF ALTERNATIVE DETAILED DESIGN STANDARD (GUIDELINES)

A. General

A 100 Introduction

101 The detailed design may be carried out in accordance with Sec.6, the detailed requirements for concrete design. An alternative detailed reference standard may be found acceptable provided the standard satisfy the following conditions.

102 The detailed design shall be carried out in accordance with a recognized reference standard, covering all aspects relevant for the structural design of offshore concrete structures. This chapter identifies areas of the detailed design standard that shall be checked, for adequate coverage. For complex structures, where higher grades of concrete are used, and where the loading conditions are severe, most or all of the items in E200 shall be covered.

Guidance note:
The detailed design reference standard to be used should be agreed at an early stage in a project, as the choice of standard might strongly influence the platform geometry and dimensions.

---end---of---Guidance---note---

A 200 Conditions

201 The reference standard shall give the design parameters required for the type of concrete, e.g. normal weight or lightweight concrete, and strength class used. For high strength concretes and lightweight concrete, the effect of reduced ductility shall be considered. This in particular applies to the stress/strain diagram in compression, and the design parameter used for the tensile strength in calculation of bond strength, and transverse shear resistance.

202 Shell types of members are typical in offshore structures; the reference standard shall cover design principles applicable to members such as domes and cylinders, where relevant. The design methods shall be general in nature, considering equilibrium and compatibility of all the six force components giving stresses in the plane of the member and all limit states.

203 The reference standard shall give the principles required for the design for transverse shear, where the general condition of combinations of simultaneously acting in plane forces, e.g. tension and compression and transverse forces shall be covered. The interaction dependant of directionality of same forces in members like shells, plates and slabs shall be included. Due consideration shall be given to the handling of action effects caused by imposed deformations.

204 The reference standard shall give principles required for the design for fatigue for all failure modes. This includes e.g. concrete in compression/compression or compression/tension, transverse shear considering both shear tension and shear compression, reinforcement considering both main bars and stirrups including bond failure, and prestressing reinforcement. Material standards might give certain fatigue-related requirements; these are normally not adequate for offshore applications. The fatigue properties will vary significantly also for materials that pass such general requirements for fatigue. For the design SN-curves representing the 5% fractile should be prepared for rebars, and in particular for items that have stress concentrations such as couplers, end anchors and T-heads.

205 The reference standard should give the principles and criteria applicable to ensure a durable design in marine environment. Important in this context is:

— the selection of adequate materials, which shall be in accordance with Sec.4
— adequate concrete cover to reinforcement, see Sec.6 Q200.
— limitation of crack-widths under SLS conditions, see Sec.5 O200.

206 The reference standard shall give the principles for tightness control. Tightness shall be considered under SLS conditions. This shall apply to ingress of water in structures in floating conditions and in installed condition when having internal underpressure as well as leakage in particular of stored hydrocarbons from structures having internal overpressure. Leakage shall also be considered in the design of the members that are affected when maintaining a pressure gradient is vital like in suction foundations, and when using air cushions.

207 Adequate tightness or leakage control shall be required in ULS and ALS for those conditions where a leakage might cause collapse or loss of the structure due to flooding or where a pressure condition required to maintain equilibrium might be lost.

208 The reference standard shall give the design principles required for design of prestressed concrete, including principles for partial prestressing, when appropriate.

209 The effect of the presence of empty ducts during phases of the construction period shall be considered. For the final condition, the effect of the presence of ducts on the capacity of cross-sections shall be considered, in particular if the strength and stiffness of the grout is less than that of the concrete. This also applies if the ducts are not of steel but of flexible materials.

210 The reference standard shall give the principles required to design all relevant types of members for second-order effects, including buckling also in the hoop direction of shell types of members.

211 The reference standard shall give the principles required in order to assess the effects of water pressure penetrating into cracks and pores of the concrete, affecting both the load effects and the resistance. The methods to be used will be dependent of how water pressure is applied in the initial calculation of action effects.

212 The reference standard shall give the principles for the local design in discontinuity regions where strut and tie models might be used to demonstrate the mechanisms for proper force transfer.

213 The reference standard shall give the principles required to permit design for imposed deformations based on strains rather than forces, in all limit states. Where brittle failure modes are involved, such as shear failure in members with no transverse reinforcement, conservative design parameters shall be assumed in order not to underestimate the risk of the potential brittle failure modes.

214 The reference standard shall give guidance for how to assess the effect of gain in strength beyond 28 days and also the effect of sustained loads or repeated loads at high stress levels in reduction of strength of concrete, when the gain in strength is intended for use in the design.

215 The reference standard shall give design principles required for demonstration of adequate fire resistance of members subjected to fire, including relevant material and strength parameters at elevated temperatures.

216 In zones with low to moderate seismic activity, the action effects obtained from an analysis in which the platform structure is modelled as linear elastic will normally be such that the structural design can be performed based on conven-
tional linear elastic strength analyses, employing normal design and detailing rules for the reinforcement design.

217 In cases where the seismic action cause large amplitude cyclic deformations which can only be sustained employing plasticity considerations, the reference standard shall give adequate requirements concerning design and detailing. The regions of the structure that are assumed to go into plasticity experiencing excessive deformations shall be carefully detailed to ensure appropriate ductility and confinement.

218 The material factors shall be such that a total safety level consistent with this standard is obtained. This shall be documented.
APPENDIX F
CRACKWIDTH CALCULATION (GUIDELINES)

A. General

A 100 Introduction

101 The general basis for calculation of crackwidth in an offshore structure is provided in Sec.6 O700.

102 This Appendix provides recommendations for calculation of crackwidth for stabilized crack pattern. Stabilized crackpattern is defined as a crackpattern developed in such a way that an increase in the load will only lead to minor changes in the number, spaces between cracks and direction of cracks.

103 Normally, a stabilized crackpattern is used in evaluation of crackwidth as the provision of minimum reinforcement in the structure is intended to ensure a well spaced developed crackpattern.

A 200 Stabilized crackpattern

201 Influence length, \( l_{sk} \)

For stabilized crackpattern, the influence length, \( l_{sk} \), equals the characteristic distance between cracks, \( s_{rk} \).

The characteristic distance between cracks for cracks normal to the reinforcement direction is predicted from the following formulae:

\[
s_{rk} = 1.7 \cdot s_{m} = 1.7 \cdot \left( s_{ro} + k_{c} \cdot A_{cef} / \Sigma [\pi \cdot \phi / (f_{tk} \cdot k_{b} / \tau_{bk})] \right)
\]

where the summation, \( \Sigma \), covers tensile reinforcement within the concrete area influencing the transfer of tensile stresses between concrete and tension reinforcement between cracks, \( A_{cef} \).

202 In plates and slabs with single bars or bundles of bars of equal diameter and constant spacing between the bars, the distance between the cracks may be calculated from:

\[
s_{rk} = 1.7 \cdot s_{m} = 1.7 \cdot \left( s_{ro} + (f_{tk} / \tau_{bk}) \cdot k_{b} \cdot k_{c} \cdot h_{cef} \cdot s_{b} / (\pi \cdot \phi) \right)
\]

where:

\[
s_{ro} = 20 \text{ mm (a constant length with presumed loss of bond)}
\]

\[
f_{tk} / \tau_{bk} = \text{the effective ratio between tensile strength and bond strength and is taken as 0.75 for deformed bars, 1.15 for post-tension bars and 1.50 for plain bars.}
\]

\[
A_{cef} = b \cdot h_{cef}, \text{ the effective concrete area in the part of the concrete tension zone which is presumed to participate in carrying tensile stresses which is transferred from the reinforcement to the concrete by bond.}
\]

\[
b = \text{the width of the effective concrete section considered (mm)}
\]

\[
h_{cef} = \text{the height of the effective concrete area = 2.5 (h - d), where (h - d) is the distance from the concrete surface on tension side to the centre of gravity of the reinforcement. For a tension zone with reinforcement of single tensile bars in one layer, } h_{cef} = 2.5 (c + \phi / 2).
\]

\[
h_{cef} \text{ shall be less than the height of the tensile zone (h - x), where x is the distance from the concrete edge on the tensile side to the neutral axis and h is the total cross-sectional height.}
\]

For double reinforce cross-sections with through going tensile stresses, \( h_{cef} \) is calculated for each side, \( h_{cef} \) shall in this case never be larger than \( h/2 \).

\[
k_{c} = \text{a coefficient which accounts for the strain distribution within the cross-section. } k_{c} = (1 + \epsilon_{y} / \epsilon_{1}) / 2 \text{ where } \epsilon_{y} / \epsilon_{1} \text{ is the ratio between minimum and maximum strain in the effective concrete area calculated for cracked cross-section. For a cross-section with through going tensile stresses, } k_{c} = 1.0.
\]

\[
k_{b} = 0.15 \cdot n + 0.85, \text{ a coefficient which accounts for reduced bond of bundled reinforcement.}
\]

\[
c = \text{the concrete cover for the reinforcement under investigation.}
\]

\[
\phi = \text{the diameter of the reinforcement bar}
\]

\[
s_{b} = \text{the distance between reinforcement bars or bundles of bars, maximum value in the calculation 15} \cdot \phi / n.
\]

\[
n = \text{number of bars in a bundle.}
\]

203 Characteristic distance between cracks, \( s_{sk} \), shall not be larger than 2.5 (h - x) and not less than 2.5 c, where c < (h-x).

204 Should the reinforcement be distributed unevenly between different parts of the cross-section, then the characteristic distance between the cracks, \( s_{sk} \), shall be predicted individually for groups with similar intensity of reinforcement.

205 For reinforcement with perpendicular reinforcement bars spaced at a distance, \( s \), then the characteristic distance between the cracks can be taken as \( n \cdot s \), where \( n \) is a whole number, and when the predicted distance between the cracks is greater than \( n \cdot s \) and less than \( (n + 0.3) \cdot s \).

A 300 Distance between cracks with deviations between the principle strain directions and the direction of the reinforcement

301 When the principal strain deviate from the direction of the reinforcement, then the distance between the crackwidth in the direction of the main reinforcement may be predicted from:

\[
s_{m} = \frac{1}{\left(\sin \nu/s_{mx} + \cos \nu/s_{my}\right)}
\]

where:

\[
\nu = \text{the angle between the principle strain and the y-direction (x-direction) when the reinforcement is presumed to be position in the x–direction (y-direction).}
\]

\[
s_{mx} = \text{the predicted distance between the cracks in the x-direction.}
\]

\[
s_{my} = \text{the predicted distance between the cracks in the y-direction.}
\]

A 400 General Method

The mean tensile strain, \( \epsilon_{sm} \), may be calculated using the principles outlined in Sec.6 H "General Design Method for Structural Members subjected to In-Plane Forces". The mean strain may be calculated based on the assumption that the concrete contribute between the cracks with an average tensile stress, \( \sigma_{c} f_{tk} \), and a corresponding strain, \( \epsilon_{cm} = \beta_{s} f_{tk} / E_{ck} \), where \( \beta_{s} \) is the ratio between the mean tensile stress and the tensile strength of the concrete in the influence area of the characteristic crack.

\[
\beta_{s} = 0.6 \text{ for short duration one time loadings}
\]

\[
0.4 \text{ for long duration or repeated loads at actual load level.}
\]

\[
E_{ck} = 9500 (f_{ck})^{0.3}
\]

A 500 Simplified Approach

The crackwidth may be calculated by the following simplified equation:
\[ w_k = s_{rk} \left\{ (1 - \beta_s \sigma_{sr2}/\sigma_{s2}) \sigma_{s2}/E_{sk} - \varepsilon_{cs} \right\} \]

Where:

- \( \sigma_{s2} \) = the stress in the reinforcement in the crack for the actual cross-sectional forces
- \( \sigma_{sr2} \) = the reinforcement stress at the crack location for those cross-sectional forces which give maximum tensile stress in the reinforcement at cracking of the concrete (max tensile stress in concrete equal to tensile strength). The calculation of reinforcement stress is based on cracked concrete.

\[ s_{rk} = \text{See A200 above.} \]

\( \sigma_{sr2} \) is calculated based on the same ratio between the cross-sectional forces (the same location of the neutral axis) as used in the calculation of, \( \sigma_{s2} \), and shall not be larger than \( \sigma_{s2} \).

For structures exposed to water pressure, the reinforcement stress, \( \sigma_{s2} \), shall include the effect of full water pressure, \( p_w \), on the crack surface.

Additional simplification may be made by presuming \( \beta_s = 0 \), thus neglecting the shrinkage strain.