DNV·GL

STANDARD

DNVGL-ST-C502 Edition February 2018

Offshore concrete structures

DNV GL AS

FOREWORD

DNV GL standards contain requirements, principles and acceptance criteria for objects, personnel, organisations and/or operations.

Any comments may be sent by e-mail to *rules@dnvgl.com*

CHANGES – CURRENT

This document supersedes the August 2017 edition of DNVGL-ST-C502.

Changes in this document are highlighted in red colour. However, if the changes involve a whole chapter, section or sub-section, normally only the title will be in red colour.

Changes February 2018

• General

— [4.9.3.2] - [4.9.3.7] are relocated to new [6.1.8] *Structures reinforced with FRP*.
— App.J *Mock-up test requirements* is new. Input, best practice and minimum requirements for mock up testing of structures to be filled with structural grout are described.
— App.K *Supplemental requirements for steel reinforcement specified for use with this standard* is new. The appendix is used in combination with the base requirements for ISO 6935-2 for specifying reinforcement bars to be used when structures are designed in accordance with this standard.

• Sec.1 Introduction

— Sec.1 has been restructured.

• Sec.3 Design documentation

— Subsection [3.1.4] has had minor updates to include documentation relevant for structures with grouted connections.

• Sec.4 Materials

— Clarifications provided to general assumptions regarding the constituents of concrete and grout in this standard, see relevant subsections of [4.2.1].
— Clarification provided on the approval of materials, see relevant subsections in [4.1.1.].
— Clarifications provided regarding fibre reinforced concrete properties, fibre distribution and associated mock-up test requirements for fibre reinforced concrete. Also requirements added to document the increase in tensile strength of fibre reinforced concrete. See relevant subsections of [4.4.1].
— The range of normal and high strength concrete is changed. A minimum grade for concrete exposed to sea water is introduced along with one for concrete subject to abrasion. Normal weight (NW) concrete grade properties for C25 have been removed and properties for C65 included. Additionally, for concrete grade larger than C65 the characteristic uniaxial tensile strength shall be documented through testing. See relevant subsections of [4.3].
— Expanded on freeze-thaw resistance and air content readings, see [4.3.2.6].
— Requirements for documenting the properties of lightweight aggregate concrete clarified in [4.3.3.8].
— Requirement to document the conversion factor between the test specimen sizes used for characteristic strength testing and normal QC testing added for concrete [4.3.3.10].
— Formula added for normalized value of Young’s Modulus for compressive cylinder strength larger than 70MPa for concrete [4.3.3.15] and grout [4.5.1.2].
— Formula for estimating the Young’s Modulus of fibre reinforced concrete and fibre reinforced grout are suggested until the values are found through testing, see [4.4.1.2] and [4.6.1.2].
— Clarifications provided regarding fibre reinforced grout properties, fibre distribution and associated mock-up test requirements for fibre reinforced grout. Also requirements added to document the increase in tensile strength of fibre reinfoced grout. See relevant subsections of [4.6.1].

— The requirements towards the minimum characteristics and properties of various types of steel reinforcement to be used with the standard is clarified with a gap analysis being necessary for steel specified to different national regulations, see relevant subsections in [4.7.1].
— The documentation requirements for embedded items, repair and non-cementitious materials have been updated. Detail also added for the specification and testing of repair materials. See relevant subsections in [4.1.2], [4.13.1] and [4.13.2].
— Testing requirements for grout and steel reinforcement have been updated, see [4.14].

• Sec.5 Loads and analyses requirements

— Additional formula for normalized value of Young's modulus is included for characteristic compressive cylinder strength of concrete larger than 70 MPa for concrete [5.5.2.1]
— Old subsection is deleted.
— Guidance is given for design requirements for the seismic ELE event in [5.1.4.2] and [5.3.2.2].
— Clarification has been provided for the abnormal environmental loading design case, see [5.3.4.2] and [5.3.4.4].
— Additional guidance has been provided for defining load categories in design [5.4.1.10] and [5.4.1.11].
— Testing is required to establish the Young's modulus and creep coefficient for lightweight aggregate concrete, see [5.5.2.5] and [5.5.3.7].
— Requirements clarified for design of structures with an intended underpressure [5.5.4.6].
— Requirements for designing for water pressure in cracks have been modified, see [5.5.4.5].

• Sec.6 Detailed design of offshore concrete structures

— Requirements for tightness post ALS event have been clarified, see [6.2.2.7].
— Requirement (0,9f_y) introduced to prevent permanent strain in the reinforcement of structures exposed to marine environment [6.2.2.5] and [6.2.2.6].
— Material coefficients have been updated and their application in design clarified, an α_c-factor equal to 0,85 for ULS/ALS/SLS loading is incorporated in the formulation for f_{cd} and E_{cd} and, formula and requirements to document the stress strain relationship for concrete have also been updated. Additionally tolerance requirements have been modified. See relevant subsections of [6.3].
— Subsection [6.4.1] on tensile strength of the fibre reinforced concrete is split in two parts; one new part [6.4.1.9] for primary members and one part [6.4.1.10] for secondary members.
— New subsection included on stiffness effect of embedded items [6.3.6.4].
— Clarifications and new subsections added related to shear in slabs and beams including limitation of the concrete shear compression failure capacity, validity of formula for grades higher than C65 and anchorage of fittings/load carrying components. See relevant subsections of [6.6].
— Clarifications have been provided for the treatment of transverse shear in the design of shear walls, plates and shell elements in [6.8.1.4] and [6.8.2.3].
— Some requirements for design of D-regions has been clarified, see relevant subsections in [6.9].
— For concrete grades larger than 65 MPa validity of formula for the design shear strength, τ_{cd}, shall be documented by testing, see [6.10.1.7].
— Guidance is provided for the detailing, construction and design of structures with T-headed reinforcement [6.3.7.1] and [6.11.1.23].
— For concrete grades larger than 65 MPa validity of formula for the design bond strength, f_{bd}, shall be documented by testing, see [6.11.1.16].
— Clarifications made for design of structures for ALS, see relevant subsections of [6.14].
— Concrete cover for bars, pre-tension reinforcement and ducts have been updated, see [6.17.2.1]
— The general requirements for bending of reinforcement have been clarified, see [6.17.4].
— Some clarifications related to minimum reinforcement for slabs/plates, beams, columns and walls, see relevant subsections of [6.17].

• Sec.7 Construction

— Included requirements for documentation and inspection of structures with grouted connections, see Table 7-1 and relevant subsections of [7.4] .
— Subdivided the requirements for onsite QC testing for grouting operations into the different relevant applications, see relevant subsections of [7.6.4].
— Included updates and new clauses relating to construction control of formwork including slipforming, see [7.7.1.18] and relevant subsections of [7.7.2].
— Requirements for construction joints have been updated, see [7.10.2.3].
— Updated and revised some subsections related to grouting operations, see relevant subsections of [7.11] and [7.17].

• Sec.9 Certification and classification

— Sec.9 has been removed from the standard. It's content will be issued separately as a DNV GL service specification, see DNVGL-SE-0295.

• App.A Environmental loading

— Requirements for geotechnical design harmonised with DNVGL-RP-C212 in [A.1.3.9].

• App.B Structural analysis - modelling

— Requirements for geotechnical design harmonised with DNVGL-RP-C212 in [B.1.1].

• App.H Requirements to content in material certificate for structural grout

— Expanded Table H-1 and Table H-2 with items on target densities during mixing, maximum hose length with minimum nominal bore and grout free fall distance in water.
— Revised recommendations for test methods in Table H-3 and Table H-4 for fresh and hardened grout. Alternative test methods for some tests and a new test on restrained shrinkage have been added.

• App.I QA/QC system for manufacture of structural grout, or equivalent material (guidelines)

— [I.1.1.9] and [I.1.1.14]: Added paragraphs on minimum required documentation of properties from manufacturer and traceability of blended grouts.

Editorial corrections

In addition to the above stated changes, editorial corrections may have been made.

CONTENTS

Contents

Contents

Contents

SECTION 1 GENERAL

1.1 Introduction

1.1.1

This detailed standard for design of offshore concrete structures is prepared based on more than 40 years of industry experience. It draws on knowledge gained through joint industry projects, experience feedback and projects run in DNV GL. This standard provides principles, technical requirements and guidelines for the design, fabrication/construction, installation and in-service inspection of offshore concrete structures.

1.1.2

The first offshore rules covering both steel and concrete structures was issued in 1974 and updated in 1977 and 1992. The DNV rules and Norwegian Standard NS3473 developed over time in similar trends. DNV-OS-C502 was based on both when introduced in 2004. The last revision NS3473, Rev 6 was issued in 2004 and was withdrawn in March 2010. DNV-OS-C502 was updated in 2007 and 2012. In 2017, DNV-OS-C502 was rebranded as DNVGL-ST-C502. Changes in the 2017 edition mainly consisted of updated company name and references to other documents within the DNV GL portfolio, i.e. no technical updates were made.

1.1.3

This standard shall be used together with the general offshore design standards for steel structures DNVGL-OS-C101, DNVGL-OS-C102, DNVGL-OS-C103, DNVGL-OS-C105 and DNVGL-OS-C106.

1.1.4

For design and construction of offshore concrete support structures for wind turbines, see DNVGL-ST-0126 *Support structures for wind turbines*

1.1.5

For design and construction of LNG terminal structures and containment systems, see DNVGL-ST-C503 *Concrete LNG terminal structures and containment systems*.

1.2 Objective

1.2.1

The objectives of this document are to:

- Provide an international standard for the design, fabrication/construction, installation 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.
- Provide technical requirements for certification of cementitious structural grouts and FRP reinforcement.
- Serve as a contractual reference document between supplier and purchasers related to design, construction and in-service inspection of offshore concrete structures.
- Serve as a guideline for designer, supplier, purchasers and regulators.

1.3 Scope

1.3.1

The standard comprises of eight (8) sections focusing on:

— safety philosophy
— design documentation
— materials
— loads and analyses requirements
— detailed design of offshore concrete structures
— construction
— in-service inspection, - maintenance and condition monitoring.

1.3.2

In addition, several appendices give more detailed information. Informative appendices are non-mandatory.

— App.A contains guidelines for evaluation of environmental loading.
— App.B and App.C contain guidelines for modelling and structural analysis.
— App.D contains guidelines for the use of alternative design standards.
— App.E contains guidelines for calculation of crack widths.
— App.F specifies requirements for fibre reinforced polymer (FRP) bars subject to DNV GL certification services..
— App.G contains minimum requirements for the QA/QC system for manufacture of FRP bars.
— App.Hspecifies requirements for cementitious grouts subject to DNV GL certification services.
— App.I contains minimum requirements for the QA/QC system for manufacture of structural grout and fibre reinforced structural grout.
— App.J contains minimum requirements for mock-up testing for cementitious grouts subject to DNV GL certification services.
— App.K contains minimum requirements for steel reinforcement specified for use with this standard.

1.3.3

This standard specifies:

— Principles, technical requirements and guidelines for the design, fabrication/construction, installation and in-service inspection of offshore concrete structures.
— Minimum technical requirements for certification of cementitious structural grouts and FRP reinforcement.

1.3.4

This standard covers fixed and floating structures where reinforced concrete, prestressed concrete and cementitious grout are used as structural materials.

1.3.5

In addition to the requirements provided in this standard, it is 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.

1.4 Application

1.4.1 General

1.4.1.1 The standard may be used in the design, fabrication/construction, installation and in-service inspection of the following types of support structures, which are referred in this standard as offshore concrete structures:

— gravity based structures (GBS) for oil/gas production offshore
— GBS for oil/gas production with oil/gas storage facility
— floating concrete structures for production/storage of oil/gas. The structure may be of any type: floating structure, e.g. tension leg platform (TLP), column stabilized units
— barge units
— deepwater caisson type concrete foundation of bridges
— floating foundations for bridges, parking houses or storage buildings
— other types of offshore/nearshore concrete structures.

1.4.1.2 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.

1.5 References

1.5.1 General

1.5.1.1 In this standard, when dated references of DNV GL standards are presented, only the edition cited applies. For undated references, the latest edition of the referenced document (including amendments) applies.

1.5.2 Standards other than DNV GL standards

1.5.2.1 In case of conflict between the requirements of DNV GL standards and a reference document other than DNV GL standards, the requirement of DNV GL standards shall prevail.

1.5.2.2 The provision for using standards other than DNV GL is that the same safety level as provided by this DNV GL standard is obtained.

1.5.2.3 Where reference is made to standards other than DNV GL, the valid revision shall be taken as the revision which is current at the date of issue of this standard, unless otherwise noted.

1.5.3 Normative references

1.5.3.1 The standards in Table 1-1 include provisions, which through reference in this text constitute provisions of this standard.

Table 1-1 DNV GL rules, offshore standards and standards

Reference	*Title*
DNVGL-ST-N001	Marine operations and marine warranty
DNVGL-OS-A101	Safety principles and arrangements
DNVGL-OS-C101	Design of offshore steel structures, general - LRFD method
DNVGL-OS-C102	Structural design of offshore ships
DNVGL-OS-C103	Structural design of column stabilised units - LRFD method
DNVGL-OS-C105	Structural design of TLPs - LRFD method
DNVGL-OS-C106	Structural design of deep draught floating units - LRFD method
DNVGL-OS-C301	Stability and watertight integrity
DNVGL-SE-0284	Type approval scheme, oil and gas
DNVGL-ST-C503	Concrete LNG terminal structures and containment Systems
DNVGL-ST-0126	Support structures for wind turbines

1.5.4 Informative references

1.5.4.1 The latest valid revision of the documents in Table 1-2, Table 1-3 and Table 1-4 apply. These include acceptable methods for fulfilling the requirements in this standard.

1.5.4.2 Other recognised standards may be applied provided it is documented that they meet or exceed the level of safety of this DNV GL standard, see App.D.

Table 1-2 DNV GL rules and offshore object standards for structural design

Reference	*Title*
DNVGL-RU-SHIP	DNV GL rules for classification: Ships (RU-SHIP) Part 5 Ship types Ch.7 Liquefied gas tankers
DNVGL-OS-B101	Metallic materials
DNVGL-OS-C401	Fabrication and testing of offshore structures
DNVGL-OS-E301	Position mooring
DNVGL-ST-0145	Offshore substations

Table 1-3 DNV GL recommended practices (RP), class guidelines (CG) and service specifications (SE)

Reference	*Title*
DNVGL-RP-A203	Technology qualification
DNVGL-RP-C201	Buckling strength of plated structures
DNVGL-RP-C202	Buckling strength of shells

Reference	*Title*
DNVGL-RP-C203	Fatigue design of offshore steel structures
DNVGL-RP-C205	Environmental conditions and environmental loads
DNVGL-RP-C207	Statistical representation of soil data
DNVGL-RP-E301	Design and installation of fluke anchors
DNVGL-RP-E302	Design and installation of plate anchors in clay
DNVGL-CG-0128	Buckling
DNVGL-RP-C212	Offshore soil mechanics and geotechnical engineering
DNVGL-RP-C211	Structural reliability analysis (replacing DNV Classification Note 30.6)
DNVGL-CG-0129	Fatigue assessment of ship structures
DNVGL-SE-0295	Offshore concrete structures and grout services
DNVGL-SE-0477	Risk based verification of offshore structures

Table 1-4 Other references

Reference	*Title*
ACI 440.1R-15	Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars
ACI 440.3R-12	Guide Test Methods for Fiber-Reinforced Polymer (FRP) Composites for Reinforcing or Strengthening Concrete and Masonry Structures
ACI 440-4R–04	Prestressing Concrete Structures with FRP Tendons
ACI 440R-07	Report on fibre-reinforced polymer (FRP) reinforcement for concrete structures.
ASTM C114	Standard Test Method for Chemical Analysis of Hydraulic Cement
ASTM C150	Standard Specification for Portland Cement
ASTM C157	Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
ASTM C187	Standard Test Method for Amount of Water Required for Normal Consistency of Hydraulic Cement Paste
ASTM C191	Standard Test method for Time of Setting of Hydraulic Cement by Vicat Needle
ASTM C204	Standard Test Method for Fineness of Hydraulic Cement by Air-Permeability Apparatus
ASTM C230	Standard Specification for Flow Table for Use in Tests of Hydraulic Cement
ASTM C348	Standard Test Method for Flexural Strength of Hydraulic Cement Mortars
ASTM C403	Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
ASTM C457	Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete
ASTM C469	Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
ASTM C490	Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete

Reference	*Title*
ASTM C496	Standard Test Method for Splitting Tensile strength of Cylindrical Concrete Specimens
ASTM C512	Standard Test Method for Creep of Concrete in Compression
ASTM C666	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
ASTM C940	Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory
ASTM C1437	Standard Test Method for Flow of Hydraulic Cement Mortar
ASTM C1581	Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage
ASTM C1698	Standard Test Method for Autogenous Strain of Cement Paste and Mortar
CSA S806-12	Design and construction of building structures with fibre-reinforced polymers
CSA A23.1-14	Concrete materials and method of concrete construction
CSA A23.2-14	Test methods and standard practices for concrete
EN 196-1	Methods of testing cement – Part 1: Determination of strength
EN 196-2	Methods of testing cement – Part 2: Chemical analysis of cement
EN 196-3	Methods of testing cement – Part 3: Determination of setting times and soundness
EN 196-6	Methods of testing cement – Part 6: Determination of fineness
EN 206	Concrete - Specification, performance, production and conformity
EN 445	Grout for prestressing tendons - Test methods
EN 447	Grout for prestressing tendons - Basic requirements
EN 1015-6	Methods of test for mortar for masonry - Part 6: Determination of bulk density of fresh mortar
EN 1015-7	Methods of test for mortar for masonry - Part 7: Determination of air content of fresh mortar
EN 12350-6	Testing fresh concrete - Part 6: Density
EN 12350-7	Testing fresh concrete - Part 7: Air content - pressure methods
EN 12350-8	Testing fresh concrete - Part 8: Self-compacting concrete - Slump flow test
EN 12390-1	Testing hardened concrete - Part 1: Shape, dimensions and other requirements for specimens and moulds
EN 12390-2	Testing hardened concrete - Part 2: Making and curing specimens for strength tests
EN 12390-3	Testing hardened concrete - Part 3: Compressive strength of test specimens
EN 12390-6	Testing hardened concrete - Part 6: Tensile splitting strength of test specimens
EN 12390-7	Testing hardened concrete – Part 7: Density of hardened concrete
EN 12504-1	Testing concrete in structures - Part 1: Cored specimens - Taking, examining and testing in compression
EN 13670	Execution of concrete structures
EN 13791	Assessment of in-situ compressive strength in structures and precast concrete components

Reference	*Title*
ISO 1920-4	Testing of concrete - Part 4: Strength of hardened concrete
ISO 10406-1	Fibre-reinforced polymer (FRP) reinforcement of concrete – Test methods – Part 1 FRP bars and grids
ISO 19900	Petroleum and natural gas industries – General requirements for offshore structures
ISO 19901-1	Petroleum and natural gas industries – Specific requirements for offshore structures – Part 1: Metocean design and operating considerations
ISO 19901-2	Petroleum and natural gas industries – Specific requirements for offshore structures – Part 2: Seismic design procedures and criteria
ISO 19901-4	Petroleum and natural gas industries – Specific requirements for offshore structures – Part 4: Geotechnical and foundation design considerations
ISO 19901-8	Petroleum and natural gas industries — Specific requirements for offshore structures — Part 8: Marine soil investigations
ISO 19903	Petroleum and natural gas industries – Fixed concrete offshore structures
MC2010	fib Model Code for Concrete Structures 2010
NORSOK N-003	Actions and Actions Effects
NORSOK N-004	Design of Steel Structures
SINTEF STF22 A98741	Eurocrete. Modifications to NS3473 when using fibre reinforced plastic (FRP) reinforcement

1.6 Definitions

1.6.1 Terms

1.6.1.1 Terms and definitions as shown in Table 1-5 are used in this standard.

Table 1-5 Definitions of terms

Terms	*Definitions*
abnormal level earthquake (ALE)	intense earthquake of abnormal severity under the action of which the structure should not suffer complete loss of integrity
accidental limit states (ALS)	limit state related to the possibility of the structure to resist accidental loads and maintain integrity and performance of the structure due to local damage or flooding
accidental loads (A)	rare occurrences of abnormal environmental loads, fire, flooding, explosions, dropped objects, collisions, unintended pressure differences, thermal shock due to LNG spilling or overflow, etc.
aggregates	constituent material of concrete or grout added to increase volume, weight or durability of the material Aggregates are the main constituent, both with respect to volume and weight, in a structural concrete mix. They may generally be divided into two groups, these being: sand or fine aggregate (materials less than 5 mm) and coarse aggregate (materials larger than 5 mm).

Terms	*Definitions*
air gap	free distance between the design wave and the underside of a topside structure supported on columns allowing the wave to pass under the topside structure
as-built documentation	documentation of the offshore structure as finally constructed [3.1.5] presents the list of documents that are part of the as-built documentation.
atmospheric zone	the external surfaces of the unit above the splash zone
cathodic protection	technique to prevent corrosion of a steel surface by making the surface to be the cathode of an electrochemical cell
cement	binder component in a structural concrete or grout mix
cement grout	general term referring to grout batched at the work site consisting of mainly cement and water May refer to neat cement grout or a cement and water mix with a limited dosage of admixtures added during batching.
certification	third-party issue of a statement, based on a decision following review, that fulfilment of specified requirements has been demonstrated related to products, processes or systems Review shall in this context mean verification of the suitability, adequacy and effectiveness of selection and determination activities, and the results of these activities, with regard to fulfillment of specified requirements by an object of conformity assessment.(ISO 17000:2004).
certification service	the process of performing certification in accordance with a service specification and with a DNV GL certificate as deliverable.
characteristic load	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.
characteristic material strength	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.
characteristic value	representative value associated with a prescribed probability of not being unfavourably exceeded during some reference period
classification	a service which comprises the development and maintenance of rules, and the verification of compliance with the rules throughout the Vessels' life. The extent of and methods for verifying compliance will be decided by the Society to establish reasonable assurance that the relevant rules are complied with.
coating	metallic, inorganic or organic material applied to steel surfaces for prevention of corrosion
concrete grade	parameter used to define the concrete strength
corrosion allowance	extra wall thickness added during design to compensate for any anticipated reduction in thickness during the operation
cryogenic temperature	being or related to very low temperature down to -200°C
deck mating	operations through which the deck floated on barges is mated with the concrete support structure
deformation loads (D)	loads effects on the structure caused by thermal effects, prestressing effects, creep/shrinkage effects, differential settlements/deformations, etc.

Terms	*Definitions*
design basis	document where owners' requirements in excess of this standard should be given
design hazards	hazards likely to occur are identified as part of the risk assessment Design hazards are mitigated into the structural design of the structure.
design life	duration to which the parameters used in structural design are related
design temperature	lowest daily mean temperature in air for areas where the unit will be transported, installed and operated for the given period of operation Temperature experienced by the element due to local effects during its design life, may include influences from cargo temperature, sea temperature, operational requirements etc.
design value	value used in the deterministic design procedure, i.e. characteristic value modified by the resistance factor or load factor
driving voltage	the difference between closed circuit anode potential and the protection potential
ductility	property of a steel or concrete member to sustain large deformations without failure
environmental loads (E)	loads from wind, wave, tide, current, snow, ice and earthquake
expected loads and response history	history for a specified time period, taking into account the number of load cycles, the resulting load levels and response for each cycle
expected value	most probable value of a load during a specified time period
extreme level earthquake (ELE)	earthquake with a severity which the structure should sustain without major damage When exposed to an ELE, a structure is supposed to retain its full capacity for all subsequent conditions.
fatigue	degradation of the material caused by cyclic loading
fatigue critical	structure with calculated fatigue life near the design fatigue life
fatigue limit states (FLS)	limit state related to the possibility of failure due to the effect of cyclic loading
fibre mass fraction	ratio of fibre mass to total mass of FRP material within a reinforcement bar
fibre	short fibres made from steel or FRP used in structural concrete or grout
fluid	a liquid that in most cases considered will be seawater, water or oil A concrete structure might be exposed to the pressure and chemical properties of such fluids on the outer or inner face.
FRP material	fibre reinforced polymer (FRP) composite made from carbon, glass, aramid or basalt
fibre reinforced concrete	structural concrete mixed with short fibre material
fibre reinforced grout	grout mixed with short fibre material
fibre volume fraction	ratio of fibre volume to total volume of FRP material within a reinforcement bar
functional loads	permanent (G) and variable loads (Q), except environmental loads (E), to which the structure is exposed
grout	see cement grout, pre-blended grout, fibre reinforced grout and structural grout
hazards identification	List of critical situations that will have the potential to cause, or contribute substantially to a major accident if they happen to fail The list is based on consequence of failure only, not on likelihood of failure of the individual hazards.

Terms	*Definitions*
headed reinforcement (T-heads)	headed reinforcement bars are ordinary reinforcement bars with circular, square or rectangular shaped steel plates attached at one or both ends, generally by means of friction welding
high strength concrete	concrete of grade in excess of C65
hindcasting	method using registered meteorological data to reproduce environmental parameters, which is mostly used for reproducing wave parameters
inspection	activities such as measuring, examination, testing, gauging one or more characteristics of an object or service and comparing the results with specified requirements to determine conformity
live loads of permanent character	live loads that the structure may be exposed to for its entire service life or a considerable part of it, e.g. weight of furniture, stored goods etc
live loads of variable character	live loads that the structure may be exposed to only for limited durations much less than the service life, such as e.g. weight of occupants and (not permanently stored) vehicles
light weight aggregate (LWA)	aggregates made from expanded clay, expanded shale, slate or sintered pulverized ash from coal-fired power plants, or from other materials with corresponding documented properties in accordance with the requirements of recognized standards e.g. ASTM, ACI, EN.
light weight aggregate concrete (LWAC)	concrete made with lightweight aggregates conforming to requirements contained in recognized standards, e.g. relevant ASTM, ACI or EN standard The provisions of this standard are valid only for LWAC having a lower limit of oven-dry density of 1200 kg/m^3 and an upper limit of 2200 kg/m^3.
limit state	state beyond which the structure no longer satisfies the performance requirements. The following categories of limit states are of relevance for structures: ULS = ultimate limit states FLS = fatigue limit states ALS = accidental limit states SLS = serviceability limit states
limit state design	design of the offshore concrete structure in the limit states of ULS, SLS, FLS and ALS
load and resistance factor design (LRFD)	method for design where uncertainties in loads are represented with a load factor and uncertainties in resistance are represented with a material factor
load effect	effect of a single design load or combination of loads on the equipment or system, such as stress, strain, deformation, displacement, motion, etc.
lowest daily mean temperature	lowest value on the annual mean daily average temperature curve for the area in question,. for temporary phases or restricted operations the lowest daily mean temperature may be defined for specific seasons Mean daily average temperature = the statistical mean average temperature for a specific calendar day. — Mean: statistical mean based on number of years of observations. — Average: average during one day and night.
lowest waterline	typical light ballast waterline for ships, transit waterline or inspection waterline for other types of units
mill certificate	document made by the manufacturer of cement 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 cement itself

Terms	*Definitions*
neat cement grout	grout made from a mixture of cement and water only
non-cementitious materials	materials such as epoxy and polyurethane which are specially made for use together with structural concrete to improve the concrete properties or to supplement, repair or replace the concrete.
non-destructive testing (NDT)	testing techniques used to evaluate the properties of materials, components or systems without causing damage such as rebound hammer, resistivity, radiographic, ultrasonic, impact eco for concrete testing and radiography, ultrasonic or magnetic powder methods for inspection of welds
normal strength concrete	concrete of grade C35 to C65
normal weight concrete	ordinary concrete with an oven-dry density of at least 2200 kg/m^3 and an upper limit of 2600 kg/m^3
offshore concrete structure	fixed and floating structures where reinforced concrete, prestressed concrete and cementitious grout are used as structural materials.
offshore standard	a DNV GL standard that contain technical requirements, principles and acceptance criteria related to certification and classification of offshore units
offshore installation	general term for mobile and fixed structures and facilities which are intended for exploration, drilling, production, processing or storage of hydrocarbons including installations intended for accommodation of personnel engaged in these activities The term covers subsea installations and pipelines but not traditional shuttle tankers, supply boats and other support vessels which are not directly engaged in the activities described above.
one-compartment damage stability	characteristic of a floating object which remains stable even if one of its compartments is flooded
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, survival and accidental)
partial 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
permanent functional loads (G)	self-weight, ballast weight, weight of permanent installed parts of mechanical outfitting, external hydrostatic pressure, prestressing force, etc.
post-tensioned reinforcement	reinforcement (normally tendons, wires, strands or bars) placed inside ducts and tensioned after the concrete has hardened
potential	voltage between a submerged metal surface and a reference electrode
pre-blended grout	grout proportioned at a factory following strict QA/QC procedures and delivered to site for mixing with a predefined proportion of water May refer to pre-packed and silo stored/transported products.
prestressing systems	tendons (wires, strands, and bars), anchorage devices, couplers and ducts or sheaths are part of a prestressing system May refer to pre-tensioned or post-tensioned systems.
pre-tensioned reinforcement	reinforcement (normally wires or strands) tensioned before concrete has been placed

DNV GL AS

Terms	*Definitions*
product certificate	certificate to document conformity with the requirements of the applicable standard. It lists material properties documented through testing Test samples shall be taken from the delivered products themselves and testing, or a part there-of shall be performed in the presence of a third party or in accordance with special agreements.
product data sheet	sheet issued by the manufacturer with data about the product which may contain design data for the product and may be appended to product or type approval certificate
quality plan	plan implemented to ensure quality in the design, construction and in-service inspection/ maintenance
recommended practice (RP)	DNV GL publication that contains sound engineering practice and guidance
reinforcement	constituents of structural concrete providing the tensile strength that will give the reinforced concrete its ductile characteristics, in this standard, reinforcement is categorised as: — ordinary reinforcement — prestressing reinforcement — fibre reinforced polymer reinforcement (limited to carbon, glass, aramid and basalt) — special reinforcement.
robustness	a robust structure is a structure with low sensitivity to local changes in geometry and loads
redundancy	the ability of a component or system to maintain or restore its function when a failure of a member or connection has occurred, for instance by introducing alternative load paths or force redistribution
reference electrode	electrode with stable open-circuit potential used as reference for potential measurements
reliability	the ability of a component or a system to perform its required function without failure during a specified time interval
repair materials	materials used to structurally repair the offshore concrete structure
risk	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.
risk based inspection	decision making technique for inspection planning based on risk – comprising the probability of failure and consequence of failure
service temperature	reference temperature on various structural parts of the unit used as a criterion for the selection of steel grades or acceptable crack width, etc. in SLS
service life	expected lifetime, or the expected period of use in service of the facility or structure
serviceability limit states (SLS)	limit state corresponding to the criteria applicable to normal use or durability
sheaths	ducts for post-tensioning tendons, typically taken to be of semi rigid or rigid type, water tight and with adequate stiffness to prevent damages and deformations
short term tensile strength	the strength of a FRP bar characterized in a standard test in terms of the rupture strength due to tension that increases at a constant rate till rupture, typically over 1 – 5 minutes
slamming	impact load on a member from a rising water surface as a wave passes May also occur within tanks due to stored liquids.

Terms	*Definitions*
sloshing	effects caused by the movement of liquid inside a container, which is typically also undergoing motion
S-N curve	is a plot of the magnitude of an alternating stress versus the number of cycles to failure for a given material
specified minimum yield strength (SMYS)	the minimum yield strength prescribed by the specification or standard under which the material is purchased
specified value	minimum or maximum value during the period considered which may take into account operational requirements, limitations and measures taken such that the required safety level is obtained
splash zone	external surfaces of the unit that are periodically exposed to water, the determination of which includes evaluation of waves, tidal variations, settlements, subsidence and vertical motions
stability	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.
standards	technical requirements and acceptance criteria. In the context of this document, the term standard shall be understood to cover document types such as codes, guidelines and recommended practices in addition to bona fide standards.
structural concrete	cementitious composite material which is the main ingredient for construction of concrete structures
structural grout	a cementitious material which is part of the load carrying system of the structure with a characteristic compressive strength higher than 35 MPa containing cement, water and often additions, admixtures and appropriate fine aggregates Structural grout may be cement grout, pre-blended grout and fibre reinforced grout.
submerged zone	part of the unit which is below the splash zone, including buried parts
survival condition	condition during which a unit may be subjected to the most severe environmental loadings for which the unit is designed when operations such as drilling 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.
target safety level	nominal acceptable probability of structural failure
temporary phase conditions	design conditions not covered by operating conditions, e.g. conditions during fabrication, mating and installation phases, transit and towing phases, accidental conditions
test report	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
tensile strength	for steel it is the minimum stress level where strain hardening is at maximum or at rupture For concrete it is the direct tensile strength.
tex	tow size in grams per km length of tow or fibre
time to rupture	time it takes from when a specified load is applied until this load causes rupture of the FRP bar, may refer to both fatigue and stress rupture normally, the time to rupture under a constant sustained load is measured

Terms	*Definitions*
tow	untwisted bundle of fibres in the form they are delivered on bobbins by the fibre supplier (synonym: roving, untwisted yarn)
transit conditions	unit movements from one geographical location to another
type approval certificate (TAC)	DNV GL certificate documenting approval of conformity with specified requirements on the basis of a systematic examination of one or more specimens of a product representative for the production
ultimate limit states (ULS)	limit state corresponding to the maximum load carrying resistance
unit	general term for an offshore structure
utilization factor	fraction of anode material that may be utilised for design purposes
variable functional loads (Q)	weight and loads caused by the normal operation of the offshore structure which 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.
VL certificate	DNV GL certificate (*VL*) A product or material certificate confirming conformity with the rules, validated and signed by a DNV GL surveyor will be denoted a *VL* certificate.
verification	confirmation, through the provision of objective evidence (analysis, observation, measurement, test, records or other evidence), that specified requirements have been fulfilled (ISO 9000:2015)
yarn	twisted bundle of fibres, twisted tow
works certificate	document signed by the manufacturer stating conformity with DNV GL rule requirements, that tests are carried on samples taken from the delivered product itself and that tests are witnessed and signed by a qualified department of the manufacturer

1.6.2 Abbreviations

1.6.2.1 Abbreviations as shown in Table 1-6 are used in this standard.

Table 1-6 Abbreviations

Abbreviation	*Description*
A	accidental loads
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ALE	abnormal level earthquake
ALS	accidental limit states
API	American Petroleum Institute
ASR	alkali silica reaction

Abbreviation	*Description*
ASTM	American Society for Testing and Materials
BS	British Standard issued by British Standard Institution
CoG	centre of gravity
D	deformation loads
DDF	deep draught floaters
E	environmental loads
ELE	extreme level earthquake
EN	european norm
ETM	event tree method
ESD	emergency shut down
FLS	fatigue limit state
FM	fracture mechanics
FMEA	failure mode effect analysis
FRP	fibre reinforced polymer
FTM	fault tree method
G	permanent loads
HAT	highest astronomical tide
HAZOP	hazard and operability study
HISC	hydrogen induced stress cracking
HPC	high performance concrete
HS	high strength
IGC	international gas carrier
IMO	international maritime organisation
ISO	international organisation of standardisation
LAT	lowest astronomical tide
LNG	liquefied natural gas
LRFD	load and resistance factor design
LWA	lightweight aggregate
LWAC	lightweight aggregate concrete
MPI	magnetic particle inspection
MSA	manufacturing survey arrangement
MSF	module support frame
MSL	mean sea level

Abbreviation	*Description*
NACE	National Association of Corrosion Engineers
NDT	non-destructive testing
NS	norwegian standard
NW	normal weight concrete
OPC	ordinary portland cement
QRA	quantitative risk analysis
RP	recommended practice
SLS	serviceability limit state
SMYS	specified minimum yield stress
TAC	type approval certificate
TTR	time to rupture
ULS	ultimate limit state
UR	utilization ratio
W/C	Water to cement ratio in a mix, may include pozzolanic or latent hydraulic additions

1.6.3 Symbols - Greek characters

1.6.3.1 Symbols, Greek characters, as shown in Table 1-7 are used in this standard.

Table 1-7 Greek characters

Symbol	*Description*
α	angle between transverse shear reinforcement and the longitudinal axis also 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.
α_c	factor applicable to E_{cn} and f_{cn} for offshore concrete structures ([6.3.1.11])
α_F	thermal expansion coefficient of FRP reinforcement
α_t	factor applicable to f_{tn} for offshore concrete structures. ([6.3.1.11])
β	opening angle of the bend ([6.12.1.12])
δ	deflection
$\Delta\sigma$	stress variation of the reinforcement (MPa) ([6.13.2.2])
ε	strain
ε_1	average principal tensile strain ([6.8.1.7])

Symbol	*Description*
ε_{cu}	max strain, NW concrete (2.5 m – 1.5)ε_{cn} ([6.3.1.14])
ε_{cm}	mean stress dependent tensile strain in the concrete at the same layer and over the same length as ε_{sm} ([6.15.8.2])
ε_{cs}	free shrinkage strain of the concrete (negative value) ([6.15.8.2])
ε_{s1}	tensile strain in reinforcement slightly sensitive to corrosion on the side with highest strain ([6.15.3.7])
ε_{s2}	tensile strain at the level of the reinforcement sensitive to corrosion ([6.15.3.7])
ε_{sm}	mean principal tensile strain in the reinforcement in the crack's influence length at the outer layer of the reinforcement ([6.15.8.2])
γ_c	material factor for concrete
γ_f	partial load factor
γ_m	material factor (material coefficient)
γ_s	material factor for steel reinforcement
γ_F	material factor to account for statistical variation in the material strength, potential placement inaccuracy in the section due to the physical characteristics of the bars and the level of control implemented during manufacturing of FRP bars
γ_{FI}	material factor to be used for ULS check with load combination type I for FRP bars
γ_{FII}	material factor to be used for ULS check with load combination type II for FRP bars
γ_{FIII}	material factor to be used for ULS check with load combination type III for FRP bars
$\gamma_{F,ssa}$	material factor to be used for long term safe service life assessment for FRP bars
γ_{FA}	material factor to be used in accidental limit states for FRP bars
γ_{FE}	material factor applied to Young's modulus to account for long term creep of the FRP bars It is used to determine strains and deformations for ULS, SLS, FLS and ALS.
γ_{FS}	material factor to be used in serviceability limit states for FRP bars
ℓ	geometric slenderness ratio
ℓ_N	force dependent slenderness
θ	angle between the inclined concrete compression struts and the longitudinal axis in the truss model method
ϕ	diameter of the reinforcement bar
ϕ_e	equivalent diameter in term of reinforcement cross-section
μ	friction coefficient
ρ	coefficient of Findley's creep rate equation
ρ	density

Symbol	*Description*
ρ_F	density of FRP bars
ρ_f	fibre density
ρ_m	matrix density
ρ_x	reinforcement ratio in x − direction = $A_{sx}/(b \cdot d)$
ρ_y	reinforcement ratio in y − direction = $A_{sy}/(b \cdot d)$
η	limit for cumulative damage ratio
η_b	conversion factor for bends for the bend radiuses covered
$\eta_{F,\ TTR}$	conversion factor derived from the characteristic time to rupture curve for the load durations under consideration
η_T	conversion factor for tensile strength of FRP reinforcement from room temperature to specified service temperature
ηt_{emp}	temperature constant to allow for inaccuracies in maintaining and recording low temperatures during grout/ concrete testing as well as inaccuracies associated with temperature forecasting offshore (App.H)
φ	creep coefficient
σ_F	stress in a FRP bar in response to specified loading (referred to nominal bar area)
σ_f	stress in the fibres in a FRP bar in response to specified loading (referred to net fibre area)
σ_c	concrete stress due to long-term loading
σ_d	design stress
σ_M	edge stress due to bending alone (tension positive) ([6.15.8.1])
σ_{max}	numerically largest compressive stress, calculated as the average value within each stress-block
σ_{min}	numerically least compressive stress, calculated as the average value within each stress-block
σ_N	stress due to axial force (tension positive) ([6.15.8.1])
σ_p	steel stress due to prestressing
σ_{trough}	stress at the trough of the stress cycle (minimum stress)
σ_{peak}	peak stress of the stress cycle (maximum stress)
τ_{cd}	bond strength
τ_{bmax}	maximum bond stress within fatigue stress block
τ_{bmin}	minimum bond stress within fatigue stress block
v_f	volume fraction of fibre in FRP bar

1.6.4 Symbols - Latin characters

1.6.4.1 Symbols, Latin characters, as shown in Table 1-8 are used in this standard.

Table 1-8 Latin characters

Symbol	*Description*
A	distance from the face of the support
A_1	loaded area
A_2	assumed distribution area
A_c	concrete area of a longitudinal section of the beam web
A_c	cross-sectional area of uncracked concrete
A_{cf}	effective-cross section area of the flange, $h_f\, b_{eff}$
A_F	cross-sectional area of FRP reinforcement
A_f	Net fibre area in a FRP reinforcement bar
$A_{F,\ BAR}$	cross-sectional area of each FRP reinforcement bar
$A_{F,\ min}$	minimum area of FRP reinforcement needed to prevent excessive cracking
$a_{f,tow}$	net fibre area of tow
$A_{F,V}$	amount of FRP shear reinforcement with spacing s [mm^2]
$A_{F,v\ min}$	minimum amount of FRP shear reinforcement with spacing s [mm^2]
A_{Fs}	nominal FRP bar surface area
A_s	cross-sectional area of steel reinforcement or Reinforcement area that is sufficiently anchored on both sides of the joint and that is not utilized for other purposes.
A_{st}	area of transverse reinforcement not utilized for other tensile forces and having spacing not greater than 12 times the diameter of the anchored reinforcement. If the reinforcement is partly utilized, the area shall be proportionally reduced
A_{sv}	amount of shear reinforcement
A_{sx}	amount of reinforcement in x-direction
A_{sy}	amount of reinforcement in y-direction
a_v	vertical acceleration
b_{eff}	part of the slab width which according to [6.1.4] is assumed as effective when resisting tensile forces
b_w	width of beam (web) [mm]
b_x	length of the side of the critical section ([6.6.5.10])
b_y	length of the side perpendicular to b_x
C	coefficient of characteristic safe service life formula for FRP bar specification
C	concrete grade (normal weight concrete)

Symbol	*Description*
C_l	factor on Wöhler curves concrete ([6.13.2])
c_1	minimum concrete cover, see Table 6-15
C_2	factor on Wöhler curves concrete ([6.13.2])
c_2	actual nominal concrete cover
C_3	factor on Wöhler curve reinforcement ([6.13.2])
C_4	factor on Wöhler curve reinforcement ([6.13.2])
C_5	fatigue strength parameter ([6.13.2])
D	Deformation load
D	distance from the centroid of the tensile reinforcement to outer edge of the compression zone
d_1	1000 mm
D_F	nominal diameter of FRP bar
D_k	diameter of the concrete core inside the centroid of the spiral reinforcement, A_{ss}
E	environmental load
e	eccentricity of loading
E_{cd}	design value of Young's Modulus of concrete used in the stress-strain curve
E_{cn}	normalized value of Young's Modulus of concrete used in the stress-strain curve
E_F	characteristic value of the Young's modulus of FRP reinforcement bar (referred to nominal bar area A_F)
E_{Fd}	design value of Young's Modulus of FRP bars
E_{sd}	design value of Young's Modulus of steel reinforcement
E_{sk}	characteristic value of Young's Modulus of steel reinforcement (200 000 MPa)
f_{bc}	concrete related portion of the design bond strength in accordance with [6.11.1.16]
f_{bd}	design bond strength, calculated in accordance with [6.11.1.16]
f_{c2d}	truss analogy: design compressive strength ([6.6.3.8]) in the compression field General: reduced design compressive strength ([6.8.1.7])
f_{cck}	characteristic compressive cylinder strength of concrete or grout
f_{cck2}	94 MPa ([4.3.3.7])
f_{cckj}	characteristic strength of the drilled cores converted into cylinder strength for cylinders with height/diameter ratio 2:1
f_{cckt}	characteristic compressive cylinder strength at 28 days based on in-situ tests
F_{cd}	compressive capacity
f_{cd}	design compressive strength of concrete/grout
f_{ck}	characteristic concrete/grout cube strength
f_{cn}	normalized compressive strength of concrete/grout
F_d	design load

Symbol	*Description*
F_F	tensile force at rupture of FRP bar
f_F	characteristic short term tensile strength (force per area) of FRP bar
$f_{F,\ bend}$	characteristic tensile strength of bent portion of FRP bar
f_{Fb}	design strength of the bend portion of FRP bar
f_{Fd}	Design strength of FRP reinforcement
$f_{F,\ TTR(i)}$	characteristic tensile strength (force per area) in FRP bar until failure at considered load duration, *i*, derived from characteristic TTR curve. *i* is taken as I, II, III corresponding to load durations of 50 years, 1 year, and 1 week respectively
F_k	characteristic load
f_{rd}	reference strength for use in fatigue calculation, dependant on the type of failure in question ([6.13.2])
$f_{rd,\ fat}$	reference strength for use in fatigue calculation, dependant on the type of failure in question ([6.13.2]) including the material specific factor C_5
f_{sd}	design strength of steel reinforcement
f_{sk}	characteristic strength of steel reinforcement
f_{ssd}	design strength of the spiral reinforcement, A_{ss}
F_{SV}	additional tensile force in longitudinal reinforcement due to shear
f_{td}	design strength of concrete/grout in uniaxial tension
f_{tk}	characteristic uniaxial tensile strength of concrete/grout
f_{tk}	f_{tk} + 0.5 p_w for structures exposed to pressure from liquid or gas in the formulae for calculating the required amount of minimum reinforcement ([6.17.6.3])
f_{tn}	normalized tensile strength of concrete/grout
F_{vn}	force corresponding to shear failure at cross wire welds within the development length
G	permanent load
g, g_o	acceleration due to gravity
H	cross-section height
h’	distance between the centroid of the reinforcement on the tensile” and compression side of the member
h_f	thickness of the flange (the slab)
I_c	moment of inertia of A_c
L	length of FRP bar
l'_b	development length for welded wire fabric
l_b	development length bond – bars or bundle of bars
l_{bp}	development length for the prestressing force
l_e	effective length, theoretical buckling length
L_i	distance between zero moment points

Symbol	*Description*
l_{sk}	influence length of the crack considering that some slippage in the bond between reinforcement and concrete may occur ([6.3.8.2])
M	moment
m	$\varepsilon_{co}/\varepsilon_{cn}$
M_f	total moment in the section acting in combination with the shear force V_f
m_f	mass fraction of fibres (average from production records)
m_m	average mass fraction of matrix resin ($m_m = 1 - m_f$)
$\mid M_{OA} \mid$	numerical smallest member end moment calculated from 1. order theory at end A
$\mid M_{OB} \mid$	numerical largest member end moment calculated from 1. order theory at end B
m_{tex}	tow or fibre mass expressed in tex [g/km]
N	exponent of Findley's creep rate equation
N	design life of concrete subjected to cyclic stresses
n_f	$N_f/f_{cd}A_c$
N_f	design axial force (positive as tension)
n_i	number of cycles in stress-block i ([6.13.1.8])
N_i	number of cycles with constant amplitude which causes fatigue failure ([6.13.1.8])
N_x	axial force in x-direction
N_{xy}	shear force in the x-y plane
N_y	axial force in y-direction
P	load
P	pressure
p_d	design pressure
Q	variable functional load
R	radius
r_c	radius of curvature
R_d	design resistance
R_k	characteristic resistance
s	centre to centre distance between the spiral reinforcement, measured in the longitudinal direction of the column ([6.4.1.6]) or, spacing between shear reinforcement in longitudinal direction
s_1	spacing of the transverse reinforcement
S_c	area moment about the centroid axis of the cross-section for one part of the concrete section
S_d	design load effect
S_k	characteristic load effect
T	specified longitudinal tolerance for the position of the bar end

Symbol	Description
$t_{app, max}$	maximum temperature of application, defined by the manufacturer, for a grout or fibre reinforced grout Shall be taken as +30°C in the absence of data from an elevated temperature test programme.
$t_{app, min}$	minimum temperature of application, defined by the manufacturer, for a grout or fibre reinforced grout Shall be taken as +5°C in the absence of data from a low temperature test programme.
$t_{test, max}$	temperature which the equipment, constituent materials and test and curing environments shall be maintained at during material testing of grout to be qualified for application at temperatures above 30°C
$t_{test, min}$	temperature which the equipment, constituent materials and test and curing environments shall be maintained at during material testing of grout to be qualified for application at temperatures below +5°C
V_{ccd}	design shear capacity of a concrete cross-section (shear compression mode of failure)
V_{cd}	design shear capacity of a concrete cross-section (shear tension made of failure)
V_f	design shear force for the cross-section under consideration
V_{max}	maximum shear force within fatigue stress block
V_{min}	minimum shear force within fatigue stress block
V_{sd}	design shear capacity of transverse reinforcement (shear tension mode of failure)
W_c	section modulus of the concrete cross-section with respect to the extreme tension fibre or the fibre with least compression
w_k	nominal characteristic crack widths
z	0.9 d for sections with a compression zone
z_1	the greater of 0.7 d and I_c / S_c

1.6.5 Verbal forms

1.6.5.1 Verbal forms, as shown in Table 1-9 are used in this standard.

Table 1-9 Definitions of erbal forms

Term	Definition
shall	verbal form used to indicate requirements strictly to be followed in order to conform to the document
should	verbal form used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required
may	verbal form used to indicate a course of action permissible within the limits of the document

SECTION 2 SAFETY PHILOSOPHY

2.1 General

2.1.1 Objective

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

2.1.1.2 This section applies to offshore concrete structures which shall be built in accordance with this standard.

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

2.1.1.4 The integrity of an offshore concrete structure designed and constructed in accordance with this standard is ensured through a safety philosophy integrating different parts as illustrated in Figure 2-1.

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

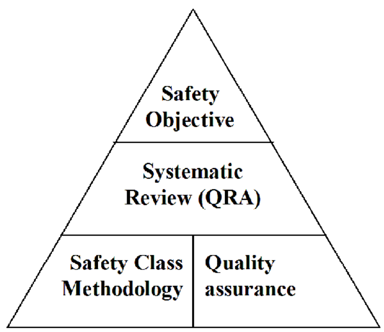


Figure 2-1 Safety philosophy structure

2.1.2 Systematic review

2.1.2.1 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.

2.1.2.2 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 are 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,
— assess 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).

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

2.1.3 Safety class methodology

2.1.3.1 Offshore concrete structures are classified as safety class 3 based on failure consequences. For definition see Table 2-1.

Table 2-1 Safety classes

Consequences of failure	*Safety class*
Minor	1
Serious	2
Very serious	3

2.1.4 Quality assurance

2.1.4.1 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.

2.1.4.2 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.

2.1.4.3 The quality system shall comply with the requirements of ISO 9001 and specific requirements quoted for the various engineering disciplines in this standard.

2.1.4.4 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.

2.1.4.5 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.

2.1.5 Health, safety and environment

2.1.5.1 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.

2.1.6 Qualifications of personnel

2.1.6.1 All activities that are performed in the design, construction, transportation, inspection and maintenance of offshore structures in accordance with this standard shall be performed by skilled 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.

2.1.6.2 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.

2.2 Design Format

2.2.1 General

2.2.1.1 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 DNVGL-OS-C101 Ch.2 Sec.1. The design principle is based on LRFD, but design may additionally be carried out by both testing and probability based design. The aims of the design of the offshore concrete structure and its elements are to:

— Withstand loads likely to occur during all temporary, operating and damaged conditions.
— Maintain acceptable safety for personnel.
— Have adequate durability against deterioration during the design life of the offshore concrete structure.
— Provide sufficient safety against pollution.

2.2.1.2 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.

2.2.1.3 Structures and elements thereof shall possess ductile resistance. Ductile behaviour of concrete structures is required in order to ensure that the structure, to some extent, withstands abnormal or accidental loads and that a redistribution of the loads takes place. The requirements provided in this standard do not ensure sufficient ductility that may be required for seismic loading. In this case, ductility shall be documented.

2.2.1.4 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 donditioned monitoring in Sec.8.

2.2.1.5 Additionally, guidelines are given in App.A to App.K.

2.2.1.6 The design life of the offshore concrete structure shall be decided by the owner of the facility. A minimum of 50 years design life shall be used.

2.2.1.7 In the case of structures reinforced with FRP reinforcement, a minimum of 50 years design life shall be used.

2.2.1.8 The design life to be used for FRP reinforced structures shall ensure that, regardless of foreseeable life extensions, the FRP bars shall not be the limiting factor to the extension of service lifetime of the structure. It is not acceptable to base future life extensions on inspection and maintenance of the FRP bars unless it is based on a documented method to determine the remaining lifetime of the bars.

2.3 Identification of major accidental hazards

2.3.1 General

2.3.1.1 The standard has 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.

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

— loss of life
— significant damage to the asset
— significant damage to the environment.

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

2.3.1.4 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.

2.3.1.5 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.

2.3.1.6 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 are seen to equate to the critical elements of the asset.

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

2.3.1.8 The above processes should consider all phases of the life cycle of the structure.

2.3.1.9 The hazard assessment shall consider, as a minimum, the following events for damage to the primary structure:

- 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.

2.3.1.10 The results of the hazard identification and risk assessment shall become an integral part of the structural design of the offshore concrete structure.

2.4 Life extensions

2.4.1 General

2.4.1.1 Life extension assessment shall be based on a combination of risk based inspection, re-evaluation of applied loads and load combinations and prediction of remaining life based on material deterioration, chloride ingress, carbonation and remaining fatigue life.

2.4.1.2 Risk based inspection shall be performed considering:

- Results of earlier inspections related to visual damage to the concrete surface and possible repairs.
- Changes that may have been engineered related to load situations both from external pay load, internal load from water/oil pressures in tanks.
- Changes in the combinations of original load situations.

2.4.1.3 In cases where either the geometry of the structure has changed or the material has degenerated, making the original global analyses invalid with respect to the prediction of internal force distribution, a new finite element model shall be prepared.

2.4.1.4 Compliance with applicable standards shall be checked. In cases where the structure does not meet the design requirements due to new loads, load cases or changes in geometry, non-linear analyses may be carried out to establish the consequence of redistributing forces to remaining structural elements.

2.4.1.5 Based on earlier and future load history, the remaining fatigue life of the structure shall be predicted in accordance with applicable standards.

SECTION 3 DESIGN DOCUMENTATION

3.1 General

3.1.1 Introduction

3.1.1.1 Documentation shall be prepared for all activities including design, construction, transportation and installation. 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 suitable for independent verification.

3.1.1.2 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.

3.1.1.3 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 [3.1.4] and [3.1.5].

3.1.2 Overall planning

3.1.2.1 A fixed/floating offshore concrete structure shall be planned in such a manner that it meets 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 fulfil the required functions.

3.1.2.2 The initial planning shall include determination and description of all the functions the structure shall fulfil, 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 may influence the layout and the structural design shall be considered.

3.1.2.3 All functional requirements to the structure affecting the layout and the structural design shall be established in a clear format such that it forms the basis for the engineering process and the structural design.

3.1.2.4 Investigation of site-specific data such as seabed topography, soil conditions and environmental conditions shall be carried out in accordance with requirements of DNVGL-OS-C101, DNVGL-RP-C212, ISO 19901-1, ISO 19901-2 and ISO 19901-4 and ISO 19901-08.

3.1.3 Documentation required in the planning stage

Description of offshore concrete structure

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

3.1.3.2 The overview should be presented in three sections:

— overview of facility
— development bases and phases
— staffing philosophy and arrangements.

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

Meteorological and ocean conditions

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

3.1.3.4 The metocean/climatology conditions section should cover at least the following:

— storm/wave/current conditions
— wind
— seawater/air temperature
— earthquakes
— icebergs/ice islands/sea ice
— cyclones
— other extreme conditions
— seabed stability
— tsunami
— atmospheric stability
— range and rates of changes of barometer pressure
— precipitation
— corrosive characteristics of the air
— frequency of lightning strikes
— relative humidity.

3.1.3.5 For ground supported structures located in seismic active zones, a site specific earthquake analysis shall be performed. This analysis shall be reported in a Seismic Hazard Assessment Report where geological and seismic characteristics of the location of the ground supported facilities and the surrounding region as well as geo-tectonic information from the location shall 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 surrounding the site, and relating it to the geological and tectonic conditions.

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.

Layout of the offshore concrete structure

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

3.1.3.7 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).

Description of primary functions

— A description of primary functions 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.

3.1.3.8 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 muster station 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.

Standards

3.1.3.9 A design brief document shall include references to standards and design specifications.

3.1.4 Documentation required prior to construction

3.1.4.1 The technical documentation of a concrete structure, available prior to construction, shall comprise:

— design basis
— design calculations for the complete structure including individual members
— project specification and procedures
— drawings issued for construction and approved by design manager.

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

3.1.4.3 The project specification shall comprise:

— Method statements detailing the execution philosophy of the construction works.
— Construction drawings, giving all necessary information such as geometry of the structure, reinforcement system, amount and position of reinforcing and prestressing steel etc.
— Material specifications and product standards shall be included for cement, aggregate, additions, admixtures, concrete, reinforcement, post tensioning stemtems, grouts, structural steels, epoxies etc..
— Certificates and data-sheets defining a coherent set of material factors and characteristic material properties for design shall be provided if not provided in this standard.
— 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.
— QA framework, plans and procedures, QC procedures and instructions, ITPs.
— Work description (procedures) related to the construction activity.

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

3.1.4.5 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 material 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.

3.1.4.6 The work description should include aspects relating to grouting works including material specification, installation method, piping systems, inlet arrangement, free fall through water if appropriate, etc.

3.1.5 As-built documentation

3.1.5.1 The as-built documentation shall comprise:

- design basis
- design brief documentation
- updated design calculations
- geotechnical design report
- quality records
- method statements
- sources of materials, material test certificates and/or suppliers' attestation of conformity, works certificates, mill certificates, 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.

3.1.6 Inspection/monitoring plans for structure in service

3.1.6.1 Documentation related to monitoring and inspection of the installation shall be prepared.

SECTION 4 MATERIALS

4.1 General

4.1.1 General

4.1.1.1 The requirements regarding properties, composition, extent of testing, inspection, etc. for materials for offshore concrete structures, i.e. concrete, grout, mortar and reinforcement, are given in this section.

4.1.1.2 The materials, for all structural components and for the structure itself, shall be specified to ensure that the required quality is maintained during all stages of construction and for the intended structural life.

4.1.1.3 Materials 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.

4.1.1.4 Specifications shall be established for all relevant materials, including constituents, to be used in the manufacture of the offshore concrete structure. The specifications shall comply with the requirements in this standard.

4.1.1.5 Approval of materials, including constituents, shall be based on test data where testing was carried out in accordance with this standard or other applicable International standards or 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.

4.1.1.6 Material properties shall be documented and it shall be verified through on-going testing that they meet the requirements, as set out in the material specification.

4.1.1.7 Testing shall be witnessed and records signed by a competent person from a qualified department different from the production department. Documentation of testing and results to be in accordance with the requirements of this standard. In addition, relevant requirements stated in this section, Sec.6 and Sec.7 shall be complied with.

4.1.1.8 Materials complying with other recognized standards may be accepted as an alternative to this standard.

4.1.1.9 Materials with properties other than specified in this section may be accepted after special consideration.

4.1.1.10 For the required contents of product or type approval certificates for FRP reinforcement and for structural grout see App.F and App.H.

4.2 Concrete/grout constituents

4.2.1 General

4.2.1.1 Constituent materials for structural concrete and grout are cement, aggregates, water and typically admixtures. They may also include additions.

4.2.1.2 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 may be detrimental to the durability of the concrete or cause corrosion of the reinforcement and shall be suitable for the intended use.

4.2.1.3 Concrete includes both fine and coarse aggregates. Structural grout includes fine aggregates or, as in the case of cement grout, no aggregates.

4.2.2 Cement

4.2.2.1 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.

4.2.2.2 Cement shall be tested in accordance with an approved method. Table 4-1 gives the tests and the preferred method of testing required for documentation. References to recognized standards are given. For undated references, the latest edition of the referenced document (including any amendments) applies.

Table 4-1 Testing of cement

Property	*Method/apparatus*	*Standard references*		
		ASTM	*EN*	*ISO*
Fineness	Blaine	C204	196-6	
Chemical composition		C114	196-2	
Normal consistency	Vicat	C187	196-3	9597
Soundness	Le Chatelier		196-3	9597
Initial/final set	Vicat	C191	196-3	9597
Strength in mortar	Rilem		196-1	

4.2.2.3 The compound (mineral) composition of cements may be calculated with sufficient accuracy from Bogue's unmodified formulae, as given in ASTM C150.

Guidance note:

The tricalcium aluminate (C_3A) content calculated in accordance with this clause should preferably not exceed 10%. However, as the corrosion protection of embedded steel is adversely affected by a low C_3A 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.2.2.4 Cement shall be delivered with a works certificate (mill certificate) containing, at least, the following information:

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

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

Guidance note:

The requirement for a works certificate may be waived if the cement 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

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

— Portland cements
— Portland composite cements
— Blast furnace cements, with high clinker content.

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

— Blast furnace cements
— Pozzolanic cements
— Composite cement.

The above types of cement have characteristics specified in international and national standards. They should 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.2.3 Mixing water

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

4.2.3.2 The required water content shall 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 [4.3.2].

4.2.3.3 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.

4.2.3.4 Salt water, e.g. raw seawater, shall not be used as mixing or curing water for structural concrete.

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

4.2.3.6 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.

4.2.4 Normal weight aggregates

4.2.4.1 Aggregate source(s) (sand and gravel) shall be investigated and reviewed 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 subject to expansion.

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.

4.2.4.2 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
— petro-graphical composition and properties that may affect the durability of the concrete.

4.2.4.3 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 in quantities that may affect the properties and the durability of the concrete adversely. Examples of harmful substances are clay like and silty particles, organic materials, and sulphates and other salts.

4.2.4.4 Aggregates shall be evaluated for risk of alkali silica reaction (ASR) in concrete in accordance with 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.

4.2.4.5 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.

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

4.2.4.7 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).

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

4.2.4.9 Testing of aggregates shall be carried out at regular intervals both at the quarry and on construction site during concrete production. The frequency of testing shall 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.

4.2.5 Lightweight aggregates

4.2.5.1 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.

4.2.5.2 Lightweight aggregates shall conform to requirements contained in recognized standards, e.g. relevant ASTM, ACI or EN.

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

4.2.6 Additions

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

4.2.6.2 Additions shall not be harmful or contain harmful impurities in quantities that may 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.

4.2.6.3 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.

4.2.6.4 Additions shall be delivered with a works certificate containing relevant chemical and physical properties.

Guidance note:

The requirement for a works certificate may be waived if the additions are 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.2.7 Admixtures

4.2.7.1 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 shall be prepared to document such verification. The test report shall form a part of the concrete mix design documentation.

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

4.2.7.3 The extent of testing is normally to be in accordance with the requirements given in recognized international standards.

4.2.7.4 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.

4.2.7.5 The total volume of admixtures shall not exceed the maximum dosage recommended by the producer and not exceed 50 g per Kg of cement.

4.2.7.6 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
— compressive and flexural strength at 1 to 3 days, 28 days and 91 days.

4.2.7.7 Admixtures shall be delivered with a works certificate containing relevant chemical and physical properties.

Guidance note:

The requirement for a works certificate may be waived if the admixtures are 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.3 Concrete

4.3.1 Concrete categorization

4.3.1.1 Normal strength concrete is a concrete of grade from C35 to C65. The concrete grade is derived from the characteristic cylinder strength of concrete in accordance with, Table 4-2.

4.3.1.2 High strength concrete is a concrete of grade in excess of C65.

4.3.1.3 Lightweight aggregate concrete (LWAC) is a concrete made with lightweight aggregates.

4.3.1.4 LWAC may be composed using a mixture of lightweight aggregates (LWA) and normal weight aggregates.

4.3.2 Concrete mix

4.3.2.1 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 consistence, 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.

4.3.2.2 The required properties of fresh and hardened concrete shall be verified by the use of recognized testing methods, international standards or recognized national standards. Recognized standards are ASTM, ACI and EN standards.

4.3.2.3 Compressive strength shall always be specified; in addition tensile strength, Young's modulus (E-modulus) and fracture energy may be specified. Properties which may cause cracking of structural concrete shall be accounted for, i.e. creep, shrinkage, heat of hydration, thermal expansion and similar effects.

4.3.2.4 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.

Guidance note:

The evaluation of the effective water/cement ratio may be undertaken in accordance with ISO 19903.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.3.2.5 The durability of structural concrete shall be 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 not be higher than 0.40.

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} to 10^{-8} m/sec). The test should 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, see [4.3.2.9].
— Low water-cement ratio
— Good concreting practice and workmanship ensuring adequate workability, proper handling, transportation, placing
— and consolidation, and no segregation.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.3.2.6 Concrete subjected to freezing and thawing shall 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 in accordance with ASTM C457 not exceeding 0.25 mm with no single results greater than 0.30 mm.

Air content readings taken from concrete after pumping or slip forming may be lower than those measured from the mixer. This is especially true for high performance concretes (HPC) with low W/C ratios. Where HPC has been specified, the freeze thaw resistance of the as-placed material may alternatively be documented through cyclical freeze-thaw resistance testing in accordance with ASTM C666 Procedure A or another recognised international standard. The acceptance limit applied during testing in accordance with ASTM C666 *Procedure A* should be a minimum durability factor of 90% after 300 cycles.

4.3.2.7 To improve the resistance against attacks from salts in the seawater, cement with a moderate C_3A content may be used, see [4.2.2].

4.3.2.8 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.

4.3.2.9 In the splash zone the cement content shall not 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 shall be established by appropriate testing.

See also [4.3.2.4].

4.3.2.10 The concrete grades are defined in [4.3.3]. The properties of hardened concrete are generally related to the concrete grade. For concrete exposed to sea water the minimum grade is C40.

Prestressed reinforced concrete structures shall not be designed with concrete grade less than C40. For concrete exposed to abrasion and scouring actions (due to ice, pebbles, sand or silts), concrete grade shall not be less than C60 and the aggregates should be abrasion resistant.

4.3.2.11 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

4.3.2.12 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.

4.3.3 Concrete characteristic strength

4.3.3.1 For concrete, the 28 days characteristic compressive strength f_{cck} shall be defined as the lower 5th percentile fround from statistical analysis of tests on cylindrical specimens with diameter 150 mm and height 300 mm.

4.3.3.2 The normalized in-situ compression strength, f_{cn}, of normal weight concrete shall be determined from the following formula for concrete with concrete grade between C35 and C90:

$$f_{cn} = f_{cck} \cdot (1 - f_{cck}/600)$$

where:

f_{cck} = characteristic concrete compressive cylinder strength in Table 4-2.

4.3.3.3 The characteristic tensile strength, f_{tk}, of the material shall be determined as the 50% quantile in the probability distribution of the strength data. The characteristic strength data shall be estimated with at least 75% confidence.

4.3.3.4 The normalized in-situ tensile strength, f_{tn}, of normal weight concrete shall be determined from the following formula for concrete with concrete grade between C35 and C90:

$$f_{tn} = f_{tk} \cdot (1 - (f_{tk}/25)^{0.6})$$

where:

$f_{tk} = 0.48\ (f_{cck})^{0.5}$

f_{tk} may alternatively be determined in accordance with the provisions in [4.3.3.12] or [4.3.3.13].

For $f_{cck.}$ larger than 65 MPa, testing is required to determine f_{tk}.

4.3.3.5 A factor $(1 - f_{cck}/600)$ is applied on the characteristic compression cylinder strength, f_{cck}, and considers transition of cylinder strength into *in-situ* strength, etc.

4.3.3.6 Normal weight concrete has grades identified by [4.3] and lightweight aggregate concrete grades are identified by the symbol LC. The grades are defined in Table 4-2 and Table 4-3 as a function of the characteristic compression cylinder strength of concrete, f_{cck}.

Table 4-2 Properties for normal weight (NW) concrete grades

Concrete grade	*C30*	*C35*	*C40*	*C45*	*C50*	*C55*	*C60*	*C65*	*C70*	*C80*	*C90*
f_{cck} [MPa] 1)	30	35	40	45	50	55	60	65	70	80	90
f_{cn} [MPa] 2)	28.5	33.0	37.3	41.6	45.8	50.0	54.0	58	61.8	69.3	76.5
f_{tk} [MPa] 3)	2.63	2.84	3.04	3.22	3.39	3.56	3.72	3.87	4.02	4.29	4.55
f_{tn} [MPa] 4)	1.95	2.07	2.18	2.28	2.37	2.45	2.53	2.61	2.68	2.80	2.91

1) f_{cck} = characteristic cylinder compressive strength

2) f_{cn} = normalized in-situ compression strength

3) f_{tk} = characteristic mean tensile strength

4) f_{tn} = normalized in-situ tensile strength

Table 4-3 Properties for lightweight aggregate concrete (LWAC) grades

Concrete grade	*LC30*	*LC35*	*LC40*	*LC45*	*LC50*	*LC55*	*LC60*	*LC70*	*LC80*
f_{cck} [MPa] [1)]	30	35	40	45	50	55	60	70	80
f_{cn} [MPa] [2)]	28.5 × η	33.0 × η	37.3 × η	41.6 × η	45.8 × η	50.0 × η	54.0 × η	61.8 × η	69.3 × η
f_{tk} [MPa] [3)]	2.63 × η	2.84 × η	3.04 × η	3.22 × η	3.39 × η	3.56 × η	3.72 × η	4.02 × η	4.29 × η
f_{tn} [MPa] [4)]	1.95 × η	2.07 × η	2.18 × η	2.28 × η	2.37 × η	2.45 × η	2.53 × η	2.68 × η	2.80 × η

[1)] f_{cck} = characteristic cylinder compressive strength
[2)] f_{cn} = normalized in-situ compression strength
[3)] f_{tk} = characteristic mean tensile strength
[4)] f_{tn} = normalized in-situ tensile strength,
$\eta = (0.15 + 0.85\, \rho/\rho_1)$ where $\rho_1 = 2200$ kg/m^3, ρ = Density of the lightweight aggregate concrete.

4.3.3.7 The strength values given in Table 4-3 apply to lightweight aggregate concrete with the following limitations and modifications:

$$f_{cck} \leq f_{cck2}\left(\frac{\rho}{\rho_1}\right)^2$$

Unless tensile strength is determined by testing, tensile strength, f_{tk}, and normalized in-situ strength, f_{tn}, of lightweight aggregate concrete shall be multiplied by the factor η equal to $(0.15 + 0.85\, \rho/\rho_1)$ as shown in Table 4-3.

For lightweight aggregate concrete with intended concrete strength $f_{cck} > f_{cck3}\, (\rho/\rho_1)^2$, it shall be shown by test samples that a characteristic strength, 15% higher than the intended, is achieved. The tests shall be carried out on concrete samples using the same material composition as intended.

In the above,

f_{cck2} = 95 MPa
f_{cck3} = 65 MPa
ρ = density of the lightweight aggregate concrete
ρ_1 = 2200 kg/m^3.

4.3.3.8 Lightweight aggregate concretes shall be documented for suitability for the intended application. The properties shall be documented in accordance with [4.3.2], as a minimum:

- Workability.
- Density.
- Young's modulus.
- Durability, see [4.3.2.5].
- Characteristic compression cylinder strength, f_{cck} (based on 150 × 300 mm cylinders).
- Characteristic tensile strength, f_{tk} see [4.3.3.12] and [4.3.3.13].

— Fatigue strength parameter, C_5, of the concrete. The factor C_5 determines the relationship between static reference strength, f_{rd}, and fatigue reference strength, $f_{rd, fat}$. The relationship is determined as $f_{rd.fat} = C_5 \cdot f_{rd}$. See [6.13.2].
— In some cases it may be appropriate to document the properties and characteristics of the lightweight aggregate, especially its durability and reactivity for application in the marine environment.

Guidance note:

S-N curves should be presented in a format compatible with S-N curves for concrete as presented in [6.13.2] in order to use the provisions for design for fatigue limit state in this standard.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.3.3.9 For normal density concrete of grade higher than C60 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 is additional to regular quality control testing if that is carried out on cube specimens only.

4.3.3.10 In cases where on-site QC specimens are cast from cubes or cylinders smaller than those used to define the characteristic compressive strength, a conversion factor between the QC size specimens and the 150 mm x 300 mm test cylinders shall be determined. If established conversion factors are available between specimen sizes for similar grades of concrete in the relevant national standards these may be used.

4.3.3.11 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.

4.3.3.12 For concrete exposed to temperatures below -60°C, the possible strength increase in compressive and tensile strength may be utilized in design for these conditions provided the strength is 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 crack widths.

4.3.3.13 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 EN 12390-6 or ISO 1920-4. The characteristic uniaxial tensile strength, f_{tk}, shall be taken as 0.8 of the characteristic splitting strength determined by testing.

Guidance note:

The reference cylinder size to find the characteristic splitting strength for use with this standard should be taken as 150 × 300 mm. Tests conducted on other specimen sizes should be accompanied by a conversion factor to convert the results to those of 150 × 300 mm cylinders.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.3.3.14 The characteristic tensile strength of the concrete, f_{tk}, may be obtained by determining the modulus of rupture by the testing of the unreinforced beams at 28 days in accordance with ASTM, ACI or EN standards. The characteristic tensile strength, f_{tk}, shall be taken as 0.6 of the characteristic modulus of rupture determined by testing.

Guidance note:

When deriving the characteristic modulus of rupture value, the four-point loading setup should be taken as the reference test setup. It has been shown that testing conducted using a three-point bending method may yield results approximately 13% higher than the four-point bending setup. If the three-point method is employed, the resulting characteristic modulus of rupture value should be adjusted accordingly.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.3.3.15 The normalized Young's modulus of concrete is controlled by the Young's modulus of its components. Approximate values for the Young's modulus E_{cn}, is taken as the secant value between $\sigma_c = 0$ and $0.4\ f_{cck}$. Approximate values for quartzite aggregates may be determined from the following equation:

$$E_{cn} = 22\ 000 \cdot (f_{cck}/10)^{0.3} \text{ MPa for } f_{cck} \leq 70\text{MPa}$$

$$E_{cn} = 4800 \cdot (f_{cck})^{0.5} \text{ MPa for } f_{cck} > 70 \text{ MPa}$$

For limestone and sandstone aggregates, the value should be reduced by 10% and 30% respectively. For basalt aggregates, the value should be increased by 20%.

4.3.3.16 For rehabilitation or for 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 scaling factor may be used in predicting the cylinder strength.

Table 4-4 Scaling factor on drilled core results

Height/diameter ratio	2.00	1.75	1.50	1.25	1.10	1.00	0.75
Scaling factor on strength values	1.00	0.97	0.95	0.93	0.89	0.87	0.75

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 4-2 and Table 4-3 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.

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

$$f_{cckt} = 300 - 10 \cdot (900 - 6 \cdot f_{cckj})^{0.5}$$

where:

f_{cckj} = the characteristic strength of the taken specimens converted into cylinder strength for cylinders with height/diameter ratio 2:1

f_{cckt} = the characteristic compressive cylinder strength at 28 days based on in-situ tests.

For design, f_{cckt}, replaces, f_{cck}, the characteristic concrete compressive strength in Table 4-2 and Table 4-3.

4.4 Fibre reinforced concrete

4.4.1 Material requirements of fibre reinforced concrete

4.4.1.1 The constituent materials of fibre reinforced concrete are cement, fine sand, aggregates, water, admixtures and short fibre material mixed to get a uniform matrix. The fibres may either be made of steel or FRP.

4.4.1.2 The normalized Young's modulus of fibre reinforced concrete is controlled by the Young's modulus of its components. Approximate values for the Young's modulus E_{cn}, is taken as the secant value between σ_c

= 0 and 0.4 f_{cck}. The following guidelines to estimate E_{cn} may be used initially in a project until the actual values are determined from testing:

Guidance note:

The following guidelines may be used for fibre reinforced concrete with quartzite aggregates:

$$E_{cn} = 22\,000 \cdot (f_{cck}/10)^{0.3} \text{ MPa for } f_{cck} \leq 70\text{MPa}$$

$$E_{cn} = 4800 \cdot (f_{cck})^{0.5} \text{ MPa for } f_{cck} > 70 \text{ MPa}$$

For limestone and sandstone aggregates, the value should be reduced by 10% and 30% respectively. For basalt aggregates, the value should be increased by 20%.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.4.1.3 The workability and quality of the mixed fibre reinforced concrete depends on the amount and length of the fibres in the mix. The workability and quality of the fibre reinforced concrete shall be documented prior to use.

4.4.1.4 The fibres shall be of sufficient length to provide bond between the concrete matrix and the fibres.

4.4.1.5 The concrete material in fibre reinforced concrete shall be in accordance with [4.3.1] and [4.3.2].

4.4.1.6 The characteristic concrete compressive cylinder strength, f_{cck}, of the material shall be determined as the 5% quantile in the probability distribution of the strength data. The characteristic strength data shall be estimated with at least 75% confidence.

4.4.1.7 f_{cck} shall be determined on standard cylinders, with 150 mm diameter and 300 mm height, tested in accordance with a recognized standard (ASTM, ACI or EN).

4.4.1.8 The characteristic tensile strength, f_{tk}, of the material shall be determined as the 50% quantile in the probability distribution of the strength data. The characteristic strength data shall be estimated with at least 75% confidence.

4.4.1.9 The characteristic tensile strength, f_{tk}, may be determined from converting the characteristic tensile strength from tensile splitting tests or from modulus of rupture tests to direct tensile strength if sufficient test data is available.

4.4.1.10 Fibre distribution for the intended application shall be assessed by a representative full-scale mock-up test. An appropriate number of specimens shall be taken from the mock-up test to identify how the formwork, method of casting and casting direction affect the fibre distribution within the structure. Resulting reduction factors to account for the fibre distribution should be established to take the effect into account for design of the fibre reinforced concrete.

4.4.1.11 The characteristic tensile strength of fibre reinforced concrete will increase as a function of the volumetric percentage of fibres mixed into the concrete. The tensile strength increases more for steel fibres than FRP fibres. For both cases it is a precondition that the fibres are mixed uniformly through the concrete. The increase in characteristic tensile strength of fibre reinforced concrete shall be documented by testing. The same type, length, volume and quality of fibre shall be used in the test.

4.4.1.12 The normalized compression strength, f_{cn}, of fibre reinforced concrete may be determined from the following formula.

$$f_{cn} = f_{cck} \cdot (1 - f_{cck}/600)$$

where:

f_{cck} = characteristic concrete cylinder strength of the fibre reinforced concrete

The factor (1-f_{cck}/600) is applied on the characteristic compression cylinder strength, f_{cck}, and considers transition of cylinder strength into in-situ strength, brittleness, etc.

4.4.1.13 The normalized tensile strength, f_{tn}, of normal weight fibre reinforced concrete may be determined from the following formula for concrete with concrete grade between C35 and C90.

$$f_{tn} = f_{tk} \cdot (1-(f_{tk}/25)^{0.6})$$

4.4.1.14 Prior to using fibre reinforced concrete in a structure, the composite concrete mix shall be documented for suitability for the intended application. The following properties of the fibre reinforced concrete shall be documented as a minimum:

— Workability.
— Young's modulus.
— Characteristic compression cylinder strength of the fibre reinforced concrete, f_{cck}.
— f_{tk}, the characteristic tensile strength of the fibre reinforced concrete (see [4.4.1.9] and [4.4.1.10] above).
— Fatigue strength parameter, C_5, of fibre reinforced concrete. The factor C_5 determines the relationship between static reference strength, f_{rd}, and fatigue reference strength, $f_{rd,\ fat}$. The relationship is determined as $f_{rd.fat} = C_5 \cdot f_{rd}$. See [6.13.2].
— The concrete material itself without fibre shall be documented in accordance with the general requirements for concrete in [4.3].

Guidance note:

S-N curves should be presented in a format compatible with S-N curves for concrete as presented in [6.13.2] in order to use the provisions for design for FLS in this standard.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.4.1.15 Static shear strength may increase due to the addition of fibre and the associate increased tensile strength. This possible increase in shear strength shall be documented for the fibre reinforced concrete member. The same type, length, volume and quality of fibre shall be used in the test. Four point bending tests shall be carried out on beams.

Guidance note:

The test specimen should have a minimum dimension of h = 200 mm, b = 100 mm, where h and b are the depth and width respectively of the specimen. The length of the specimen shall be minimum 1350 mm and the shear span, a, minimum 500 mm, i.e. a/h > 2.5. The concrete specimen should be reinforced with longitudinal steel reinforcement. The purpose of this test is to verify the contribution of the tensile strength of the fibre reinforced concrete into the shear strength formula in [6.6.2] based on f_{tk} and the design methodology method in this standard.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.4.1.16 The durability of the fibres for the application shall be documented. Steel fibre reinforced concrete shall not be used in concrete exposed to environmental classes XD2, XS2, XF1 and XA1 or higher. For these exposure classes fibers resistant to corrosion shall be used. FRP and stainless steel fibres are resistant for corrosion. The durability of FRP fibres shall be documented when exposed to marine environment.

4.4.1.17 Crack width predictions depend on the tensile strength of concrete. The higher the tensile strength the longer the distance is between cracks and the wider the crack width becomes. Beams tests shall be carried out to document the relationship between crack width and tensile strength for the actual fibres to be used.

4.5 Structural grout

4.5.1 Material requirements

4.5.1.1 The constituents of grout are cement, water and often admixtures, fine aggregates may also be included. These shall meet the same requirements as those given in [4.2]. Structural grout in this standard shall have a characteristic compressive strength higher than 35 MPa.

Structural grout may be pre-blended, both pre-packed and bulk/silo stored/transported, and cement grout.

4.5.1.2 The normalized Young's modulus of structural grout is controlled by the Young's modulus of its components. Approximate values for the Young's modulus, E_{cn}, is taken as the secant value between $\sigma_c = 0$ and $0.4\ f_{cck}$.

Guidance note:

The following guidelines may be used initially in a project until the actual parameters are determined from testing of structural grout with quartzite aggregates:

$$E_{cn} = 22\ 000 \cdot (f_{cck}/10)^{0.3} \text{ MPa for } f_{cck} \leq 70\text{MPa}$$

$$E_{cn} = 4800 \cdot (f_{cck})^{0.5} \text{ MPa for } f_{cck} > 70 \text{ MPa}$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.5.1.3 All grout constituent materials shall be proportioned by mass except the mixing water, which may alternatively be proportioned by volume. The water/cement ratio shall not be higher than 0.45.

Guidance note:

The proportioning of site-batched grout should be within an accuracy of 2% for cement and admixtures and 1% for water.

Grout intended for use in the marine environment should have a minimum cement content of 600 kg/m^3.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.5.1.4 Maximum aggregate size shall be specified based on the intended application, for example space in between forms and placing method (size of the hose, pumping head, etc.).

4.5.1.5 The in-place properties of the grout material shall be documented by appropriate large scale test setups (mock-up tests) in advance of the grouting operation. The test-setup shall reflect the actual conditions and equipment at the site including a realistic typical hose diameter and length to assess pumpability of the material. Further guidelines regarding mock-up tests are given in App.J.

Further, if contingency procedures involve other grout placement configurations these shall be reflected in the test setups. Full filling of the intended volume shall be demonstrated and documented.

Guidance note:

It is of high importance that the structural grout has volumetric stability in order to fill the intended volume, as high autogenous and/or drying shrinkage in the grout will reduce the load capacity of the structural element. Assessment of volumetric stability should therefore be documented prior to commencement of operations.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.5.1.6 The grout mix used for injection in prestressing ducts shall be designed for the specified properties which shall at least include: fluidity and bleeding (in the plastic condition), autogenous shrinkage and compressive strength (in hardened condition).

4.5.1.7 The properties of structural grout shall be documented in a product or type approval certificate, defining at least the following limitations and properties:

— Main operational limitations: qualified temperature for grout application, thickness range, pumping length range and elevation head for specific hose diameter.

- General properties: density, flowability, setting time (initial and final), air content, stability etc.
- Mechanical properties: shrinkage, creep, characteristic compressive cylinder strength, Young's modulus, Poisson's- ratio and splitting tensile strength or flexural strength. In all cases, mean value, standard deviation, and number of samples tested shall be reported. If property evolution with time and temperature is of interest for the intended application, this shall be documented.
- Fatigue strength parameter, C_5, of the grout determines the relationship between static reference strength f_{rd} and fatigue reference strength $f_{rd,\ fat}$. The relationship is determined as $f_{rd,\ fat} = C_5 \cdot f_{rd}$. Reference is made to [6.13.2].

For complete overview of required documentation, see App.H.

Guidance note:

S-N curves should be presented in a format compatible with S-N curves for concrete as presented in [6.13] in order to use the provisions for design for FLS in this standard.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.5.1.8 For grout, the 28 days characteristic compression strength, f_{cck}, shall be determined as the 5% quantile in the probability distribution of the strength data from statistical analysis of tests on cylindrical specimens with diameter 150 mm and height 300 mm. The characteristic strength data shall be estimated with at least 75% confidence.

4.5.1.9 The characteristic tensile strength, f_{tk}, of the material shall be determined as the 50% quantile in the probability distribution of the strength data. The characteristic strength data shall be estimated with at least 75% confidence.

Guidance note:

As a guideline f_{tk} may be obtained from the following equation:

$$f_{tk} = 0.48 \cdot (f_{cck})^{0.5}$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.5.1.10 When test data for direct tensile strength is not available, the characteristic tensile strength, f_{tk}, may be determined by conversion from tensile splitting strength or flexural strength tests.

Guidance note:

The conversion factor to obtain the characteristic tensile strength may be taken as 0.8 for the tensile splitting strength and 0.4 for the flexural strength. The characteristic strength is defined in [4.5.1.9].

The reference cylinder size to find the characteristic tensile splitting strength for use with this standard should be taken as 150 mm × 300 mm. Tests conducted on other specimen sizes should be accompanied by a conversion factor to convert the results to those of 150 mm × 300 mm cylinders.

When deriving the characteristic flexural strength value the fourpoint bending set up should be taken as the reference test setup. It has been shown that testing conducted using a three-point bending method yields results 13% higher than the four-point bending setup. If the three-point method is employed the resulting characteristic flexural strength value shall be adjusted accordingly.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.5.1.11 In cases where on-site QC specimens are cast from cubes or cylinders smaller than those used to define the characteristic compressive strength, a conversion factor between the QC size specimens and the 150 mm x 300 mm test cylinders shall be determined.

4.5.1.12 The normalized compression strength, f_{cn}, of structural grout shall be determined from the following formula:

$$f_{cn} = f_{cck} \cdot (1-f_{cck}/600)$$

where:

f_{cck} = characteristic concrete cylinder compression strength of the structural grout.

The factor $(1\text{-}f_{cck}/600)$ is applied on the characteristic compression cylinder strength, f_{cck}, and considers transition of cylinder strength into *in-situ* strength, brittleness, etc.

4.5.1.13 The normalized tensile strength, f_{tn}, of structural grout shall be determined from the following formula:

$$f_{tn} = f_{tk} \cdot (1-(f_{tk}/25)^{0.6})$$

4.5.1.14 App.I provides guidelines on QA/QC systems for the manufacturing of structural grout.

4.5.1.15 For requirements to general grouting operations see [7.17].

4.5.2 Pre-blended grout

4.5.2.1 Pre-blended grout shall be tested and delivered in accordance with a standard (ASTM, ISO or EN). Recommended testing for fresh and hardened grout is given in App.H.

4.5.2.2 Pre-blended grout shall be delivered with a type approval certificate stating at least the limitations and properties specified in App.H.

4.5.2.3 The grout manufacturer shall have a quality system in operation that accounts for the requirements in this section and ensures full traceability of the grout manufacture. The responsibility for operation of the quality system shall be with one dedicated person at the manufacturing site.

4.5.2.4 The extent of production testing shall be sufficient to confirm conformity of the as-produced grout with the type approval certificate, see App.I.

4.5.2.5 The plan for the tests during production shall be specified by the grout manufacturer and included in the QA system of the manufacturing plant, the QA system shall as a minimum include the requirements specified in App.I.

4.6 Fibre reinforced structural grout

4.6.1 Material requirements for fibre reinforced structural grout

4.6.1.1 The constituent materials of fibre reinforced grout are cement, fine aggregates, water, admixtures and short fibre material mixed to get a uniform matrix. The short fibre material may either be made of steel or FRP. Fibre reinforced structural grout in this standard shall have a characteristic compressive strength higher than 35 MPa. Fibre reinforced structural grout may be pre-blended or cement grout.

4.6.1.2 The normalized Young's modulus of fibre reinforced grout is controlled by the Young's modulus of its components. Approximate value for the Young's modulus E_{cn}, is taken as the secant value between $\sigma_c = 0$ and $0.4\ f_{cck}$.

Guidance note:

The following guidelines may be used initially in a project until the actual parameters are determined from testing of fibre reinforced grout with quartzite aggregates:

$$E_{cn} = 22\ 000 \cdot (f_{cck}/10)^{0.3} \text{ MPa for } f_{cck} \leq 70 \text{ MPa}$$

$$E_{cn} = 4800 \cdot (f_{cck})^{0.5} \text{ MPa for } f_{cck} > 70 \text{ MPa}$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.6.1.3 The workability and quality of mixing the fibre reinforced grout depend on among other properties, the amount and length of fibres in the mix. The workability and quality of the fibre reinforced grout shall be documented prior to use.

4.6.1.4 The fibres shall be of sufficient length to provide bond between the grout matrix and the fibres.

4.6.1.5 The grout material in fibre reinforced grout shall be in accordance with the requirements in [4.5.1] and [4.5.2].

4.6.1.6 The characteristic compression cylinder strength, f_{cck}, shall be determined as the 5% quantile in the probability distribution of the strength data. The characteristic strength data shall be estimated with at least 75% confidence.

4.6.1.7 f_{cck} shall be determined on water cured standard cylinders of size 150 mm diameter and 300 mm high tested in accordance with a recognized standard (ASTM, ACI or EN).

4.6.1.8 The characteristic tensile strength, f_{tk}, of the material shall be determined as the 50% quantile in the probability distribution of the strength data. The characteristic strength data shall be estimated with at least 75% confidence.

4.6.1.9 The characteristic tensile strength, f_{tk}, may be determined from converting the characteristic tensile strength from tensile splitting tests or from flexural strength tests to direct tensile strength if sufficient data is available.

4.6.1.10 Fibre distribution for the intended application shall be assessed by a representative full scale mock-up test. An appropriate number of specimens shall be taken from the mock-up test to identify how the formwork, method of casting and casting direction affect the fibre distribution within the structure. Resulting reduction factors to account for the fibre distribution should be established to take the effect into account for design of the fibre reinforced grout.

4.6.1.11 The characteristic tensile strength of fibre reinforced grout will increase as a function of the volumetric percentage of fibres mixed into the grout. The tensile strength increases more for steel fibres than FRP fibres. For both cases it is a precondition that the fibres are mixed uniformly through the grout. The increase in characteristic tensile strength of fibre reinforced concrete shall be documented by testing. The same type, length, volume and quality of fibre shall be used in the test.

4.6.1.12 The normalized compression strength, f_{cn}, of fibre reinforced grout may be determined from the following formula:

$$f_{cn} = f_{cck} \cdot (1-f_{cck}/600)$$

where:

f_{cck} = characteristic grout cylinder strength of the fibre reinforced grout.

The factor $(1-f_{cck}/600)$ is applied on the characteristic compression cylinder strength, f_{cck}, and considers transition of cylinder strength into in-situ strength, brittleness, etc.

4.6.1.13 The normalized tensile strength, f_{tn}, of fibre reinforced grout shall be determined from the following formula:

$$f_{tn} = f_{tk} \cdot (1-(f_{tk}/25)^{0.6})$$

4.6.1.14 Prior to using fibre reinforced grout in a structure it shall be documented for suitability for the intended application and be delivered with a product certificate. The following properties of the grout shall be documented as a minimum:

- Main operational limitations: qualified temperature for grout application, thickness range, pumping length range and elevation head for specific hose diameter.
- General properties: density, flowability, setting time (initial and final), air content, stability etc.

- Mechanical properties: shrinkage, creep, characteristic compressive cylinder strength, Young's modulus, Poisson's- ratio and splitting tensile strength or flexural strength. In all cases, mean value, standard deviation, and number of samples tested shall be reported. If property evolution with time and temperature is of interest for the intended application, this shall be documented.
- Fatigue strength parameter, C_5, of the grout determines the relationship between static reference strength f_{rd} and fatigue reference strength $f_{rd,\ fat}$. The relationship is determined as $f_{rd,\ fat} = C_5 \cdot f_{rd}$. Reference is made to [6.13.2].
- The grout material without fibres shall be documented in accordance with the requirements in [4.5.1].

For complete overview of required documentation see App.H.

4.6.1.15 Static shear strength may increase due to the addition of fibres and the associated increased tensile strength. This possible increase in shear strength shall be documented for the fibre reinforced grout member. The same type, length, volume and quality of fibre shall be used in the test.

4.6.1.16 The durability of the fibres for the application shall be documented. Steel fibre reinforced grout shall not be used in the cover zone and exposed to environmental classes XD2, XS2, XF1, XA1 or higher in grouted connections, clamps, etc. unless its durability in these environments is documented. The durability of FRP fibres shall be documented when exposed to a marine environment.

4.6.1.17 Crack width predictions depend on the tensile strength of grout. The higher the tensile strength the longer the distance is between cracks and the wider the crack width becomes. Four point beam tests shall be carried out to document the relationship between crack width and tensile strength for the actual fibre to be used.

4.6.1.18 App.I provides guidelines on QA/QC systems for the manufacturing of Structural Grout, these requirements shall also apply to the manufacturing of fibre reinforced grout.

4.6.2 Preblended grout with fibres

4.6.2.1 Pre-blended grout with fibres shall be tested and delivered in accordance with a standard recognized in the place of use. Recognized relevant standards are ASTM, ISO and EN. Recommended testing for fresh and hardened grout is given in App.H.

4.6.2.2 Pre-blended structural grout with fibres shall be delivered with a product certificate stating at least the limitations and properties in App.H.

4.6.2.3 The grout manufacturer shall have a quality system in operation that accounts for the requirements in this section and ensures full traceability of the grout manufacture. The responsibility for operation of the quality system shall be with one dedicated person at the manufacturing site.

4.6.2.4 The extent of production testing shall be sufficient to confirm conformity of the as-produced grout with the product certificate.

4.6.2.5 The plan for the tests during production shall be specified by the grout manufacturer and included in the QA system of the manufacturing plant, the QA system shall as a minimum include the requirements specified in App.I.

4.7 Steel reinforcement

4.7.1 General

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

4.7.1.2 Reinforcing steel shall comply with ISO 6935 Part 2 incorporating the supplemental requirements specified in App. K. Welded fabric shall comply with ISO 6935 Part 3. .

When specifying reinforcement bars the following shall be addressed:

— hot-rolled
— ribbed bars
— crescent shaped ribs
— weldable quality
— grade B500B or B500C, or equivalent
— high ductility.

Other international standards for reinforcing steel may also be used provided similar quality can be documented. See App. K for requirements to be fulfilled. If these are not met, a gap analysis shall be performed and consequences of gaps on the provisions of this standard shall be evaluated.

4.7.1.3 Consistency shall be ensured between material properties assumed in the design and requirements of this standard. Where the use of seismic detailing is required, the reinforcement provided shall meet the ductility requirements of this standard and the applicable national regulations. The requirements for detailing provided in this standard do not ensure sufficient ductility that may be required for seismic loading. In this case, ductility shall be documented.

4.7.1.4 Reinforcement steel 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.

4.7.1.5 Galvanised ordinary reinforcement may be used where provisions are made to ensure that there is no adverse reaction between the coating and the cement which would have a detrimental effect on the bond to the galvanised reinforcement.

4.7.1.6 Stainless steel may be used provided similar quality as that for ordinary steel reinforcement can be documented, see [4.7.1.2] for requirements. Note that the requirements of [K.2.2] are not valid for stainless steel reinforcement.

4.7.1.7 Epoxy coated ordinary reinforcement may be used provided similar quality as that for ordinary steel reinforcement can be documented, see [4.7.1.2] for requirements.

4.7.1.8 Tempcore reinforcement may be used provided similar quality as that for ordinary steel reinforcement can be documented, see [4.7.1.2] for requirements. .

4.7.2 Mechanical splices and end anchorages for reinforcement

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

4.7.2.2 Mechanical splices and end anchorages shall be delivered with a product or type approval certificate.

4.7.2.3 Friction welded end anchorages on reinforcement bars (T-heads) shall be 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.

4.7.3 Approval of welding procedures

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

4.7.4 Steel reinforcement characteristic strength

4.7.4.1 For reinforcement steel, the characteristic strength, f_{yk}, is determined as the 5% defective fractile.

4.7.4.2 For the fatigue limit state (FLS), the characteristic SN-curve shall be determined statistically as a 2.5% defective fractile for reinforcement, couplers, welded connections, etc. unless other values are specified in the reference standard for that design.

4.8 Steel prestressing reinforcement

4.8.1 General

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

4.8.1.2 Prestressing steel shall be delivered with a works certificate.

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

4.8.1.4 For use in the marine environment, possible negative effects of the marine environment on the fatigue strength shall be accounted for in the Wöhler curves.

4.8.2 Components for the prestressing system

4.8.2.1 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.

4.8.2.2 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.

4.8.2.3 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.

4.8.2.4 Components for the prestressing system shall be delivered with a product certificate.

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

4.8.2.6 Parameters needed to calculate friction losses, between the prestressing steel and the ducts/sheaths, anchorage loss and steel relaxation shall be documented.

4.8.3 Steel prestressing reinforcement characteristic strength

4.8.3.1 For prestressed reinforcement the characteristic strength is equal to the yield strength f_{sy} or the 0.1-proof stress determined as the 5% defective fractile.

4.8.3.2 For the fatigue limit state FLS, the characteristic SN-curve shall be determined statistically as a 2.5% defective fractile for reinforcement, prestressing assemblies, couplers, etc. unless other values are specified in the reference standard for that design.

4.9 Fibre reinforced polymer reinforcement

4.9.1 General

4.9.1.1 The scope of the provisions for FRP materials in this standard is limited to bars of carbon, glass, aramid or basalt fibre reinforced composites.

4.9.1.2 The requirements in this section do not cover subsequent machining, assembly into semifinished products such as nets or cages and issues regarding construction on site such as storage and handling of the bars, assembly of reinforcement and casting of the concrete.

4.9.1.3 FRP reinforcement bars shall be suitable for the intended service conditions and shall have adequate properties with respect to strength, elongation to break, time to rupture, fatigue, toughness, bond properties, alkali resistance, and chemical composition. These properties shall be determined by a recognized test method and specified by the supplier. Testing requirements are given in [4.14.11].

4.9.1.4 Consistency shall be ensured between bar properties assumed in the design and requirements of the standard used. In general, FRP bars shall be used with the load bearing fibres oriented predominantly in the longitudinal direction of the bars and with a cross-section that varies such as to provide interlocking in the concrete and a surface that provides adequate bonding to the concrete.

4.9.1.5 FRP bars shall be delivered with a product or type approval certificate. The parameters to be shown on the certificate are specified in App.F. All bars shall be clearly identifiable.

4.9.1.6 Coated reinforcement may be used provided the requirements to mechanical properties for ordinary reinforcing bars are met, the effect of the coating on bonding is documented and the coating process is covered by the QA/QC system of the bar manufacturer.

4.9.1.7 Main sub-contractors and raw material suppliers of the bar manufacturer should operate a quality system that is formally accepted by the bar manufacturer.

4.9.1.8 The bar manufacturer shall have a quality system in operation that accounts for the requirements in this section and ensures full traceability of the bar manufacture. The responsibility for operation of the quality system shall be with one dedicated person at the manufacturing site.

4.9.1.9 App.G provides guidelines on QA/QC systems for the manufacturing of FRP bars.

4.9.2 Mechanical splices and anchorages for fibre reinforced polymer reinforcements

4.9.2.1 Anchorage and splicing arrangements shall be restricted to types that have been qualified for the bar type and dimension in question.

4.9.2.2 Mechanical splices and end anchorages for FRP bars shall be delivered with a product or type approval certificate.

4.9.3 Fibre reinforced polymer prestressed bars

4.9.3.1 FRP reinforcing bars may be used as prestressing bars in reinforced concrete structures. The prestressing may be either a pre-tensioning system or a post-tensioning system, see [6.1.8].

4.9.4 Fibre reinforced polymer reinforcement characteristic strength

4.9.4.1 The properties of the FRP bars shall be documented by relevant recognized tests. As a minimum, the testing described in [4.14.11] shall be performed. Strength and stiffness values shall be represented in terms of characteristic values.

4.9.4.2 Characteristic bar properties for use in design shall be determined in advance from tests on specimens representative of continuous production and specified in the product or type approval certificate or in a data-sheet attached with the certificate.

Guidance note:

The coefficient of variation used for design should be assumed with care. It is advisable to assume a conservative, large value, to make sure that variations that may occur in production but are not reflected in the tested sample are accounted for.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.9.4.3 The characteristic strength of FRP bars is equal to the characteristic short term strength of FRP bars, which shall be defined the as the lower 5^{th} percentile with 75% confidence level from the sample mean and standard deviation of strength data from tests on a representative sample of specimens.

4.9.4.4 The characteristic time to rupture curve of FRP bars shall also be defined as the lower 5^{th} percentile with 75% confidence level from the sample mean and standard deviation of life data from tests on a representative sample of specimens.

4.9.4.5 The design temperatures are reference temperatures representing the intended use. The standard reference temperature is room temperature (20-23°C). Material factors determined from test data obtained at room temperature shall be modified by the application of temperature conversion factors, η_T, determined through testing at relevant temperatures.

Guidance note:

η_T may be assumed to be equal to 1.0 for application in the temperature range from -20°C to 20°C. η_T should be determined for the full range of application temperatures.

For intended service in tropical areas and for documentation of fire resistance, a temperature representative of the maximum temperature that the FRP bars will be exposed to in the specified design conditions should be used. This temperature may account for measures taken to limit the temperature such as cooling measures implemented on sun-exposed surfaces, cover thicknesses used and insulation/fire protection applied. For intended service in arctic conditions and cryogenic service, an extreme low temperature should be used. Materials near heat-emitting systems, e.g. machinery parts etc. should be able to withstand the local temperatures.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.9.4.6 The effect of concrete embedment, alkali exposure, bends, etc. shall be considered in determining the strength of FRP bars in accordance with [4.14.11].

4.10 Steel fibres

4.10.1 General

4.10.1.1 Steel fibres which are used in concrete or grout shall be suitable for the application intended and provide sufficient performance in concrete or grout.

4.11 Fibre reinforced polymer fibres

4.11.1 General

4.11.1.1 FRP fibres are produced by carbon, glass, basalt and aramid. The FRP fibres shall be tested and found suitable for application in concrete structures. It shall be documented that the fibre is durable in concrete structures exposed to the actual environmental conditions.

4.12 Embedded materials

4.12.1 General

4.12.1.1 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. Their properties shall be documented with respect to their intended application.

4.13 Other materials

4.13.1 Repair materials

4.13.1.1 The composition and properties of repair materials shall be such that the material fulfils 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. Repair materials may be concrete of similar characteristics to the parent material or pre-blended specialist materials.

4.13.1.2 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.

4.13.1.3 The extent of on-site qualification and QC testing of repair materials shall be specified in each case.

4.13.1.4 Pre-blended repair materials shall be delivered with a works certificate.

Guidance note:

The requirement for a works certificate may be waived if the materials are 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.13.2 Non-cementitious materials

4.13.2.1 The composition and properties of non-cementitious materials shall be determined so that each material fulfils its intended use. Special emphasis shall be given to ensure that such materials are as similar as possible to the adjacent material, particularly in the sense of elasticity and temperature dependent properties. Their properties shall be documented with respect to their intended application.

4.13.2.2 Epoxy coating for low cover or other applications relevant for maintaining design intent as well injection materials for crack filling and use in construction joints shall be delivered with a works certificate.

Guidance note:

The requirement for a works certificate may be waived if the materials are 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

4.13.3 Equivalent materials

4.13.3.1 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.

4.14 Testing of materials

4.14.1 Testing of freshly mixed concrete

4.14.1.1 Requirements to the testing of freshly mixed concrete are given in [7.4] and [7.6].

4.14.2 Testing of concrete in the structure

4.14.2.1 Requirements to the testing of concrete in structures are given in [7.6].

4.14.3 Grout for post tensioning tendons

4.14.3.1 The requirements for on-site QC testing of freshly mixed grout are given in[7.4] and [7.6].

4.14.4 Grout for underbase grouting and structural connections

4.14.4.1 The requirements for on-site QC testing of cement grout are given in [7.6].

4.14.4.2 The requirements for on-site QC testing of pre-blended grout are given in [7.6].

4.14.4.3 Pre-blended grout, both pre-packed and bulk/silo stored, intended for use in structural connections shall be delivered with a product or type approval certificate. Recommended prequalification testing of fresh and hardened grout to document material properties is outlined in App.H.

4.14.5 Reinforcement steel

4.14.5.1 Reinforcement steel shall be delivered with a works certificate. See [4.7.1.4].

4.14.5.2 Initial type testing shall be in accordance with [4.7.1.2].

4.14.6 Prestressing steel

4.14.6.1 Prestressing steel shall be delivered with a works certificate. See [4.8.1.2].

4.14.7 Mechanical splices for reinforcement

4.14.7.1 Mechanical splices shall be delivered with a product or type approval certificate. See [4.7.2.2]. The certificate shall document that the mechanical splices are suitable for their intended application and have the same safety as the spliced reinforcement bars.

4.14.8 Components for the prestressing system

4.14.8.1 Components for the prestressing system shall be delivered with a product certificate. See [4.8.2.4]. The certificate shall document that the components for the prestressing system are suitable for their intended application and have the same safety as the prestressing rods or tendons.

4.14.9 Welding procedures

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

4.14.10 Testing of repair materials

4.14.10.1 The repair materials shall be documented in accordance with relevant recognised international standard, i.e. ASTM, ACI, EN and ISO. The repair materials shall be suitable for use in offshore concrete structures and have comparable properties to the parent material under repair. The suitability of the repair material shall be documented.

4.14.11 Testing of fibre reinforced poluymer materials

4.14.11.1 The bars shall be delivered with a product or type approval certificate specifying the properties required, testing shall be accordance with the requirements of App.F.

SECTION 5 LOADS AND ANALYSES REQUIREMENTS

5.1 Requirements to design

5.1.1 General

5.1.1.1 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.

5.1.1.2 The functional requirements will affect the layout of the structure, thus influencing the loading scenarios that shall be considered in the design of the structure. The functional requirements shall be related to both the site-specific conditions and the requirements of the platform as a production facility, for production of hydrocarbons and other activities associated with operations of a field.

5.1.2 Site related functional requirements and environmental considerations

5.1.2.1 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.

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

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

5.1.2.4 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.

5.1.3 Facility operational requirements

5.1.3.1 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, for operation, inspection and condition monitoring, etc.
e) interface to topsides/plant
f) installations for supply boats and other vessels servicing the platform/installation.

5.1.3.2 All hazard scenarios that are associated with the operation/maloperation and function 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.

5.1.4 Structural requirements

5.1.4.1 Structures and structural members shall perform satisfactorily during all design conditions, with respect to structural strength, mooring, stability, ductility, durability, displacements, 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.

5.1.4.2 Ground supported structures located in seismically active areas shall be designed to have adequate strength to withstand the effects of an extreme level earthquake (ELE) as well as sufficient strength, ductility and energy dissipation capacity to remain stable during the rare motions of greater severity associated with abnormal level earthquake (ALE). The sufficiency of the structural strength, ductility and energy dissipation capacity shall be documented. Provisions of this standard do not ensure sufficient ductility that may be required for seismic loading. In this case, ductility shall be documented.

The seismic ULS design event is the ELE. The structure, foundation and secondary structural components shall withstand an ELE event with little or no damage. Safety, production and evacuation systems shall be fully functional during and after the ELE event. The structure shall be inspected after an ELE occurrence.

The seismic ALS design event is the ALE. The ALE is an intense earthquake of abnormal severity with a very low probability of occurring during the structure's design service life. The ALE may cause considerable damage to the structure. However, the structure shall be designed such that overall structural integrity is maintained to avoid structural collapse causing loss of life and/or major environmental damage.

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

a) has adequate robustness with small sensitivity to local damage
b) may be constructed in a controlled manner
c) provides simple stress paths that limit stress concentrations
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.

5.1.4.4 For additional requirements see Sec.6.

5.1.5 Materials requirements

5.1.5.1 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 materials are given in Sec.4.

5.1.5.2 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.4.

5.1.6 Execution requirements

5.1.6.1 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, Sec.7 and Sec.8.

5.1.7 Temporary phases requirements

5.1.7.1 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 DNVGL-ST-N001 *Marine operations and marine warranty*.

5.1.7.2 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. Adequate freeboard shall be provided. One-compartment damage stability should normally be provided except for short transient phases. The stability and freeboard shall be in accordance with DNVGL-RU-OU-0102 *Floating production, storage and loading units*.

5.1.7.3 Weight control required for floating structures and temporary 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. See DNVGL-ST-N001 for relevant weight control requirements.

5.1.7.4 No permanent cracks caused by yield in the reinforcement shall occur during temporarily load conditions. This means that the stress in the reinforcement shall be less than 0.9 f_{sk} for ULS combinations applying γ_F equal to 1.0 for all loads occurring during the temporarily phase. See also [6.2.2.5] and [6.15.3.10].

5.2 Design principles

5.2.1 General

5.2.1.1 The design shall be performed in accordance with the limit state design as detailed in DNVGL-OS-C101 Ch.2 Sec.1. The design shall provide adequate strength and tightness in all design situations such that the assumptions made are complied with. This may be achieved by at least the following:

- design of concrete structures shall be in accordance with this standard
- foundation design shall be in accordance with DNVGL-OS-C101 Ch.2 Sec.10 and DNVGL-RP-C212
- design of steel structures shall be in accordance with DNVGL-OS-C101 Ch.2 Sec.3, 4, 5 and 8
- possible interface between steel structure and concrete structure shall be included in the design
- design for load and load effects shall be in accordance with DNVGL-OS-C101 Ch.2 Sec.2. See also special requirements to concrete structures in this section
- design for accidental limit states shall be in accordance with DNVGL-OS-C101 Ch.2 Sec.6. See also identifications of hazards in this standard and Sec.6 for reinforced concrete design
- cathodic protection shall be designed in accordance with DNVGL-OS-C101 Ch.2 Sec.9
- stability of the structure afloat shall be calculated in accordance with DNVGL-RU-OU-0102 *Floating production, storage and loading units*.

5.2.2 Design loads

5.2.2.1 The characteristic values of loads shall be selected in accordance with DNVGL-OS-C101 Ch.2 Sec.2 and this standard.

5.2.2.2 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 DNVGL-OS-C101 Ch.2 Sec.1 and specifically for concrete in [5.4.1].

5.2.3 Design resistance

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

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

5.2.3.3 The design resistance shall be determined using this standard.

5.3 Load and load effects

5.3.1 General

5.3.1.1 The load and load effects shall be in accordance with DNVGL-OS-C101 Ch.2 Sec.2. The loads are generally classified as:

a) environmental, E
b) functional:

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

5.3.1.2 The loads shall include the corresponding external reaction. The level of the characteristic loads shall be chosen in accordance with the condition under investigation:

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

5.3.1.3 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 shall be verified.

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

5.3.1.5 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.

5.3.1.6 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.

5.3.1.7 For ground supported structures located in seismic active zones, a seismic hazard assessment shall be carried out as detailed in [3.1.3.5]. Seismic loads shall be specified in terms of a seismic design spectrum or a set of real or artificially simulated earthquake time histories. A minimum of four time histories shall be used to capture the randomness in seismic motions.

5.3.1.8 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 DNVGL-RP-C212.

5.3.2 Environmental loads

5.3.2.1 Wind, wave, tide and current are sources of environmental loads (E) on many structures located offshore. See App.A for more details. In addition, depending on location, earthquake or ice loads or both may be significant environmental loads.

5.3.2.2 ISO 19901-2:2004 provides detailed recommendation for estimating seismic loads for ELE event.

5.3.2.3 Earthquake induced hazards such as liquefaction, slope instability, faults, tsunamis, mud volcanoes and shock waves are out of the scope of this standard. Nevertheless, they shall be duly considered in the design if applicable.

5.3.2.4 The computation of ice loads is highly specialized and location dependent and is not covered in detail 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 DNVGL-OS-C101 Ch.2 Sec.2.

5.3.2.5 Wave loads 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 App.A.

5.3.2.6 Diffraction analysis

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 App.A.

5.3.2.7 Additional requirements for dynamic analysis under wave loads

In cases where the structure responds 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.

Ringing may control the 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.

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.

5.3.2.8 Model testing

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

5.3.2.9 Current load

Currents through the depth, including directionality, shall be combined with the design wave conditions. The characteristic current load shall be determined in accordance with DNVGL-OS-C101 Ch.2 Sec.2. For more details, see App.A.

If found necessary scour protection should be provided around the base of the structure. See DNVGL-OS-C101 Ch.2 Sec.10.

5.3.2.10 Wind Loads

Wind loads may be determined in accordance with DNVGL-OS-C101 Ch.2 Sec.2.

Wind forces on an offshore concrete structure will consist of two parts:

a) wind forces on topside structure
b) wind forces on concrete structure above sea level.

For more details, see App.A.

5.3.3 Functional loads

5.3.3.1 Functional loads are considered to be all loads except environmental loads and include both permanent and variable loads. The functional loads are defined in DNVGL-OS-C101 Ch.2 Sec.2.

5.3.3.2 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).

5.3.3.3 Variable functional loads (Q) originates from normal operations of the structure and varies 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 storage compartments, etc. (may also be considered as deformation loads)
— loads occurring during installation and drilling operations, etc.
— ordinary boat impact, rendering and mooring.

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

5.3.3.5 Certain loads, which are 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.

5.3.3.6 Potential imposed deformations are derived from sources that include:

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

See also [5.5.4.1].

5.3.4 Accidental loads

5.3.4.1 The accidental loads (A) are generally defined in DNVGL-OS-C101 Ch.2 Sec.2.

5.3.4.2 Primary sources of accidental loads include:

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

5.3.4.3 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.

5.3.4.4 Rare occurrences of abnormal environmental loads

This will include relevant abnormal environmental loads, such as wave, seismic, ice, etc. For rare occurrences of abnormal environmental loads the annual frequency of occurrence shall be 10^{-4}. For estimation of seismic loads for ALE an event see ISO 19901-2:2004.

5.3.4.5 Fires

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:

a) burning blowouts in wellhead area
b) fire related to releases from leaks in risers; manifolds, loading/unloading or process equipment, or storage tanks, including jet fire and fire ball scenarios
c) burning oil/gas on sea
d) fire in equipment or electrical installations
e) pool fires on deck or sea
f) 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.

5.3.4.6 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.

5.3.4.7 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 may 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 may 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 may 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 structure 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 structure should be considered to be between 10 m below LAT and 13 m above HAT.

5.3.4.8 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 may 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.

5.3.4.9 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.

5.3.4.10 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.

5.3.4.11 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 may be assumed to be statistically 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^{-3} 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

5.4 Load combinations and partial safety factors

5.4.1 Partial load factors, γ_f

5.4.1.1 The load factors are specified in DNVGL-OS-C101 Ch.2 Sec.1, *Design by LRFD method* and in Table 5-1 and Table 5-2.

5.4.1.2 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 in order to provide an equivalent level of safety. The equivalent safety shall be documented. Requirements to special evaluations are given in App.D.

5.4.1.3 When checking the serviceability limit state, SLS, the partial load factor γ_f shall be 1.0 for all loads.

5.4.1.4 When checking the fatigue failure limit state, FLS, the partial load factor γ_f shall be 1.0 for all loads.

5.4.1.5 In the ALS, the partial load factor shall be 1.0 for all loads.

5.4.1.6 For structures with steel reinforcement, the ultimate limit state, ULS, shall be checked for two load combinations, a) and b), with load factors in accordance with Table 5-1 (see DNVGL-OS-C101 Ch.2 Sec.1 Table 1).

Table 5-1 Recommended partial factors, γ_f, for loads for the ultimate limit state (ULS) Load combinations (from DNVGL-OS-C101) for structures with steel reinforcement

Combination of design loads	*Load categories*				
	G	*Q*	*E*	*D*	*P*
a)	1.3	1.3	0.7 [1)]	1.0	0.9/1.1 [2)]
b)	1.0	1.0	1.3 [1)]	1.0	0.9/1.1 [2)]

Load categories are:

G = permanent load
Q = variable functional load
E = environmental load
D = deformation load
P = prestressing load.

1) Factor may have to be amended for areas with other long term distribution functions than North Sea conditions.
2) The more conservative value of 0.9 and 1.1 shall be used as a load factor in the design.

For description of load categories, see DNVGL-OS-C101 Ch.2 Sec.2 and [5.4.1.8] through [5.4.1.13] below.

5.4.1.7 For structures with FRP reinforcement, the ultimate limit state, ULS, shall be checked for load combinations in accordance with Table 5-2. It shall be noted that design of structures reinforced by FRP, three new load combinations c), d) and e) are identified in addition to the load combinations in Table 5-1.

Table 5-2 Recommended partial factors, γ_f, for loads for the ultimate limit state (ULS) load combinations for structures with FRP reinforcement (including FRP fibres)

Combination of design loads	*Load categories*					
	G	Q_1	Q_2	*E*	*D*	*P*
a)	1.3	1.3	1.3	0.7 [1)]	1.0	0.9/1.2 [2)]
b)	1.0	1.0	1.0	1.3 [1)]	1.0	0.9/1.2 [2)]
c)	1.3	1.3			1.3	0.9/1.2 [2)]
d)	1.3		1.3		1.0	0.9/1.2 [2)]
e)	1.0		1.3		1.0	0.9/1.2 [2)]

Combination of design loads	Load categories					
	G	Q_1	Q_2	E	D	P

Load categories are:

G = permanent load

Q_1 = variable functional load of permanent character are live loads that the structure may be exposed to for its entire service life or a considerable part of it, e.g. load from prestressing, dead weight of the structure, weight of furniture, stored goods, etc.

Q_2 = variable functional load of variable nature are live loads that the structure may be exposed to only for limited durations much less than the service life, such as e.g. weight of occupants and (not permanently stored) vehicles.

E = environmental load

D = deformation load

P = prestressing load

1) Factor may have to be amended for areas with other long term distribution functions than North Sea conditions.

2) The more conservative value of 0.9 and 1.2 shall be used as a load factor in the design.

5.4.1.8 The loads shall be combined in the most unfavourable way, provided that the combination is physically possible and permitted in accordance with 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 is less than 10^{-4}.

5.4.1.9 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, a load factor of 1.2 should normally be used provided that the load effect is determined with normal accuracy. Where second order effects are important, a load factor of 1.3 shall be used.

5.4.1.10 A load factor of 1.0 shall be applied to the weight of soil included in the geotechnical calculations. If weight of soil is intended for ballasting purposes, it shall be treated as permanent load.

5.4.1.11 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 partial factor for action in the design shall be as per Table 5-1 or Table 5-2, whichever is applicable, for prestressing load (P).

5.4.1.12 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.

5.4.1.13 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.

5.4.2 Combinations of loads

5.4.2.1 DNVGL-OS-C101 Ch.2 Sec.2 Table 2 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.

5.4.2.2 For temporary phases, if 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 significant financial losses, a shorter return period than that given in DNVGL-OS-C101 Ch.2 Sec.2 Table 2 for environmental loads may be considered.

5.4.2.3 The return conditions to be considered should be related to the duration of the operation. As a general guidance, the criteria given in Table 5-3 may be applied:

Table 5-3 Environmental criteria

Duration of use	*Environmental criteria*
Up to 3 days	Specific weather window
3 days to 1 week	More than 1 year, seasonal
1 week to 1 month	10 years return, seasonal
1 month to 1 year	100 years return, seasonal
More than 1 year	100 years return, all year

5.5 Structural analysis

5.5.1 General

5.5.1.1 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.

5.5.1.2 Complex or unusual structural types may 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.

5.5.1.3 In order to ensure successful structural 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.

5.5.1.4 Interfaces between structural designers, topsides designer, hydrodynamic analysts, 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.

5.5.1.5 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 is 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 all components.

5.5.1.6 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.

5.5.1.7 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.

5.5.1.8 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.

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

5.5.1.10 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.

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

5.5.1.12 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.

5.5.1.13 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.

5.5.1.14 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.

5.5.1.15 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.

5.5.1.16 Computer spread-sheets are electronic methods of performing hand calculations and shall be subject to the same requirements. Where such spread-sheets 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 elements in the spread-sheet that will be used for the component being assessed.

5.5.1.17 Special forms of analysis for concrete structures, such as the strut and tie approach, may also be used, but shall conform to up to date, 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.

5.5.1.18 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.

5.5.1.19 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

5.5.1.20 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
- that the specified material properties have been used
- that sufficient and correct loads have been applied
- that suitable and justifiable boundary conditions have been simulated
- that an appropriate analysis type and methodology have been used for the analysis.

5.5.1.21 Verification of the results of an analysis will in general vary depending on the nature of that analysis. Typical output quantities that shall be checked, as appropriate, include the following:

- individual and summed reactions, to ensure that these balance the applied loads
- deformations of the structure, to verify that these are sensible and that they demonstrate compatibility between components
- natural periods and mode shapes, if appropriate
- load paths, bending moment diagrams, stress levels, etc., to check that these satisfy equilibrium requirements.

5.5.1.22 Successful execution of an analysis shall be recorded and pertinent parties informed of results and conclusions so that implications for the design process are formally recognized.

5.5.1.23 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:

- 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.
- Summed magnitude and direction of all loads.
- Pertinent results from the analysis and crosschecks to verify the accuracy of the simulation.
- Clear presentation of those results of the analysis that is required for further analysis, structural design or reassessment.

5.5.1.24 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 are determined.

— Section strains, used to determine crack widths and assess water tightness; stress occurrences, used to check the fatigue life of the structure.

5.5.2 Young's modulus to be used in load effect analyses

Concrete

5.5.2.1 In the calculation of strains and section forces, the relation between Young's modulus of concrete E_c and compressive cylinder strength f_{cck} may be taken as:

$$E_{cn} = 22\,000 \cdot (f_{cck}/10)^{0.3} \text{ MPa for } f_{cck} \leq 70 \text{ MPa}$$

$$E_{cn} = 4800 \cdot (f_{cck})^{0.5} \text{ MPa for } f_{cck} > 70 \text{ MPa}$$

if the factor is not determined by testing.

5.5.2.2 E_{cn} may be determined as the secant modulus (see Sec.4) by testing E-modulus in accordance with appropriate international standard. The strength f_{cck} is determined with the same cylinder samples. E_{cn} 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.

5.5.2.3 To consider loading of early age concrete the characteristic cylinder strength at the actual time of loading may be used.

5.5.2.4 The effect of cracking shall be considered in cases where structural displacements cause increased forces and moments, see [5.5.12].

5.5.2.5 Until the value of Young's modulus for lightweight aggregate concrete is confirmed by testing, the Young's modulus obtained in accordance with [5.5.2.1], shall be reduced by multiplying the value by a factor $(\rho/\rho_1)^2$ where ρ_1 = 2200 kg/m^3.

5.5.2.6 For impact type of loading or rapid oscillations the moduli of elasticity calculated in accordance with [5.5.2.1] and [5.5.2.2] may be increased by up to 15%, dependent on strain rate.

5.5.2.7 The Young's modulus predicted in [5.5.2.1] 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 Young's modulus may be taken as 90 per cent of E_{cn} given in [5.5.2.1]. For temperatures above 200°C the concrete strain properties, including creep and thermal strain, shall be determined specially.

Steel reinforcement

5.5.2.8 The characteristic Young's modulus of non-prestressed reinforcement may be taken as

$$E_{sk} = 200\,000 \text{ MPa.}$$

5.5.2.9 At high temperatures of short duration (fire) the Young's modulus of steel may be taken in accordance with [5.5.2.8] 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.

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

FRP reinforcement

5.5.2.11 The characteristic value of stiffness for FRP reinforcement shall be estimated by the sample mean of stiffness data from tests on a representative sample of specimens. It shall be reported in the product or type approval certificate for relevant temperatures.

5.5.2.12 At high temperatures of short duration (fire), the Young's modulus of FRP shall be documented.

5.5.3 Effects of temperature, shrinkage, creep and relaxation

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

5.5.3.2 The linear coefficient of thermal expansion for both normal weight 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 light weight aggregate 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 the linear coefficient of thermal expansion.

For concrete exposed to low temperatures the linear coefficient of thermal expansion shall be determined by relevant tests of the material.

5.5.3.3 Values of concrete creep and shrinkage shall be chosen on the basis of the conditions surroundings of the structure (temperature, relative humidity, etc.), sectional dimensions, concrete mixture and age.

5.5.3.4 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

$$\varepsilon_{cc} = \varphi \varepsilon_c = \frac{\varphi \sigma_c}{E_{ck}}$$

where:

φ = the creep coefficient

σ = the concrete stress due to long-term loading.

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

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

5.5.3.7 Until the value of creep coefficient for lightweight aggregate concrete is confirmed by testing the creep coefficient φ may be assumed equal to the value of normal weight concrete multiplied by a factor $(\rho/\rho_1)^2$ for ρ > 1800 kg/m^3. For lightweight concrete with ρ < 1500 kg/m^3, a factor 1.3 $(\rho/\rho_1)^2$ may be used. For intermediate values of ρ, linear interpolation may be applied where ρ_1 = 2200 kg/m^3

5.5.3.8 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 5-1 may be used. Normally, testing is expected to be based on at least 10 000 hours loading.

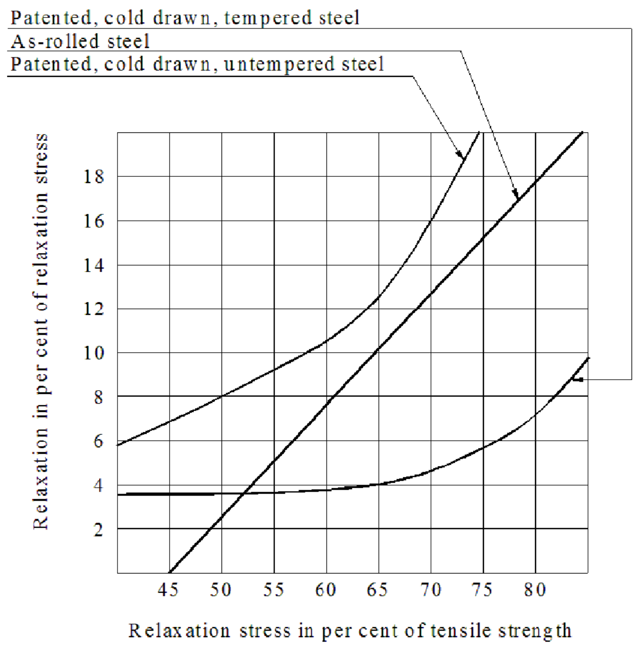


Figure 5-1 Long-term relaxation in prestressing steel

5.5.3.9 If the steel experiences a temperature, T, higher than T = 20°C for a long period of time, a quantity $k_1(T-T_1)$ shall be added to the relaxation in percentage 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.

5.5.3.10 The effect of relaxation in prestressed FRP reinforcement shall be calculated in proportion to the time period over which the relaxation occurs. It shall be reported in the product or type approval certificate for relevant temperatures.

5.5.4 Special load effects

5.5.4.1 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 [5.3.3].

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 in which sufficient rotational capacity exists. Verification of sufficient rotational capacity may in most cases be based on simplified considerations.

5.5.4.2 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.

In practice, effects due to imposed deformations may be calculated using a linear elastic model, and a constant Young's modulus throughout the structure. Possible stiffness reductions 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.

5.5.4.3 Creep effects shall be considered where relevant. An accurate calculated assessment of creep in shell structures is only be obtained by computer calculations using non-linear finite element programs. [6.3.8] outlines procedures to roughly estimate the effects of creep.

5.5.4.4 The effect of water pressure in the concrete shall be fully considered when relevant.

5.5.4.5 The effect of hydrostatic forces acting on the faces of cracks shall 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. See also [6.3.9].

5.5.4.6 For structures designed with an intended underpressure, relative to external pressure, a design condition where the intended underpressure is lost shall be evaluated. An evaluation of the consequences of underpressure loss shall be performed. It shall be documented that during and after the loss scenario, the structure still meets its performance requirements with respect to structural integrity, serviceability, oil containment, stability, etc. Relevant limit states shall be considered in this demonstration.

5.5.4.7 The long-term effect of water absorption shall be considered in the estimation of concrete weights in particular for floating structures. This also applies for concrete and grout, with and without fibres.

5.5.5 Physical representation

5.5.5.1 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 App.B.

5.5.6 Loads

5.5.6.1 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 variation of such loads. For more details see App.B.

5.5.7 Mass simulation

5.5.7.1 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 App.B.

5.5.8 Damping

5.5.8.1 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.

5.5.9 Linear elastic static analysis

5.5.9.1 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 App.C.

5.5.10 Dynamic analysis

5.5.10.1 Fixed structures with natural periods of the global structure greater than 2.5s may be susceptible to dynamic response due to wave load during in-service conditions, at least for fatigue assessment. Structures in shallow water subject to large waves 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 App.C.

5.5.11 Pseudo-static analysis

5.5.11.1 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 App.C.

5.5.12 Non-linear analysis

5.5.12.1 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 cracking regions 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 abnormal level earthquake.
— For slender members in compression, where deflection effects are significant.

For further details, see App.C.

5.5.13 Probabilistic analysis

5.5.13.1 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.

5.5.13.2 Such methods typically linearize load effects. This restricts 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.

5.5.13.3 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.

5.5.14 Reliability analysis

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

5.5.15 Analyses requirements

5.5.15.1 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.

5.5.15.2 Table 5-4 gives general guidance as to the type of analysis that shall be adopted for each design condition for the structure. Further details are provided from [5.5.16] to [5.5.23].

Table 5-4 Appropriate types of analysis

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

5.5.16 Analysis of construction stages

5.5.16.1 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.

5.5.16.2 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 loads for temporary support, such as crane footings, shall be included in the analysis.

5.5.16.3 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.

5.5.17 Transportation analysis

5.5.17.1 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.

5.5.17.2 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 excitated by this global motion. If dynamic effects are deemed important, they shall be incorporated in accordance with [5.5.10]. The analysis of the tow shall be in accordance with the DNVGL-ST-N001 *Marine operations and marine warranty*.

5.5.17.3 Fatigue damage may 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 [5.5.20].

5.5.17.4 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.

5.5.18 Installation and deck mating analysis

5.5.18.1 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.

5.5.18.2 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 un-grouted state.

5.5.19 In-service strength and serviceability analyses

5.5.19.1 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 abnormal wave effects using ALS load factors, unless it is conclusively demonstrated that this limit state is always less onerous than the corresponding ULS condition.

5.5.19.2 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.

5.5.19.3 It is generally acceptable to base all strength analysis of an in-service concrete platform on deterministic analysis, predicting response to specific 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 is obtained due to larger dynamic effects at smaller wave periods.

5.5.20 Fatigue analysis

5.5.20.1 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.

5.5.20.2 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 through the lifetime of the structure shall be analysed separately and fatigue damage shall be accumulated over each phase.

5.5.20.3 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.

5.5.20.4 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.

5.5.20.5 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.

5.5.21 Seismic analysis

5.5.21.1 ISO 19901-2 provides recommendations for the seismic analysis of offshore concrete structures for both ELE and ALE earthquakes.

5.5.22 Accidental and overload analyses

5.5.22.1 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.

5.5.22.2 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 necessary if the structure is expected to deform substantially under the accidental loads. Appropriate boundary conditions shall be provided far enough away from the damaged region for inaccuracies to be minimized.

5.5.22.3 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.

5.5.22.4 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.

5.5.22.5 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.

5.5.23 Platform removal/reuse

5.5.23.1 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.

5.5.23.2 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.

5.5.23.3 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.

5.6 Topside interface design

5.6.1 Introduction

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

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

5.6.1.3 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.

5.6.2 Basis for design

5.6.2.1 As part of establishing and maintaining adequate handling of topside/substructure interface throughout the design process, all necessary design information shall be defined. Plans shall 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.

5.6.2.2 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.

5.6.2.3 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.

5.6.2.4 The documentation to be provided as basis for proper interface design shall 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.

5.6.3 Deck/shaft structural connection

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

5.6.3.2 The physical interface is very often present between a steel module support frame 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 offshore concrete structure shafts. The area between the tubular bearings is typically grouted before activation of prestressed anchor bolts.

5.6.3.3 The design of intersection between the module support frame, grout and top of shaft(s) shall 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 shall also be accounted for.

5.6.3.4 If non-rigid topside to substructure connection is selected, such as an array of elastomeric bearings, consideration shall 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.

5.6.3.5 Depending on the connection selected, the detailing and layout shall allow for necessary inspection and maintenance. Special consideration shall be given to gaining access to fatigue prone details and, if access is not possible, a suitably large design fatigue life shall be selected. Any materials used shall 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) shall be clearly communicated.

5.6.4 Topsides - substructures mating

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

5.6.4.2 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.

5.6.5 Transportation

5.6.5.1 The dynamic motions during the towage of fixed concrete installations are usually small. Accelerations and tilting angles in the intact and damaged condition shall be accurately defined. Consequences for design of topside, substructure and their connections shall be addressed.

5.7 Concrete barges

5.7.1 General

5.7.1.1 Concrete barges classed by DNV GL shall be designed and constructed in accordance with DNV GL rules for classification of ships DNVGL-RU-SHIP Pt.5 Ch.11 Sec.6 *Concrete Hull*.

SECTION 6 DETAILED DESIGN OF OFFSHORE CONCRETE STRUCTURES

6.1 General

6.1.1 Introduction

6.1.1.1 Other design standards may be used as an alternative for detailed design of offshore concrete structures due to local preferences. An opening for this is given within this standard provided the requirements to the detailed standard given in App.D are sufficiently covered. The level of safety shall be as required by DNV GL standard. The compliance with this requirement shall be documented.

6.1.2 Material

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

6.1.2.2 For definition of normal strength concrete, high strength concrete and lightweight aggregate concrete see [4.3.1].

6.1.3 Load effects

6.1.3.1 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.

6.1.3.2 In slender structures the effect of the structural displacements shall be accounted for in the calculation of forces and moments (2^{nd} order effects).

6.1.3.3 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 [6.15.8]. 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.

6.1.3.4 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.

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

6.1.3.6 Calculation of load effects in shear walls and shells may be based on assumptions other than the theory of elasticity if sufficient knowledge on the stress conditions of the actual structure is available based on tests or nonlinear calculations. Force models as indicated in [6.9] *Regions with discontinuity in geometry and loads* may be used if relevant models may be established for the structure in question.

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

6.1.3.8 In structural analysis of FRP reinforced structures, non-linear redistribution of internal force resultants is not accepted due to the linear stress-strain curve of FRP reinforcement.

6.1.3.9 For FRP reinforced structures, force models as indicated in [6.9] *Regions with discontinuity in geometry and loads* shall be applied with care allowing no redistribution in the FRP reinforcement.

6.1.4 Effective flange width

6.1.4.1 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.

6.1.4.2 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.

6.1.4.3 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.

6.1.4.4 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.

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

6.1.5 Composite structures

6.1.5.1 Composite structures are structures where concrete and structural steel act together. Steel and concrete members shall be designed in accordance with DNVGL-OS-C101 and this standard, respectively, or other International applicable standards. The same safety level shall be achieved as in this standard. The general requirements of this standard still apply.

6.1.5.2 A composite structure can be assumed to perform as a monolithic unit if the shear forces between members of the composite can be transferred by reinforcement, shear keys, or by other devices. The force in the shear connectors shall be calculated in accordance with an International recognized standard for composite structures.

6.1.5.3 In the ultimate and fatigue limit states, forces shall be calculated considering the characteristics of the connection, i.e. fully or partially bonded, between members of the composite.

6.1.5.4 The capacity of the individual structural members of the composite structure shall be also checked for the loads applied on the members before they are acting as a unit. In the serviceability limit state, it shall be considered whether the respective loads are applied before or after the members are acting together.

6.1.5.5 Composite member deflection may be estimated assuming a cracked concrete section to calculate the section moment of inertia. The height of the concrete compression zone shall be calculated based on the acting loads.

Composite structures with studs

6.1.5.6 Material factor for studs may be assumed equal to the material factor for steel reinforcement, Table 6-1.

6.1.5.7 Studs may be considered to contribute to the shear capacity of the concrete component provided that they extend through the concrete core and meet the requirements for transverse shear reinforcement stated in [6.6]. Contribution from studs to the shear capacity of the concrete component may be calculated according to [6.6].

6.1.5.8 Studs shall be designed for the combination of shear stresses, caused by the shear transverse force in the interface between concrete and steel, and the normal stresses, in case studs are assumed to contribute to the shear capacity of the section.

6.1.5.9 Studs shall not crush the concrete in their vicinity.

Guidance note:

This is ensured by limiting the shear stresses in the studs.

$$\tau_s \leq \frac{0.29 \times \alpha \times D^2 (f_{cck} E_{cn})^{0.5}}{\gamma_s \times 0.25 \times \pi \times D^2}$$

where:

D = diameter of studs [mm]

E_{cn} = Young's modulus of concrete

f_{cck} = characteristic cylinder compression strength of concrete [MPa]

α = 0.2 (h_s/D + 1) ≤ 1, where h_s is the stud height [mm]

γ_s = material factor for steel studs

τ_s = shear stress in the studs [MPa].

Studs should not be placed at a distance longer than 22 × t_h × $(235/f_{yk-p})^{0.5}$ at the steel plate in compression, in order to avoid plate buckling, where f_{yk-p} is the characteristic yield stress of the plate.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.1.5.10 S-N curves used for the fatigue limit state check of steel members and studs shall be documented considering the influence of the connection between studs and steel members, e.g. type of welding.

6.1.6 Prestressed structures with unbonded tendons

6.1.6.1 Un-bonded tendons for prestressed structures may be used provided that corrosion protection is adequately documented and a risk assessment of accidental situations that may result in the sudden failure of the un-bonded tendon is carried out.

6.1.6.2 The risk assessment of accidental situations shall include the evaluation of the consequences of the failure of the tendon itself, i.e. risk of hitting people, structure, equipment, etc. by the sudden failed tendons, and the potential collapse of the structure due to the loss of prestressing force.

6.1.6.3 Design shall account for the effects of the use of un-bonded tendons on the structural performance: crack width distribution, development of forces in the tendons, etc.

6.1.7 Yield line theory

6.1.7.1 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 is governed by a ductile mode of failure (structural detail has sufficient capacity in shear and moment to accommodate the required rotation)
- Second order effects are 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 shall be documented.

6.1.7.2 Redistribution of shear and moment caused by presumed yielding of FRP is not accepted.

6.1.8 Structures reinforced with fibre reinforced polymer

6.1.8.1 The capacity and service behaviour of prestressed FRP systems may be handled in a similar way as for prestressed steel systems, i.e. by applying a normal compression force and a moment in case the prestressing is applied eccentric in the section. This applies both for flexural capacity predictions, shear strength predictions, deflection calculation and crack width calculations.

6.1.8.2 Due consideration shall be given to the consequences of the differences between Young's modulus of FRP and steel on the anchorage, shrinkage and creep losses.

6.1.8.3 In calculation of moment capacity of FRP pre-stressed members, the strain/stress change in the FRP reinforcement caused by external loading shall be included in the internal force and moment calculation, and the stress in the FRP reinforcement shall not exceed the permissible stress according to the load combinations specified in [5.4].

6.1.8.4 For pre-tensioned systems, the FRP bars may be pre-stressed to the required level in accordance with [6.15.9]. After hardening of the concrete and the development of sufficient bond strength, the FRP reinforcement is cut in the normal manner and the pre-tensioning is transferred to the concrete member.

6.1.8.5 The post-tension systems shall be grouted as otherwise required for post-tensioning using steel tendons. The ducts shall be of a non-corrosive material and suitable for transferring the forces between the FRP reinforcement and the surrounding concrete.

6.1.8.6 For post-tension systems, the tensioning system gripping methods may damage the FRP reinforcement. Generally, the tensioning stress level is relatively low compared to the short term strength of the FRP reinforcement. Post-tensioning system shall be proven. The post-tensioning anchorage system shall be documented for the post-tensioning level to be applied and shall be made from non-corrosive material if exposed to a corrosive environment.

6.2 Design principles

6.2.1 General

6.2.1.1 Design in compliance with this standard is based either on calculations or on testing, or a combination of these.

6.2.2 Limit states

6.2.2.1 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).

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

6.2.2.3 In FLS, it shall be demonstrated that the structure sustains 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.

6.2.2.4 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 should include:

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

6.2.2.5 In order to prevent permanent strain in the reinforcement, stresses in the reinforcement of structural elements exposed to marine environment shall not exceed $0.9f_y$ for all possible characteristic ULS load combinations, i.e. with γ_f = 1.0, during its entire design life. In these checks a material factor equal to 1.0 may be used.

6.2.2.6 In addition to [6.2.2.5], stresses in the reinforcement of structural members which are relied upon for oil containment shall not exceed $0.9f_y$ for: 1) all possible characteristic ALS load combinations, i.e. with γ_f = 1.0, 2) all possible characteristic ULS load combinations that may occur on the damaged structure until the oil is safely removed. For the latter evaluation, structural analyses shall be carried out on the damaged structure.

6.2.2.7 Structural members which are relied upon for oil containment shall satisfy liquid tightness requirements as specified in [6.15.6].

In case of occurrence of an ALS event, tightness requirements post-ALS shall be satisfied as per [6.15.6] until all the oil is safely removed. For the post-ALS evaluation, structural analysis shall be carried out on the damaged structure. See also [6.14.1.5] for further requirements post-ALS events.

6.2.3 Characteristic values for material strength

6.2.3.1 The characteristic strength of materials shall be determined in accordance with design standards and recognized standards for material testing (ASTM, ACI, EN, ISO).

6.2.3.2 The in-situ strength, f_{cn}, of concrete, grout, fibre reinforced concrete and fibre reinforced grout may be determined from the characteristic compressive strength, f_{cck}, see [4.3] to [4.6].

6.2.3.3 Estimation of a characteristic value with confidence implies that instead of using an unbiased best estimate of the characteristic value, a conservative estimate of the characteristic value is used such that the probability of the true value being more favourable than the conservative estimate is at least equal to the specified confidence.

6.2.3.4 For definition of characteristic values used to represent soil properties, see DNVGL-RP-C212.

6.2.3.5 When underestimation of the resistance of a member may result in a non-conservative design, the characteristic strength shall be a high strength value. The suggested high characteristic strength value is the 95th percentile estimated from tests. The above situation may be found, for example, in the design of weak links.

6.2.3.6 When evaluating the actions imposed by the weak elements on adjoining members, in addition to considering a high characteristic strength, a material factor of 1.0 shall be used.

6.2.4 Partial safety factors for materials

6.2.4.1 The partial factors for the materials, γ_m, in reinforced concrete structures (concrete, steel and FRP reinforcement, grout, fibre reinforced concrete and fibre reinforced grout) shall be chosen in accordance with this standard and for the limit state considered. In addition, material factors for FRP reinforcement are dependent on the duration of the load under consideration.

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

6.2.4.3 Foundation design shall be performed with soil material factors in accordance with DNVGL-OS-C101.

6.2.5 Design by testing

6.2.5.1 If the loads acting on a structure, or the resistance of materials or structural members cannot be determined with reasonable accuracy, model tests should be carried out. Reference is made to [6.16].

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

6.2.5.3 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.

6.3 Basis for design by calculation

6.3.1 Design material strength

6.3.1.1 The material coefficients, γ_m, take into account the uncertainties in material strength and cross-sectional dimensions among others. The material coefficients are determined without accounting for reduction of capacity caused by corrosion or mechanical deterioration.

6.3.1.2 The material coefficients, γ_m, for concrete and steel reinforcement are given in Table 6-1.

Table 6-1 Material coefficients for concrete and reinforcement

Material	*Symbol*	*Ultimate ULS*	*Fatigue FLS*	*Accidental ALS*	*Serviceability SLS*
Reinforced concrete/grout[3] (steel)	γ_c	1.50[1] (1.35)[2]	1.50[1] (1.35)[2]	1.30[1] (1.20)[2]	1.00
Steel reinforcement	γ_s	1.25[1] (1.15)[2]	1.10[1] (1.00)[2]	1.10[1] (1.00)[2]	1.00
Fibre reinforced concrete/grout	γ_c	1.50[1] (1.35)[2]	1.50[1] (1.35)[2]	1.30[1] (1.20)[2]	1.00
Plain concrete/grout,	γ_c	1.80	1.80	1.50	1.00

1) Design with these coefficients allows for tolerances in accordance with [6.3.6] or alternatively on cross-sectional dimensions and placing of reinforcements that do not reduce calculated resistance by more than 10%. If specified tolerances are in excess of those given in [6.3.6] or the specified tolerances lead to greater reductions in calculated resistance, the excess tolerances or the reduction in excess of 10% shall be accounted for in resistance calculations. Alternatively, material coefficients may be taken according to those given under note 2.

2) When the design is based on dimensional data that include specified tolerances at their most unfavourable limits, structural imperfections, placement tolerances as to positioning of reinforcement, then these material coefficients may be used. When these coefficients are used, any geometric deviations from the approved-for-construction drawings shall be evaluated and considered in relation to the tolerances used in the design calculations.

3) Material factors for reinforced grout may be used in design where the grout itself is reinforced by steel reinforcement or where it may be demonstrated that steel reinforcement or anchor bolts in the surrounding structure contribute to reinforce the grout (see [6.20]).

6.3.1.3 The in-situ compression strength, f_{cn}, and tensile strength, f_{tn}, of normal weight concrete, grout, fibre reinforced concrete and fibre reinforced grout shall be determined according to [4.3] to [4.6].

6.3.1.4 If the design is carried out by testing, the requirements given in [6.16.5] shall apply.

6.3.1.5 When underestimation of concrete design strength may lead to non-conservative design, a special appraisal of the material coefficients and the nominal value of the in-situ strength shall be performed. See also [6.2.3.7] and [6.2.3.8].

6.3.1.6 For reinforcement consisting of FRP bars, consistent sets of characteristic material parameters and material factors for each limit state, which have been determined by a formal qualification process according to DNVGL-SE-0160, shall be used for design. Material factors for strength and stiffness for the different limit states shall be reported in the product or type approval certificate.

6.3.1.7 For FRP reinforced structures, the ultimate limit state shall be checked for the appropriate load combinations according to Sec.5 using a material factor for strength that reflects the duration of the extreme load in each load combination as well as effects of embedment and alkali exposure. The effect of temperature is covered by the temperature conversion factors mentioned in [4.9.4.5].

6.3.1.8 The load durations considered in design for FRP reinforced structures shall not be less than those specified in Table 6-2 for the applicable limit states according to Table 5-2.

The material coefficients, γ_m, for FRP reinforcement are given in Table 6-2.

Table 6-2 Material coefficients for FRP reinforcement (including FRP fibers)

Load combination type	*Duration*	*Load combination according to Table 5-2*	*Material factor[3] for strength*
I: Permanent load + live loads of permanent character[1]	50 years	c	γ_{FI}
II: I + extreme value of live loads of variable character[2] (e.g. weight of occupants)	1 year	d, e	γ_{FII}
III: II + extreme value of environmental load (wind, waves, current)	1 week	a	γ_{FIII}

1) Live loads of permanent character are live loads that the structure may be exposed to for its entire service life or a considerable part of it, e.g., weight of furniture, stored goods, etc.

2) Live loads of variable character are live loads that the structure may be exposed to only for limited durations much less than the service life, such as e.g. weight of occupants and (not permanently stored) vehicles.

3) Values for γ_{FI}, γ_{FII} and γ_{FIII} shall be calculated as described in [6.3.1.13].

Temperature loads may be either type II or III depending on duration of the temperature load.

6.3.1.9 For fatigue limit state, a material factor $\gamma_{F,SSA}$, which accounts for the duration of the loading, shall be used. The load duration used in the damage accumulation shall not be taken less than 5 years in each stress block.

6.3.1.10 For ALS, a material factor for strength, γ_{FA}, taking account of the duration of the relevant accident scenarios shall be used for FRP reinforcement with due consideration of the consequences of the accident and the duration of these consequences. The duration should in general not be taken less than 24 hours, see [6.3.1.13].

For SLS, a material factor for strength, γ_{FS}, taking account of the design life of the structure shall be used for FRP reinforcement, see [6.3.1.13].

6.3.1.11 Design values for the concrete/grout are:

$$E_{cd} = \alpha_c E_{cn}/\gamma_c$$

$$f_{cd} = \alpha_c f_{cn}/\gamma_c$$

$$f_{td} = \alpha_t f_{tn}/\gamma_c.$$

where:

E_{cd} = design value of Young's modulus used in the stress-strain curve

E_{cn} = normalized value of Young's modulus used in the stress-strain curve, reference is made to Sec.4.

f_{cd} = design compressive strength

f_{cn} = normalised compressive strength, see [6.3.1.3]

f_{td} = design strength in uni-axial tension

f_{tn} = normalised tensile strength, see [6.3.1.3]

γ_c = material factor (Table 6-1)

α_c = equal to 0.85 is applicable to offshore concrete structures in ULS, SLS and ALS calculations. For FLS calculations, it shall be equal to 1.0

α_t = equal to 1.0 is applicable to offshore concrete structures in ULS, SLS, FLS and ALS calculations.

6.3.1.12 Design values for the steel reinforcement are:

E_{sd} = E_{sk}/γ_s

f_{sd} = f_{sk}/γ_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

γ_s = steel reinforcement material factor (Table 6-1).

6.3.1.13 Design values for FRP bar reinforcement are:

E_{Fd} = E_F/γ_{FE}

f_{Fd} = f_F/γ_m.

where:

γ_{FE} = material factor for Young's modulus, E_F, which accounts for long term creep effects in the bars.

γ_m = material factor for strength of FRP reinforcement bars taking into account the duration of the loading, service temperature as well as manufacturing and placement considerations. For implementation of $\gamma_m$in ULS, ALS and SLS see below:

γ_m for FRP bars in the ultimate limit state (ULS)

γ_m shall be implemented in design in ULS as γ_{FI}, γ_{FII} or γ_{FIII} depending on the load combination type, specified in Table 6-2, under consideration. It is a function of:

— γ_F, a material factor to account for statistical variation in the material strength, potential placement inaccuracy in the section due to the physical characteristics of the bars and the level of control implemented during manufacturing, and

— $\eta_{F,\,TTR}$, derived from the characteristic time to rupture curve for the load durations for the different load combination types.

$$\gamma_m = \gamma_F \cdot \eta_T \cdot \eta_{F,\,TTR}$$

where:

γ_F = 1.25, for certified bar products meeting all manufacturing QA/QC requirements specified in App.G, produced under an established certification scheme.

η_T = Service temperature conversion factor. See [4.9.4.5].

$\eta_{F,\,TTR}$ = $(f_F/f_{F,\,TTR(i)})$

where:

f_F = characteristic short term tensile strength (force per area) of FRP bar

$f_{F,\,TTR(i)}$ = characteristic tensile strength (force per area) in FRP bar until failure at considered load duration i, to be documented through extrapolation of TTR test data

i = I (50 years), II (1 year) or III (1 week) corresponding to the load durations as per Table 6-2.

Guidance note:

Certified bar products meeting all manufacturing QA/QC requirements specified in App.G during initial establishment period of the certification scheme.

A thorough evaluation of statistical variation in the material strength, potential placement inaccuracy in the section due to the physical characteristics of the bars and the level of control implemented during manufacturing needs to be carried out to establish a material factor for this case. As a guide the material factor should not be chosen less than 1.4.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

γ_m for FRP bars in the accidental limit state (ALS)

γ_m shall be implemented in design in ALS as γ_{FA}, it is a function of:

- γ_F, a material factor to account for statistical variation in the material strength, potential placement inaccuracy in the section due to the physical characteristics of the bars and the level of control implemented during manufacturing, and
- $\eta_{F,\ TTR}$, derived from the characteristic time to rupture curve for the expected accidental load duration and associated consequences.

$$\gamma_m = \gamma_F \cdot \eta_T \cdot \eta_{F,\ TTR}$$

where:

γ_F = 1.2

η_T = service temperature conversion factor. See [4.9.4.5].

$\eta_{F,\ TTR}$ = $(f_F/f_{F,\ TTR(i)})$.

where:

f_F = characteristic short term tensile strength (force per area) of FRP bar

$f_{F,\ TTR(i)}$ = characteristic tensile strength (force per area) in FRP bar until failure at considered load duration i, to be documented through extrapolation of TTR test data

i = expected duration of the accidental scenario and consequences under consideration. Shall not be taken to be less than 24 hours.

γ_m for FRP bars in the serviceability limit state (SLS)

γ_m shall be implemented in design in SLS as γ_{FS}, it is a function of:

- γ_F, a material factor to account for statistical variation in the material strength, potential placement inaccuracy in the section due to the physical characteristics of the bars and the level of control implemented during manufacturing, and
- $\eta_{F,\ TTR}$, derived from the characteristic time to rupture curve for the load duration relating to the design life of the structure.

$$\gamma_m = \gamma_F \cdot \eta_T \cdot \eta_{F,\ TTR}$$

where:

γ_F = 1.2

η_T	=	service temperature conversion factor. See [4.9.4.5]
$\eta_{F,\,TTR}$	=	$(f_F/f_{F,\,TTR(i)})$

where:

f_F	=	characteristic short term tensile strength (force per area) of FRP bar
$f_{F,\,TTR(i)}$	=	characteristic tensile strength (force per area) in FRP bar until failure at considered load duration *i*, to be documented through extrapolation of TTR test data
i	=	duration corresponding to the design life of the structure. Shall not be taken to be less than 50 years, see [2.2.1.7] and [6.2.1.8].

6.3.2 Stress strain curve for concrete

6.3.2.1 Concrete stress-strain relationship in compression shall 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 6-1 may be used.

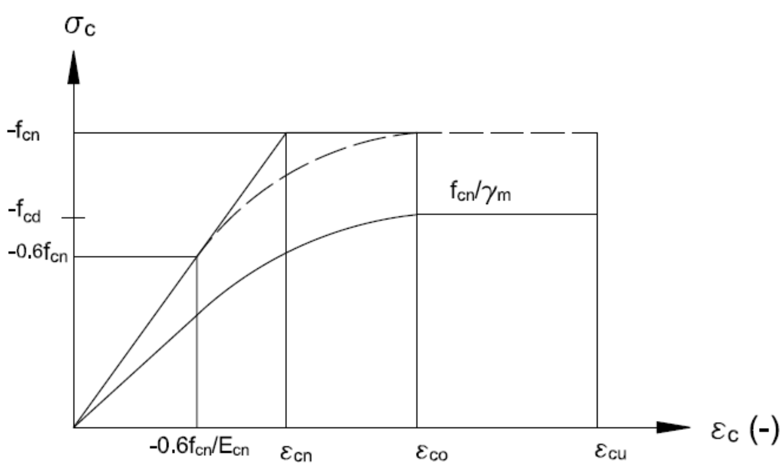


Figure 6-1 General stress-strain diagram for calculation of resistance of normal weight concrete in compression

Note: Compression is defined as negative and hence the values of ε and σ are negative for concrete subject to compression.

For

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

Then

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

For

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

Then

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

For

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

Then

$$\sigma_c = E_{cn}\varepsilon_c$$

Where:

$$\varepsilon_{cu} = -3.5 \times 10^{-3}, \text{ } f_{cck} < 50\text{MPa}$$

$$\varepsilon_{cu} = -\left(2.6 + 35\left(\frac{90 - f_{cck}}{100}\right)^4\right) \times 10^{-3}, \text{ } 90\text{MPa} > f_{cck} > 50\text{MPa}$$

And

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

Where

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

$$\varepsilon_{co} = \varepsilon_1 - k_e f_{cn}$$

where:

ε_1 = - 1.9 ‰ and k_e = 0.004 ‰/MPa.

Other parameters are defined in Sec.4.

Concrete having tensile strain shall be assumed to have no stress, unless otherwise is specified.

6.3.2.2 For concrete grades > C65 and for all lightweight aggregate concretes, the values of E_{cn} and ε_{co} shall be determined by testing of the type of concrete in question. Concrete having tensile strain shall be assumed to have no stress, unless otherwise is specified.

6.3.2.3 For fibre reinforced concrete of all grades, the values of E_{cn} and ε_{co} shall be determined by testing of the type of fibre reinforced concrete in question. Concrete having tensile strain shall be assumed to have no stress, unless otherwise is specified.

6.3.2.4 For normal weight concrete of grades between C35 and C45, the following simplified stress-strain diagram may be used.

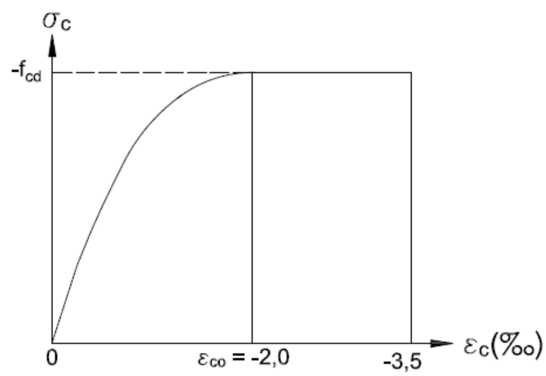


Figure 6-2 Simplified stress-strain diagram for normal weight concrete of grades between C35 and C45 subjected to compression

$$\sigma_c = -f_{cd} \frac{\varepsilon_c}{\varepsilon_{co}} \left(2 - \frac{\varepsilon_c}{\varepsilon_{co}} \right)$$

ε_{co} = - 2‰ is strain at the point of maximum stress.

Guidance note:

For lightweight aggregate concrete of grades between LC30 and LC35, a simplified bilinear stress-strain diagram may be applied for calculation of capacities.

The maximum strain limit for lightweight aggregate concrete in compression is:

$$\varepsilon_{cu} = \varepsilon_1 \left(0.3 + \frac{0.7\rho}{\rho_1} \right)$$

where ε_1 = -3.5‰, ρ_1 = 2200 kg/m^3 and ρ = density of lightweight aggregate concrete.

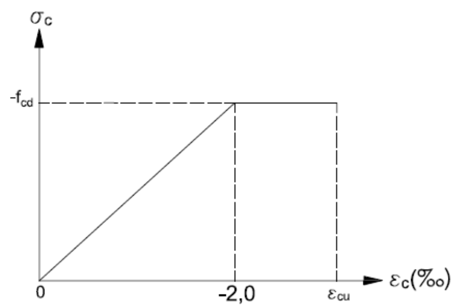


Figure 6-3 Simplified stress-strain diagram for lightweight aggregate concrete of grades between LC30 and LC35

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.3.2.5 Prior to using lightweight aggregate concrete in a structure or barge, the stress strain relationship till failure shall be documented.

6.3.2.6 For calculation of capacities for axial forces and bending moments, different stress distributions from those given herein ([6.3.2.1], [6.3.2.4] and [6.3.2.5]) may be applied as long as they do not result in a higher sectional capacity.

6.3.3 Stress strain curve for structural grout and fibre reinforced grout

6.3.3.1 Stress-strain relationship, in compression, for structural grout with characteristic cylinder strength less than 65 MPa may be taken in accordance to [6.3.2.1].

6.3.3.2 For structural grout with characteristic cylinder strength larger than 65 MPa and for fibre reinforced grout of all grades, the values of E_{cn} and ε_{co} shall be determined by testing. Grout having tensile strain shall be assumed to have no stress, unless otherwise is specified.

6.3.4 Steel reinforcement stress – strain curves

6.3.4.1 For steel reinforcement, a relationship between stress and strain which is representative for the type in question shall be used.

The stress-strain diagram for design is found by dividing the characteristic strength, f_{sk}, by the material coefficient γ_s.

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

6.3.4.3 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 ε_{sy} and ε_{su}. The steel may be assumed to have the same strain properties and yield stress in both compression and tension. If buckling of steel reinforcement in compression is expected to occur, properties in compression shall be modified accordingly.

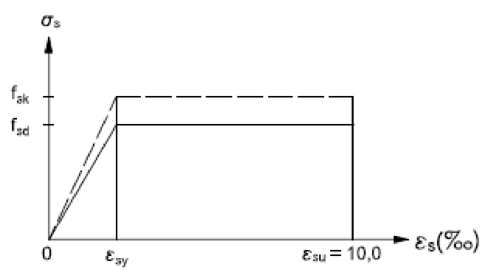


Figure 6-4 Stress-strain diagram for steel reinforcement in accordance with Sec.4

6.3.4.4 For temperatures equal to or larger than 150°C, the stress-strain diagram for ribbed bars in accordance with Sec.4 may be assumed to be in accordance with Figure 6-5 for steel reinforcement.

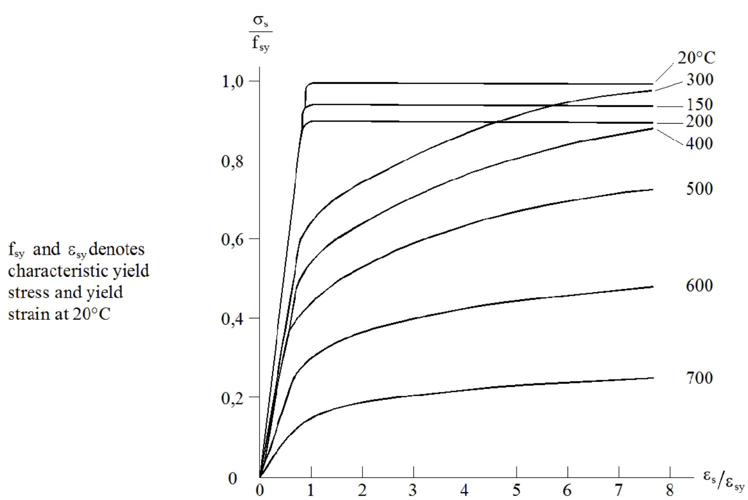


Figure 6-5 The relation between stress and short-term strain for ribbed bars at temperatures above 150°C

The diagram in Figure 6-5 does not include thermal strain or creep strain caused by high temperature.

6.3.4.5 Steel reinforcement exposed to low temperature shall remain ductile under the applicable temperature range. For reinforcement subjected to cryogenic temperatures such as for LNG applications, reference is made to DNVGL-ST-C503.

6.3.5 Fibre reinforced polymer reinforcement stress – strain curves

6.3.5.1 The design Young's modulus of FRP reinforcement bars is defined as E_{Fd}.

6.3.5.2 The stress-strain curve for FRP reinforcement in tension shall be considered as linear until failure at a design strength of f_{Fd}. The value of f_{Fd} depends on the duration of load combinations defined in Table 6-2.

6.3.5.3 FRP reinforcement shall not be considered to work in compression.

6.3.5.4 The impact of temperature on the strength of the FRP reinforcement shall be considered in design, see [4.9.4.5] for more details.

6.3.6 Geometrical dimensions in the calculation of sectional capacities

6.3.6.1 When allowing larger deviations in dimensions than those specified in Table 6-3, the deviations in sectional dimensions and reinforcement position shall be considered in the design. Smaller deviations than the specified tolerances may be considered.

Table 6-3 Acceptable deviations during construction

Type of dimensional deviation	*Maximum tolerance*
Overall dimension	±25 mm
Cross-sectional	±8%
Perpendicularity	8 ‰
Inclination	3 ‰
Local variations (1 m measuring length)	8 mm
Local variations (2 m measuring length)	12 mm
Concrete cover to individual bars	+20/-10 mm
Concrete cover, average value	± 10 mm

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%.

6.3.6.2 If the most unfavourable combination of specified tolerances for sectional dimensions and reinforcement positions are considered, and conformity control subsequently verifies that the actual deviations exceed those specified, then the increased material coefficients in accordance with Table 6-1 shall be used.

Should the as-built documentation show that the intended deviation in tolerances are not met, then the section shall be re-evaluated in all relevant limit states.

6.3.6.3 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 it can be documented through testing that the effected zone is less than 100 mm a more favourable cross-section may be considered. If the structure is set at least 100 mm into rock, the entire concrete section may be calculated as effective for transfer of forces to the ground.

6.3.6.4 The effect of any embedded items of substantially lower stiffness compared to the surrounding concrete, for instance grouted or unbonded tendons, shall be considered where it has a significant influence on the load-bearing capacity. Transfer of compression stresses in lateral direction of ducts of plastic and ungrouted ducts should normally be ignored when calculating axial compression and shear compression capacity. The need for reinforcement for splitting tensile forces should be evaluated.

6.3.7 Tension in structural members

6.3.7.1 Tensile forces shall be provided for by reinforcement with the following exceptions:

- tension caused by shear force
- tension caused by anchorage or splicing of reinforcement, except for anchorage of headed reinforcement (T-heads). T-heads are anchored behind main longitudinal reinforcement, see 6.11.1.23 for guidance on the placement of T-headed reinforcement
- tension caused under partially loaded areas if no increase in the concrete strength is considered.

In these cases, tension may be assumed transferred by the concrete by design in accordance with this standard.

6.3.8 Creep effects

6.3.8.1 Creep effects shall be considered where relevant. Rough estimates of creep effects may be obtained by methods originally developed for simple columns. Two methods are referred to, the so-called “creep factor method” and the creep eccentricity method.

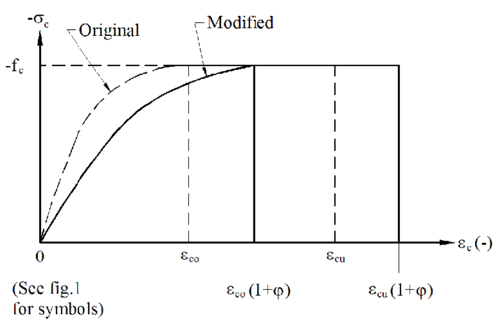


Figure 6-6 Modified stress-strain relationship for concrete

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 $(1 + \varphi)$, φ being the creep factor, see Figure 6-1 and Figure 6-6.

The values of φ should be carefully determined in accordance with recognized principles. The creep factor, φ, should be determined for relevant temperature range, concrete grade, humidity, and surface/volume ratio.

"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 should be noted:

- The total eccentricity should be small enough so that cracking is avoided.
- The value of the load causing creep should be small enough so as to avoid non-linear material behaviour under short term loading.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.3.9 Effect of hydrostatic water pressure

6.3.9.1 The effect of hydrostatic pressure on the concrete strength shall be evaluated where relevant. This effect is a result of the build-up of concrete pore pressure. For lightweight aggregate concrete, this effect may be significant.

6.3.9.2 Water entering cracks, i.e. bending and shear cracks, will have an effect on sectional forces. This effect, which is larger at deeper locations where water hydrostatic pressure is higher, shall be evaluated and if found to be important it shall be included in design.

6.4 Bending moment and axial force (ultimate limit state)

6.4.1 General

6.4.1.1 The capacity for bending moment and axial force may 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 [6.3].

When load effects are determined by applying plastic design analysis techniques, the structures shall be composed of members that 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 [6.1.7].

6.4.1.2 Load effects determined by applying plastic design analysis techniques shall not be applied in FRP reinforced structures.

6.4.1.3 The average calculated compressive strain over the cross-section shall not exceed $(\varepsilon_{co} + \varepsilon_{cu})/2$. Strain caused by shrinkage and linear creep shall be added and the total strain shall be within the above limit.

6.4.1.4 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 subsection 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.

6.4.1.5 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.

6.4.1.6 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.

6.4.1.7 For columns with spiral reinforcement as described in [6.17.10.9] and with normal weight concrete of grades no higher than C45, the sectional resistance capacity may 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 $0.25D_k$, an increased compressive design strength of the concrete may be assumed equal to:

$$f_{cd} + 6 \cdot \frac{f_{ssd} \cdot A_{ss}}{D_k \cdot s} \cdot \left(\frac{1 - 4 \cdot e}{D_k} \right)$$

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_{ssd} is the design strength of the spiral reinforcement, A_{ss}
- e is the eccentricity of loading.

The strains ε_{co} and ε_{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.

6.4.1.8 The capacity of an unreinforced cross-section shall be determined with the concrete stress-strain relationship given in [6.3.2] assuming the concrete does not take tension. The eccentricity shall not be larger than that giving a compressive zone of at least a half of the cross-sectional depth.

6.4.1.9 For primary members tensile strength of the fibre reinforced concrete shall not be considered in design. Ordinary reinforcement shall be used to provide capacity for the effect of bending moments and axial loads. Primary members are members that are important for the integrity of the structure and where failure will have substantial consequences.

6.4.1.10 For secondary members the tensile strength of fibre reinforced concrete containing at least 1 volume per cent steel fibre may be taken as $k_w f_{td}$. For design of cross-sections subjected to axial tension, the factor, k_w, shall be taken as 1.0, when designing for bending moment or bending moment in combination with axial compression the factor, k_w, shall be taken as 1.5 – h/h_l, but not less than 1.0.

h = the cross-sectional height, and h_1 = 1.0 m. Secondary members are members where failure will be without significant consequence.

6.5 Slender structural members

6.5.1 General

6.5.1.1 For structural instability a simplified method of analysis will, in general, be considered acceptable if it is 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 [5.5.12].

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.

6.5.1.2 Displacements caused by short-term actions shall be calculated in accordance with the stress-strain curve given in [6.3.2].

6.5.1.3 The effect of creep shall be calculated in accordance with the history of actions on the structure and characteristic actions, see also [5.5.3].

6.5.1.4 A structural member shall be assumed as slender if, in accordance with [6.5.1.10] to [6.5.1.12], the effect of displacements is significant.

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

6.5.1.5 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.

Reinforcement at least equal to what was assumed when calculating the displacements shall be provided in the structural members.

6.5.1.6 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.

6.5.1.7 The eccentricity shall not be assumed to be less than the largest of 20 mm, l_e/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_e, 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.

6.5.1.8 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.

6.5.1.9 The geometrical slenderness ℓ shall normally not exceed $80\sqrt{1+4\omega_t}$.

where:

$$\omega = \sum \frac{f_{sd} \cdot A_c}{f_{cd} \cdot A_c}$$

A_s = the area of reinforcement

A_c = the cross-sectional area of un-cracked concrete.

The force dependent slenderness, ℓ_N, of a structural member is calculated from the equation:

$$\lambda_n = \lambda \sqrt{\frac{-n_f}{1 + 4\omega_t}}$$

where:

$\lambda = l_e / i$, i = $\sqrt{I_c / A_c}$

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

$$\omega_t = \left(\sum f_{sd} A_s\right) / \left(f_{cd} A_c\right)$$

I_c = the moment of inertia of A_c

N_f = design axial force

l_e = 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 may be entered as two thirds of the total reinforcement area if more accurate values are not used.

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

6.5.1.11 The effect of displacements may be neglected if the force dependent slenderness ℓ_N based on the design actions is less than 10.

6.5.1.12 For a structural member with braced ends, without, lateral forces, this limit may be increased to:

$$\lambda_N = 18 - 8 \mid M_{OA} \mid / \mid M_{OB} \mid$$

where:

$\mid M_{OA} \mid$ = numerical smallest member end moment calculated from 1^{st} order theory

$\mid M_{OB} \mid$ = numerical largest member end moment calculated from 1^{st} order theory.

if the structural member is designed over its entire length for the numerically largest end moment calculated not considering the displacements (first order theory).

The ratio M_{OA}/M_{OB} is the ratio between the numerically smallest and largest end moment calculated not considering the displacements (first order theory). The ratio shall be entered with a positive value when the end moments give tension on the same side of the member (single curvature) and with a negative value when the opposite is the case (double curvature).

6.5.1.13 If the largest end moment is less than that resulting from the smallest eccentricity in accordance with [6.5.1.7], the ratio shall be set to 1.0.

6.5.1.14 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 [6.5.1.9]. The effect of creep may be ignored.

6.5.1.15 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.

6.5.1.16 When designing thin-walled structures, consideration shall be made to local displacements where this will influence the design action effects. The calculation shall be based on approved methods and the principles given in [6.5.1.1] to [6.5.1.10] where these apply.

6.5.1.17 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.

6.6 Shear forces in beams and slabs

6.6.1 Basis

6.6.1.1 The requirements in this subsection 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.

6.6.1.2 Structural members having a smaller ratio between length and depth shall be designed in accordance with [6.9]. The formulation presented here is applicable for solid sections. Hollow sections shall be designed with the general method given in [6.8].

6.6.1.3 The capacity with respect to tensile failure (V_{cd} + V_{sd}) and compressive failure (V_{ccd}) shall be checked. The calculation may be performed in accordance with the simplified methods in [6.6.2], truss model method in [6.6.3] or the general method given in [6.8].

6.6.1.4 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.

6.6.1.5 In support regions, an internal force system shall be chosen in accordance with [6.9].

Tensile failure capacity for direct force applied within a distance a ≤ 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 = distance from the face of the support

d = 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.

6.6.1.6 Shear reinforcement shall be included in the calculations of the capacity only if the provided reinforcement is at least as given in [6.17.9.6] 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 stirrups, s, measured along the longitudinal axis shall satisfy the following:

$s \leq 0.6 \cdot h'(1 + \cot \alpha) \leq h'$ and

$s \leq 500$ mm, see [6.17.9.6].

If the shear force is greater than $2 \cdot f_{td} \cdot b_w \cdot d$, or if in combination with shear force there is significant axial tension, or if the action has fatigue effect, the spacing between stirrups, s, shall satisfy the following:

$s \leq 0.4\ h' \cdot (1 + \cot \alpha)$ nor

$s \leq 0.7 \cdot h'$

where:

α	=	the angle between shear reinforcement and the longitudinal axis
h'	=	the distance between the centroid of the reinforcement on the tension and compression sides of the member.

Only shear reinforcement of an angle between 45 and 90 degrees with the longitudinal axis shall be included in the calculations. Inclined shear reinforcement shall be slanted to the same side of the cross-section as the principal tensile stresses.

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.

6.6.1.7 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 may be designed for the principal shear force at the considered position.

Guidance note:

Resistance to transverse forces in plates and shells should be determined by recognized methods based on equilibrium considerations.

The capacity of a plate or shell may, for example, be considered by establishing uniaxial conditions, by means of equivalent beams or one-way slabs, for all directions, α, in steps of 5°. The corresponding forces will be N_α, the axial force, M_α the bending moment and V_α the transverse force . The magnitude of these forces and moments are found by normal equilibrium considerations. The equivalent area of longitudinal reinforcement in direction α may be calculated as:

$$A_{s\alpha} = A_{sx} \cos^4\alpha + A_{sy} \cos^4\alpha$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.6.1.8 A beam flange subjected to shear forces in its plane can be designed in accordance with the rules for combined action effects in [6.8] or [6.9].

6.6.1.9 The compression failure capacity shall never be larger than the shear force, which combined with other design load effects results in a principal compression stress equal to f_{cd}.

6.6.1.10 FRP bars used for shear reinforcement shall be placed perpendicular to the member longitudinal axis. Consequently, the angle α between the shear reinforcement and the longitudinal axis in [6.6.1.5] shall be taken as 90 degrees.

6.6.1.11 FRP reinforcement may be used as shear reinforcement in reinforced concrete structures. A maximum strain shall be utilized in the shear strength calculation when using the simplified method in [6.6.2].

Guidance note:

A recommended value for maximum strain to be utilized in shear strength calculations is 4‰.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.6.1.12 When designing members with FRP shear reinforcement the provisions of [6.6.2] and [6.6.3] shall apply.

A_s and A_{SV} shall be replaced by A_F and $A_{F,V}$, respectively, in the design formulations.

6.6.1.13 When using the truss model method described in [6.6.3], the maximum stress f_{Fb} in the prefabricated shear reinforcement is:

$$f_{Fb} = \eta_b \cdot f_{F\,d}$$

where:

f_{Fb} = design tensile strength of the bend of FRP bar

η_b = experimentally determined conversion factor for bends.

f_{Fd} is design tensile strength of straight FRP reinforcement for appropriate load combination defined in Sec.5.

6.6.2 Simplified method

6.6.2.1 The validity of the formula used in the simplified method to calculate the concrete shear capacity in tension for concrete grades higher than C65 shall be documented by testing. These includes formula in [6.6.2.2], [6.6.2.3] and [6.6.2.4]. Otherwise, the concrete shear capacity in tension for concrete grades higher than C65 shall be limited to that of C65.

6.6.2.2 For a structural member without shear reinforcement, the shear capacity at tensile failure should be taken as V_{cd}. The capacity for shear force without a coinciding axial force should be taken as:

$$V_{cd} = V_{co} = 0.3\left(f_{td} + \frac{k_A A_s}{\gamma_c b_w d}\right) b_w d k_V \le 0.6 f_{td} b_w d k_v$$

where:

A_s = the cross-section area of properly anchored reinforcement on the tension side [mm^2]

b_w = width of beam [mm]

d = distance from centroid of tensile reinforcement to compression edge [mm]

d_1 = 1000 mm

k_A = 100 MPa

k_v = For slabs and beams without shear reinforcement the factor k_V is set equal to 1.5 – d/d_1, but not greater than 1.4 nor less than 1.0.

6.6.2.3 The capacity at tensile failure for shear force in combination with axial compression may be taken as:

$$V_{cd} = V_{co} + 0.8 \cdot M_o \cdot \left|\frac{V_f}{M_f}\right| \leq \left(f_{td} \cdot k_v - \frac{0.25 \cdot N_f}{A_c}\right) \cdot b_w \cdot z_1$$

where:

M_o	=	$-N_f \cdot W_c/A_c$
N_f	=	axial design load, positive as tension
V_f	=	design shear force for the cross-section under the considered condition
M_f	=	total bending 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 f_{cd}
W_c	=	the section modulus of the concrete cross-section with respect to the extreme tension fibre or the fibre with least compression
I_c	=	the moment of inertia for the un-cracked concrete section
S_c	=	area moment about the centroid axis of the cross-section for one part of the concrete section
z_1	=	the greater of 0.7 d and I_c/S_c
b_w	=	width of beam web [mm].

6.6.2.4 The capacity for shear force with coinciding axial tension should be taken as the greatest of:

$$V_{cd} = V_{co}\left(1 - \frac{N_f}{1.5 \cdot f_{td} \cdot A_c}\right) \geq 0$$

and

$$V_{cd} = V_{co}\left(1 - \frac{\varepsilon_s}{\varepsilon_{sy}}\right)$$

where:

ε_s = the strain in the most stressed longitudinal reinforcement calculated on the basis of all simultaneous acting load actions, where the effect of constraint is included.

When calculating V_{cd}, no part of the longitudinal reinforcement in the considered section shall have greater design strain than ε_{sy}.

6.6.2.5 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 for steel reinforced members.

6.6.2.6 The capacity portion V_{sd} is determined by the force component in the direction of the shear force from steel 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:

$V_{sd} = \Sigma\ (f_{sd} \cdot A_{SV} \cdot \sin \alpha)$

For transverse reinforcement consisting of units with spacing s measured along the longitudinal axis, this becomes:

$$V_{sd} = \left(\frac{f_{sd} \cdot A_{sv} \cdot z}{s}\right)(1 + \cot\alpha) \cdot \sin\alpha$$

z should 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.

6.6.2.7 The capacity for compression failure shall be taken as:

$$V_{ccd} = 0.30 \cdot f_{cd} \cdot b_w \cdot z(1 + \cot\alpha) < 0.45 \cdot f_{cd} \cdot b_w \cdot z$$

6.6.2.8 When applying [6.6.2.2] to members with FRP reinforcement as longitudinal tensile reinforcement, modifications of k_v and k_A are required due to the different Young's modulus of the FRP compared to steel. This affects the crack width and aggregate interlock when calculating the contribution from concrete, V_{co}. k_A shall therefore be taken as:

$k_A = 100 \cdot E_F/E_{sk}$

Where k_A has units of MPa.

k_v shall be determined through testing.

6.6.2.9 For concrete members reinforced with FRP bars as shear reinforcement, the shear strength of the concrete section shall be taken as the lower of:

- V_{sd}, calculated using $f_{Fb} = \eta_b \cdot f_{Fd}$. The material factor for strength shall correspond to the duration of the load.
- $V_{co} + V_{sd}$, where V_{sd} is calculated using f_F for a maximum strain. The material factor for stiffness shall be used to determine f_F.

For a concrete section with no shear reinforcement (slabs, wall etc.) the shear capacity shall be taken as V_{co}.

Guidance note:

A recommended value for maximum strain is 4‰.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.6.3 Truss model method

6.6.3.1 The capacity for shear force only or in combination with other action effects should be calculated based on an assumed internal truss model with compressive concrete struts at an angle, θ, 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 [6.6.2] shall not be included in the capacity.

6.6.3.2 For members subjected to shear force not in combination with axial compression, the angle θ shall be chosen between 25° and 60°.

6.6.3.3 For members subjected to shear force with axial compression, the angle θ may be chosen less than 25°, but not less than that corresponding to the direction of the principal compression calculated for uncracked concrete.

6.6.3.4 For members subjected to shear force in combination with not negligible axial tension, the angle shall normally be taken as θ = 45°.

6.6.3.5 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 θ 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:

α is the angle between the transverse reinforcement and the longitudinal axis

θ is the angle between the inclined concrete compression struts and the longitudinal axis.

6.6.3.6 For transverse reinforcement consisting of units with a spacing s measured along the longitudinal axis, the shear capacity becomes:

$$V_{sd} = \left(\frac{f_{sd} \cdot A_{SV} \cdot z}{s} \right) (\cot \theta + \cot \alpha) \sin \alpha$$

6.6.3.7 The shear reinforcement for the most unfavourable load case may be designed for the smallest shear force within a length z · cot θ, corresponding to projection of the inclined crack, measured along the longitudinal axis.

6.6.3.8 The capacity at compression failure shall be taken as:

$$V_{ccd} = f_{c2d} \cdot b_w \cdot z \frac{(\cot \theta + \cot \alpha)}{1 + \cot^2 \theta}$$

The design compressive strength, f_{c2d} in the compression field shall be determined for the calculated state of strain in accordance with [6.8]. When θ is assumed between 30 and 60 degrees, the design compressive strength should be assumed as:

$$f_{c2d} = 0.6 f_{cd}$$

6.6.3.9 For reinforced concrete members reinforced with FRP reinforcement as shear reinforcement the f_{sd} in [6.6.3.5] and [6.6.3.6] shall be taken in accordance with the reduced strength formulation for bent FRP shear reinforcement in accordance with [6.6.1.10].

6.6.4 Additional force in the longitudinal reinforcement from shear force

6.6.4.1 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 [6.6.3.6]).

The force F_{SV} shall be assumed to act in both chords if this is unfavourable, i.e. areas near points with zero moment.

6.6.4.2 When calculating according to the truss model method, a tensile force, F_{sv} shall be assumed on both sides of the cross-section:

$$F_{SV} = 0.5 \cdot V_f \cdot (\cot \theta - \cot \alpha) \geq 0$$

6.6.4.3 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.

6.6.5 Slabs subjected to concentrated actions

6.6.5.1 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 subsection. This subsection is not applicable for cases in which concentrated actions induce tension perpendicular to the middle plane of the slab, as a result, for example, of a concentrated load and bending moment. In these cases, a detailed evaluation of the transfer of tension forces shall be performed.

6.6.5.2 The calculation should 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.

6.6.5.3 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 $1.0 \cdot d$ 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 $1.0 \cdot d$
— the perimeter of the governing section shall be minimized, but straight edges may be assumed i.e. corners are not rounded, see Figure 6-7.

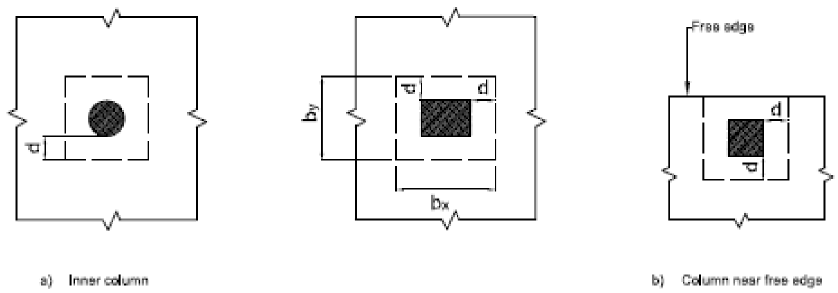


Figure 6-7 Cross-section for design check of shear capacity for concentrated load on plates

6.6.5.4 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 $2 \cdot d$.

6.6.5.5 When a concentrated load is applied in the vicinity of a free edge, in addition to the section given in [6.6.5.2], a governing section shall be assumed extending to the free edge and perpendicular to this, see Figure 6-7.

6.6.5.6 Similar rules apply to corners of slabs, see Figure 6-8 a. 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 6-8 b.

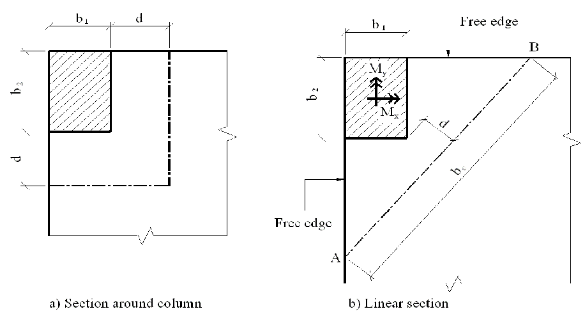


Figure 6-8 Cross-section for design check of plates with columns at the corner

6.6.5.7 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 6-9.

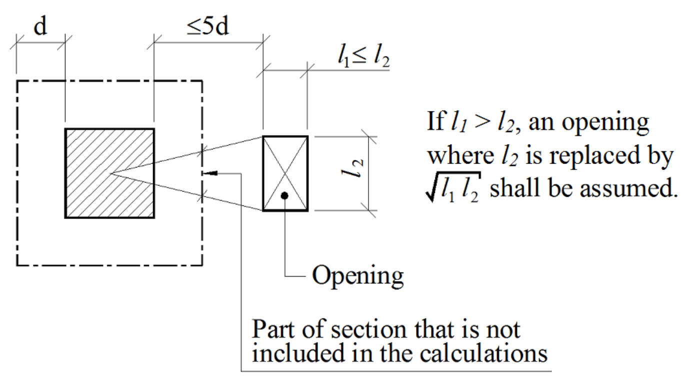


Figure 6-9 Reduction in capacity near opening in plates

6.6.5.8 The distribution of shear forces along the critical section should be calculated in accordance with the theory for elastic plates.

6.6.5.9 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.

6.6.5.10 For a rectangular section, this portion of the moment should be taken as:

$$\frac{M_f}{\left(1+\frac{b_y}{b_x}\right)}$$

Here b_y is the length of the side of the critical section that is parallel to the moment axis and b_x 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.

6.6.5.11 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.

6.6.5.12 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 [6.6.2.1].

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, $\rho_x = A_{sx} /(b \cdot d)$ and $\rho_y = A_{sy} /(b \cdot d)$, 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.

$$\rho = \sqrt{\rho_x \rho_y}$$

The reinforcement ratios shall be determined as average values over a width $2 \cdot d$ to each side of the loaded area. The capacity shall be reduced in accordance with the regulations in [6.6.2.3], 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 [6.6.2].

6.6.5.13 If the shear capacity of a slab without shear reinforcement calculated in accordance with [6.6.5.1] to [6.6.5.12] is less than the calculated action effect, shear reinforcement shall be provided in areas where the shear capacity is insufficient.

6.6.5.14 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:

$$V_{sd} = \sum f_{sd} A_{SV} \sin \alpha$$

V_{sd} shall at least be equal to $0.75 \cdot V_{cd}$.

6.6.5.15 The required shear reinforcement calculated in the governing section shall be distributed along at least two rows at a distance 0.5 d to $1.0 \cdot d$ from the face.

6.6.5.16 Outside the section 1.0 d from the face, the required shear reinforcement shall be calculated for plane sections in accordance with [6.6.2.4] and [6.6.2.5] and be distributed in accordance with [6.6.1.5]. The distance between the reinforcement units in the direction perpendicular to the governing section should be up to 0.75 d in the span direction.

6.6.5.17 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.

6.6.5.18 Compression failure caused by shear force shall be considered in accordance with [6.6.2.6] for sections at the face of the loaded area.

6.6.5.19 For concrete members reinforced with FRP bars as longitudinal tensile reinforcement, the provision of [6.6.5.12] and [6.6.5.14] shall be supplemented by the requirements in [6.6.2.7] and [6.6.2.8] for the prediction of the shear strength V_{cd}.

6.6.6 Anchorage of fittings

6.6.6.1 For anchorage of load-carrying components where the anchorage failure is of major consequences, the reliability of the system shall be documented in ULS considering that the structure may be extensively cracked. To achieve a reliable anchorage a good practice is to provide additional reinforcement or post-tensioned long bolts or tendons which transfer the load as compression forces. It is required that load-carrying components attached to the structural member at an angle to the main load carrying system, shall be anchored in to the compression zone, behind the main longitudinal reinforcement.

6.7 Torsional moments in beams

6.7.1 General

6.7.1.1 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 may normally be performed without considering torsional moments.

6.7.1.2 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_{c2d}, where f_{c2d} 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 may 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 $0.4 \cdot f_{cd}$. The concrete outside the stirrups shall always be neglected if the concrete surface is convex.

6.7.1.3 The individual cross-sectional parts should be designed for the calculated shear forces in accordance with the general method in [6.8] or in accordance with the requirements of [6.7.1.4] to [6.7.1.7].

6.7.1.4 Internal forces shall be determined in accordance with recognized methods based on the equilibrium requirements under the assumption that the concrete does not 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 transfers compression.

6.7.1.5 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_{c2d} according to [6.8.1.6] and [6.8.1.7]. 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_{c2d}.

6.7.1.6 The capacity at tensile failure shall be determined by the maximum tensile forces that the torsional reinforcement transfers in the assumed spatial truss. The design may be based on a consideration of shear walls. It shall be demonstrated that the corresponding internal forces in the corners are transferred.

6.7.1.7 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.

6.7.1.8 Torsional reinforcement shall be provided as closed stirrups with proper anchorage. In structures or structural members which according to this standard shall be designed for torsion, this stirrup reinforcement in each face shall have a minimum cross-section of:

$$0.25 A_c \frac{f_{tk}}{f_{sk}}$$

where A_c 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 [6.7.1.2] and [6.7.1.3] for a solid cross-section. The tensile strength, f_{tk}, shall not be lower than 2.55 MPa.

6.7.1.9 If the load transfer is totally dependent on the torsional capacity, the 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.

6.7.1.10 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.

6.7.1.11 The longitudinal reinforcement may be less than this provided axial compression is acting simultaneously or the stirrup reinforcement is placed nearby 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.

6.7.1.12 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.

6.7.1.13 For reinforced concrete members reinforced with FRP bars as torsional reinforcement the provision of [6.7.1.6] and [6.7.1.7] shall be supplemented by limiting the tensile strain in the torsional reinforcement.

Guidance note:

A recommended value for maximum strain to be utilized is 4‰.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.7.1.14 For reinforced concrete members reinforced with FRP reinforcement as torsional reinforcement. The minimum torsion reinforcement provided by [6.7.1.8] shall modified by replacing f_{sk} with the tensile stress of the FRP reinforcement corresponding to a maximum strain.

Guidance note:

A recommended value for maximum strain to be utilized is 4‰. The corresponding tensile strength is $f_{sk} = E_F \times 4‰$.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.8 General design method for structural members subjected to in-plane forces

6.8.1 General

6.8.1.1 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.

6.8.1.2 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.

See [6.8.1.10]

6.8.1.3 Strains and stresses shall be calculated as average values over a cracked region. The strains may 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.

6.8.1.4 Design of shear walls, plates and shells shall 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. Transverse force shall be treated in addition to in-plane forces.

6.8.1.5 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.

6.8.1.6 The design shall be based on stress-strain relationships for both reinforcement and concrete in areas subjected to a biaxial stress state that is documented to give agreement between calculated capacity and tests. For steel reinforcement, the relation between average strain and average stress given in [6.3.4.3] may be assumed. For FRP reinforcement, the stress strain relation is given in [6.3.5].

6.8.1.7 For concrete subjected to compression, the relationship between strain and stress given in [6.3.2.1], with the stress ordinate reduced by the factor f_{c2d}/f_{cd} may be assumed.

For concrete in the assumed compression field a reduced design compressive strength shall be taken as:

$$f_{c2d} = \frac{f_{cd}}{0.8 + 100 \cdot \varepsilon_1} \leq f_{cd}$$

where ε_1 is the average principal tensile strain.

6.8.1.8 The average tensile stresses between cracks shall be determined by relationships documented by representative tests, if tensile stresses in the concrete between cracks are considered in design.

6.8.1.9 It shall be demonstrated that the cracks transfer both the shear stresses in the concrete and the tensile stresses in the reinforcement which are derived from the equilibrium requirements.

DNV GL AS

6.8.1.10 If the concrete tensile stresses between the cracks are not considered (σ_1 = 0), the check of the stress condition in the cracks may be waived.

6.8.1.11 The stresses in the steel reinforcement at the cracks shall be determined from the equilibrium conditions and shall not exceed the design strength of the steel reinforcement. For FRP reinforcement, stresses shall not exceed a design stress corresponding to a maximum strain. The design strength shall be calculated considering a material factor related to the duration of the loading according to C108.

Guidance note:

A recommended value for maximum strain to be utilized is 4‰.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.8.2 Membrane (in-plane) shear resistance

6.8.2.1 Resistance to membrane forces in plates and shells shall be determined by recognized methods based on equilibrium considerations. The tensile strength of concrete shall be neglected.

6.8.2.2 For membrane forces only, i.e. when the slab element is subjected to in-plane forces only (see Figure 6-10) 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 (σ_c) at angle θ 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:

$$F_x = N_x + \left|N_{xy}\right| \cdot \cot(\theta)$$

$$F_y = N_y + \left|N_{xy}\right| \cdot \tan(\theta)$$

$$\sigma_c = \frac{\left|N_{xy}\right|}{b \cdot \sin\theta \cdot \cos\theta}$$

(the units for F are in force/unit length) valid for positive values of F_x and F_y and taking tensile stresses as positive.

The angle θ may be chosen arbitrarily for each loading case and each slab element, paying due regard to the requirements of [6.17] concerning minimum reinforcement.

For $N_x < -|N_{xy}| \cdot \cot\theta$ no reinforcement is required in the x-direction. F_y and σ_c are then given by:

$$F_y = N_y - \frac{N_{xy}^{\ 2}}{N_x}$$

$$\sigma_c = \frac{N_x + \frac{N_{xy}^{\ 2}}{N_x}}{h}$$

For $N_y < -|N_{xy}| \tan\theta$ no reinforcement is required in the y-direction F_x and σ_c are then given by:

$$F_x = N_x - \frac{N_{xy}^{\ 2}}{N_y}$$

$$\sigma_c = \frac{N_y + \frac{N_{xy}^2}{N_y}}{h}$$

Finally, a situation may occur where both N_x and N_y are negative and $N_x \cdot N_y > N_{xy}^2$. No reinforcement is required and principal membrane forces may be calculated in accordance with conventional formulae.

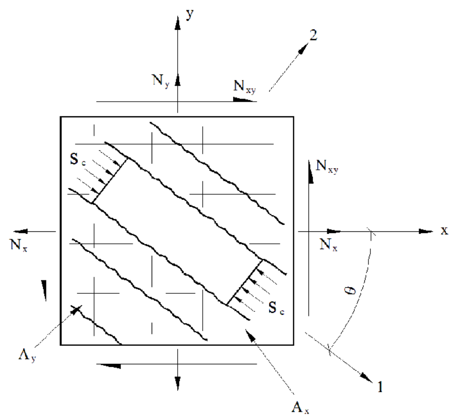


Figure 6-10 Slab element subjected to membrane forces

6.8.2.3 Membrane forces and bending moments combined

In cases where a slab element is subjected to combinations of moments and membrane forces, or to moments only, the slab element may be regarded as a sandwich consisting of two outer layers and a central zone. The applied forces and moments may be resolved into statically equivalent membrane forces on the outer layers as shown in Figure 6-11. Each layer is then designed in accordance with the general principles given for membrane forces only. Transverse force shall be treated in addition to in-plane forces

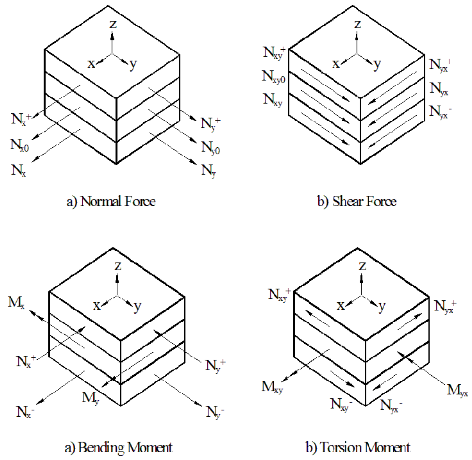


Figure 6-11 Applied forces and moments resolved into membrane forces in sandwich layers

6.9 Regions with discontinuity in geometry or loads

6.9.1 General

6.9.1.1 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 satisfies 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 un-cracked structure in accordance with the theory of elasticity.

6.9.1.2 Internal forces in the member at a distance less than d from the support or from concentrated loads shall be determined inline with the provisions of [6.9]. The provisions of [6.9] may be used to determine internal forces in the member at distances up to 2 · d the support or from concentrated loads.

6.9.1.3 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.

6.9.1.4 Tensile forces caused by possible deviation in the assumed compressive field direction shall be considered.

The reinforcement shall be shaped in accordance with the analytical model and be anchored in accordance with the provisions of [6.11] at the assumed joints.

6.9.1.5 Calculated concrete stresses in struts shall not exceed f_{c2d} as given in [6.8.1.7]. When calculating f_{c2d}, 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.

6.9.1.6 It shall be demonstrated that the calculated forces in the assumed struts and ties are transferred in the joints, with design concrete compressive strength in accordance with [6.9.1.5], and the other provisions of this standard. Increased design concrete compressive strength may be taken into account for partially loaded areas according to provisions in [6.12] if splitting reinforcement is provided.

6.9.1.7 If the reduced compressive concrete strength f_{c2d} 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 having bi- or triaxial compression.
— $0.9 \cdot f_{cd}$ in joints where tensile strains occur in only one direction , i.e. tensile reinforcement is provided in only one direction
— $0.7 \cdot f_{cd}$ in joints where tensile strains occur in more than one direction i.e. tensile reinforcement is provided in more than one direction

6.9.1.8 When applying the truss analogy in area with discontinuity in geometry, the maximum stress in the FRP bars shall not exceed the design strength specified in [6.6.1.10] for the appropriate load combination as specified in [5.4.1].

6.10 Shear forces in construction joints

6.10.1 General

6.10.1.1 In concrete joints between hardened concrete and concrete cast against it, the transfer of shear forces should be assumed in accordance with the provisions given in this subsection.

6.10.1.2 Construction joints shall not be assumed to transfer larger forces than if the structure was monolithically cast.

6.10.1.3 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 toothing shall have a length parallel with the direction of the force not exceeding 8 times the depth and the side surfaces of the toothing shall make an angle with the direction of the joint no less than 60°. The minimum depth shall be 10 mm.

6.10.1.4 The design shear strength of concrete, τ_{cd}, may 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.

6.10.1.5 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:

$$V_d = \tau_{cd} \cdot A_c + f_{sd} \cdot A_s (\cos \alpha + \mu \cdot \sin \alpha) - \mu \cdot \sigma_c \cdot A_c < 0.3 \cdot f_{cd} \cdot A_c$$

where:

A_S	=	the reinforcement area that is sufficiently anchored on both sides of the joint and that is not utilised for other purposes
α	=	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
μ	=	the friction factor
σ_c	=	the smallest simultaneously acting concrete stress perpendicular to the contact surface.

6.10.1.6 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.

6.10.1.7 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 6-4 that gives the minimum capacity shall be used in the design.

τ_{cd} is a function of f_{td}. For concrete grades higher than C65 the validity of the formula for τ_{cd} in Table 6-4 shall be documented by testing otherwise τ_{cd} shall be limited to that of C65.

Table 6-4 Values for force transfer in construction joints

Contact surface	*$\Sigma A_s > 0.001\ A_c$ or $\sigma_c < -0.4$ MPa*			
	Combination 1		*Combination 2*	
	τ_{cd}	μ	τ_{cd}	μ
Smooth	0	0.70	0	0.7
Rough	0	1.50	$0.6f_{td}$	0.8
Toothed	0	1.80	$1.5\ f_{td}$	0.8

6.10.1.8 When the contact surfaces are toothed, the design shear strength τ_{cd} shall be assumed to act on a cross-section giving the smallest net area at the base of the toothing.

6.10.1.9 The design strength, τ_{cd}, in the contact surface shall be determined for the concrete part having the lowest strength.

6.10.1.10 Reinforcement may be omitted in rough or toothed construction joints transferring shear forces, in the following cases:

— Where the parts are sufficiently secured against moving from each other perpendicular to surfaces by other means. The capacity shall be calculated in accordance with [6.10.1.5] to [6.10.1.9].
— In structures with uniformly distributed dominantly static live load not exceeding 5 kPa and minor failure consequences. The design bond strength of the concrete shall be taken as $0.5 \cdot \tau_{cd}$, and the forces in the concrete joint shall be determined in accordance with the method described for composite structures in [6.1.5].
— In structures where the composite action between the parts is not accounted for when calculating the capacity. It shall be verified that this has no detrimental effects in the serviceability limit state.

6.10.1.11 When calculating capacity for transfer of shear forces in concreted joints between precast members, the provisions in [6.10.1.5] to [6.10.1.9] may be waived provided there is sufficient basis to assuming other values than given in Table 6-4.

6.10.1.12 For concrete members reinforced with FRP reinforcement as longitudinal reinforcement crossing a construction joint, the provisions in [6.10.1.5] shall be modified by replacing f_{sd} with the design stress of the FRP reinforcement corresponding to a maximum strain. The design strength shall be calculated considering a material factor related to the duration of the loading according to [6.3.1.8].

Guidance note:

A recommended value for maximum strain to be utilized is 4‰.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.10.1.13 For concrete members with FRP bars as reinforcement crossing a construction joint, the minimum amount of reinforcement required in accordance with the provisions in [6.10.1.6] shall be modified with the following factor $200/E_F$, with E_F in MPa when the alternative compressive stress criteria is not satisfied.

6.11 Bond strength and anchorage failure

6.11.1 General

6.11.1.1 The distances between the reinforcement bars shall be such as to ensure good bond.

6.11.1.2 Reinforcement in different layers shall be aligned in planes leaving sufficient space to allow for the passage of an internal vibrator.

6.11.1.3 Lap joints shall be made in a way that secures transfer of force from one reinforcement bar to another. The reduction of strength of a lap joint due to closely spaced lap joints shall be taken into account where relevant.

6.11.1.4 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).

6.11.1.5 Resistance against bond and anchorage failure shall 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 shall not be taken higher than 70% of the value for good bond zones.

Consideration shall be given to the state of stress in the anchorage zone. Adequate bond resistance shall be assured by transverse reinforcement, stirrups, spirals, hooks or mechanical anchorages.

6.11.1.6 Individual reinforcement bars shall have a development length no less than:

$$l_b = \frac{0.25 \cdot \phi \cdot \sigma_s}{f_{bd}} + t$$

where:

ϕ	=	the diameter of the reinforcement bar
σ_s	=	the calculated stress in the reinforcement bar in ultimate limit state at the cross-section in question
f_{bd}	=	the design bond strength, calculated in accordance with [6.11.1.16]
t	=	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 $3 \cdot \phi$.

6.11.1.7 Required lap length when splicing shall be taken equal to the calculated development length. The required lap length shall be not less than the greater of $20 \cdot \phi$ and 300 mm. The development length shall not be assumed to be effective over a length exceeding $80 \cdot \phi$.

6.11.1.8 Bundled reinforcement bars shall have a development length no less than:

$$l_b = \frac{0.25 \cdot \phi_e \cdot \sigma_s}{k_n \cdot f_{bc} + f_{bs}} + t$$

where:

ϕ_e = equivalent diameter in term of reinforcement cross section
f_{bc} = design bond strengths in accordance with [6.11.1.16] with $\phi = \phi_e$
f_{bs} = design bond strengths in accordance with [6.11.1.16] with $\phi = \phi_e$
k_n = 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 [6.11.1.6].

The development length shall not be assumed to be effective over a length exceeding $80 \cdot \phi_e$.

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.

6.11.1.9 The development length for steel welded wire fabric shall be no less than:

$$l'_b = l_b - 0.3 \cdot \sum \frac{F_{vn}}{\gamma_s \cdot \phi \cdot f_{bd}}$$

where:

$\Sigma F_{vn} / \gamma_s$ = sum of forces F_{vn} corresponding to shear failure at cross wire welds within the development length
l_b = development length in accordance with [6.11.1.6]
l'_b = shall not be taken as larger than the development length in accordance with [6.11.1.25]
f_{bd} = design bond strength calculated in accordance with [6.11.1.16], see also [6.11.1.6]
F_{vn} = $0.3 \cdot A_s \cdot f_{sk} \geq 4$ kN, where A_s is the cross-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.

6.11.1.10 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:

α is a factor given in Table 6-5

β is a factor given in Table 6-5

ϕ is the nominal diameter of the reinforcement unit

σ_p is the reinforcement stress due to prestressing

f_{bc} is the concrete related portion of the design bond strength in accordance with [6.11.1.16].

Table 6-5 Coefficients to be used when calculating development length for prestressed reinforcement units

Type of reinforcement	*Smooth release of prestressing tension force*		*Sudden release of prestressed tension force*	
	α	β	α	β
Plain wire	10	0.20	-	-
Indented wire	0	0.17	10	0.21
Strand	0	0.14	5	0.17
Ribbed bar	0	0.07	0	0.08
FRP	To be provided in FRP product or type approval certificate			

The part $\alpha \cdot \phi$ in the formula for l_{bp} defines a length where no force transmission is assumed.

6.11.1.11 Post tensioning anchorages shall be designed for the design strength of the tendon. The anchorage unit shall 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 shall be approved.

6.11.1.12 The design of anchorage zones shall be in accordance with recognized methods. Reinforcement shall be provided, where required, to prevent bursting or splitting. The design strength of such reinforcement should be limited in order to control cracking due to the applied force:

— to 300 MPa in case of steel reinforcement
— to the stress corresponding to a strain of 2‰ in case of FRP reinforcement. In order to assess the stress corresponding to this strain E_{Fd} shall be used.

6.11.1.13 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.

6.11.1.14 Development of tensile force caused by external loads shall be calculated in accordance with [6.11.1.6]. 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 σ_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}.

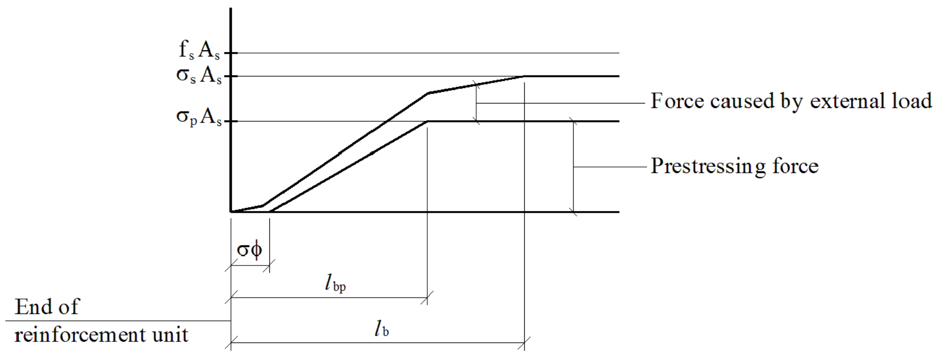


Figure 6-12 Prestressed force introduction length where prestressed force is anchored in bond

6.11.1.15 Transverse tensile forces in the development zone shall be resisted by reinforcement, unless it is shown that reinforcement may be omitted.

6.11.1.16 The design bond strength f_{bd} for ribbed bar, indented bar, indented wire and strand should be taken as:

$$f_{bd} = f_{bc} + f_{bs} \leq 2 \cdot k_1 \cdot f_{td}$$

where:

$$f_{bc} = k_1 k_2 f_{td} \left(\frac{1}{3} + \frac{2c}{3\emptyset} \right)$$

$$f_{bs} = k_3 \left(\frac{A_{st}}{s_1 \cdot \phi} \right) \leq 1.5 MPa$$

where:

k_1	=	factor depending of the type of reinforcement, given in Table 6-6
c	=	the least of the dimensions c_1, c_2 and $(s_1 - \phi)/2$ given in Figure 6-13
ϕ	=	the diameter of the anchored reinforcement
k_3	=	factor dependent on the transverse reinforcement and its position as given in Figure 6-14.

The design bond strength, f_{bd}, is a function of the characteristic tensile strength of concrete, f_{tk}. For concrete grades higher than C65 the validity of the above formula for f_{bd} shall be documented by testing, otherwise f_{bd} shall be limited to that of C65.

The factor k_3 is taken as zero for strands

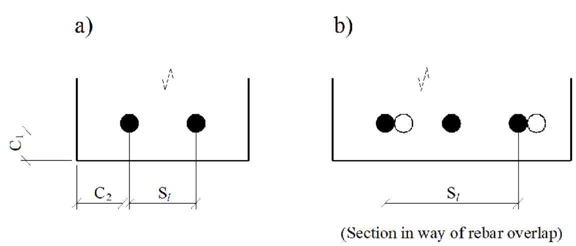
A_{st} = the area of transverse reinforcement not utilized for other tensile forces and having a spacing not greater than 12 times the diameter of the anchored reinforcement. If the reinforcement is partly utilized, the area shall be proportionally reduced

s_1 = 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 $(6\,c + \phi)$ whichever is the larger, k_2 has the value 1.0 if s is less than the larger of $5 \cdot \phi$ and $(3c + \phi)$. For intermediate values interpolate linearly.

6.11.1.17 For plain reinforcement take:

$$f_{bd} = k_1 \cdot f_{td}$$



a) distance for anchorage, b) distance for splices.

Figure 6-13 Values of concrete cover and bar spacing for calculation of bond strength

Table 6-6 Values of k_1 for various types of reinforcement

Type of reinforcement	k_1
Ribbed bar	1.4
Intented bar and wire	1.2
Strand	1.2
Plain bar	0.9
Plain wire in welded wire fabric and prestressed reinforcement	0.5
FRP	To be provided in FRP product or type approval certificate

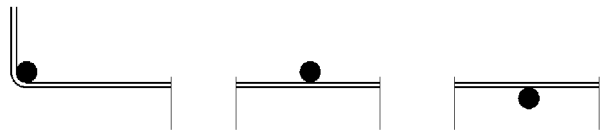


Figure 6-14 Values of k_3 for various types of transverse reinforcement for calculation of bond strength

6.11.1.18 When calculating development of force in reinforcement which during concreting has an angle less than 20° to the horizontal plane, the following reduction of the portion f_{bc}, of the design bond strength f_{bd} according to [6.11.1.16] 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 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.

6.11.1.19 At a simply supported end, the development length determined according to [6.11.1.6] to [6.11.1.15] 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_{bd} shall not have a higher value than what corresponds to the maximum value in accordance with [6.11.1.16].

6.11.1.20 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.

6.11.1.21 If reinforcement in several layers are spliced or anchored in the same section, the capacity shall be limited to the value that may 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.

6.11.1.22 Reinforcement may also be anchored with special anchor units such as T-heads. A combination of several anchorage methods may be utilized. The total anchorage capacity should 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.

6.11.1.23 Where headed reinforcement (T-heads) are used for anchorage of reinforcement, special attention shall be paid in design and construction to ensure the hooking of the plate behind the outer most main reinforcement. Hooking of the plate is required to:

— fully utilize the anchored bar
— avoid detrimental effects arising from unhooked plates generating splitting tensile forces in the anchorage zone of the main longitudinal reinforcement.

6.11.1.24 For steel 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 [6.17.4.8]. 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 equal to or greater than 135°, the straight part may be reduced to five times the diameter of the bar.

For bars of steel compliant with [4.7], see also [6.17.4], 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 equal to or greater than 135° the force may 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 weldable grade B500B and B500C compliant with [4.7] 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 $4 \cdot \phi$ and otherwise bent in accordance with [6.17.4].

6.11.1.25 For FRP reinforcement, the capacity of the reinforcement in the bend shall be calculated in accordance with [6.6.1.10].

6.11.1.26 If the development length of steel reinforcement is not calculated in accordance with [6.11.1.6] to [6.11.1.8], the anchorage length of reinforcement in one layer in normal density concrete may simplified be determined as follows:

a) For ribbed bars of steel compliant with [4.7], the anchorage length shall be taken as $50 \cdot \phi$. This applies provided the concrete cover is at least ϕ and the spacing between the anchored bars is at least $8 \cdot \phi$. If transverse reinforcement is located closest to the concrete surface and the concrete cover of the anchored reinforcement is at least $1.5 \cdot \phi$, the spacing shall be at least $5 \cdot \phi$.
b) For plain bars with end hooks, the anchorage length is taken as $40 \cdot \phi$ assuming that $f_{sk} \leq 250$ MPa.
c) For welded wire fabric, the anchorage length shall be at least so large that:
    — 3 transverse bars are located in the anchorage zone for welded wire fabric of bars with diameters from 4 to 9 mm.
    — 4 transverse bars are located in the anchorage zone for welded wire fabric of bars with diameters from 10 to 12 mm.

In addition, the anchorage length shall be no less than:

— $30 \cdot \phi$ for mesh made of indented bars
— $40 \cdot \phi$ for mesh made of plain bars.

The development of the force along the anchorage length may be assumed uniform.

For reinforcement which has a concrete depth below the reinforcement larger than 150 mm or an angle less than 20° to the horizontal plane, the anchorage length shall be increased by $10 \cdot \phi$ for ribbed bars and welded wire fabric of indented bars, and $20 \cdot \phi$ for plain bars. These provisions are not applicable to FRP bars.

6.11.1.27 The required minimum reinforcement in accordance with [6.17] shall be spliced for its full capacity.

6.11.1.28 Along the development length, a transverse reinforcement or stirrups shall be provided in accordance with [6.17.3.3], unless a more accurate assessment is made. If this reinforcement is provided with FRP rods, it shall be designed considering a stress corresponding to a strain of 4‰. In order to assess the stress corresponding to this strain E_{Fd} shall be used.

6.12 Partially loaded areas

6.12.1 General

6.12.1.1 Where a compression force F_f 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 may be assumed transferred further in the same direction and distributed over a larger distribution area, A_2 in the concrete member. This provision is applicable for design in ULS. For fatigue life prediction, any increase in strength shall be documented.

6.12.1.2 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.

6.12.1.3 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.

6.12.1.4 If more than one load acts simultaneously the respective distribution areas shall not overlap each other.

6.12.1.5 The compressive capacity for normal density concrete may be taken as:

$$F_{cd} = A_1 \cdot f_{cd} \sqrt[3]{\frac{A_2}{A_1}}$$

6.12.1.6 The compressive capacity for lightweight aggregate concrete may be taken as:

$$F_{cd} = A_1 \cdot f_{cd} \sqrt[4]{\frac{A_2}{A_1}}$$

6.12.1.7 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 6-15.

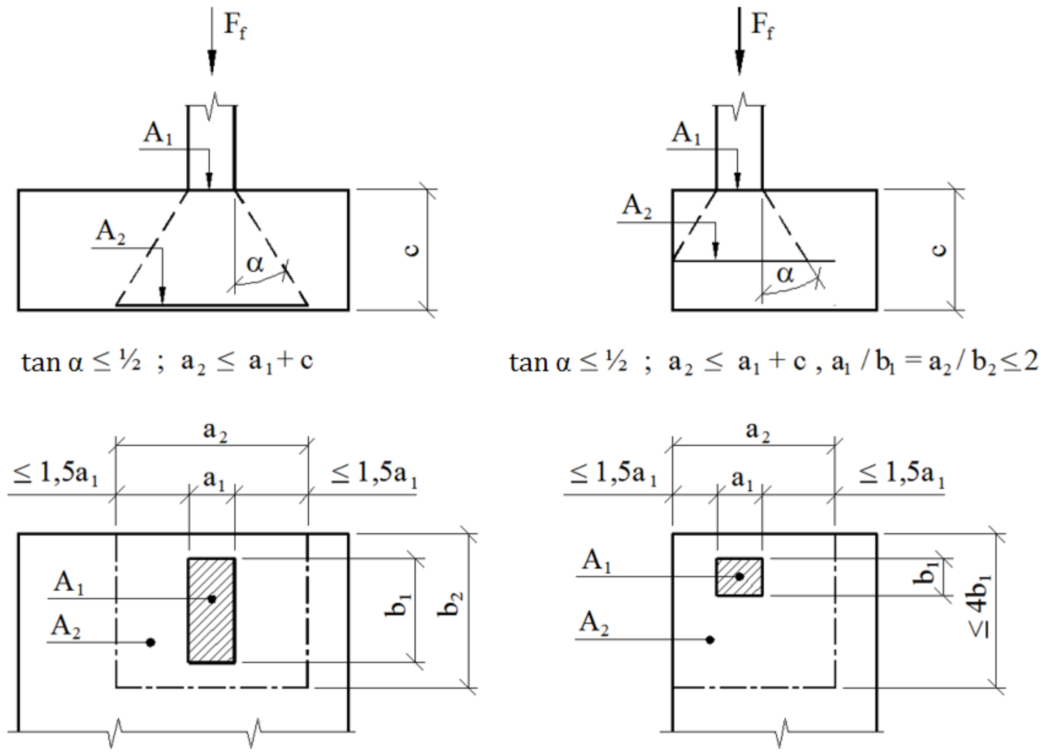


Figure 6-15 Geometrical limitations for partial loaded areas

6.12.1.8 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:

$$F_{cd} = A_1 \cdot f_{cd} \sqrt{\frac{A_2}{A_1}} \leq 3 \cdot A_1 \cdot f_{cd}$$

The compressive capacity for lightweight aggregate concrete may be taken as:

$$F_{cd} = A_1 \cdot f_{cd} \sqrt[3]{\frac{A_2}{A_1}} \leq 2 \cdot A_1 \cdot f_{cd}$$

see Figure 6-15.

6.12.1.9 Provisions in [6.12.1.5] to [6.12.1.8] are applicable for design in ULS. Fatigue life shall be predicted based on f_{cd} unless increased strength under fatigue loading is properly documented.

6.12.1.10 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:

$$0.25 \cdot F_f (1 - a_1/a_2) \text{ and } 0.25 \cdot F_f (1 - b_1/b_2)$$

see Figure 6-15.

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 develop caused by transverse expansion of soft supports (shims), fluid pressure or similar.

6.12.1.11 In case the transverse tensile reinforcement provision specified in [6.12.1.10] is met with FRP bars, the bars shall be designed for a tensile stress corresponding to a maximum strain. If bends are provided in the transverse tensile reinforcement, the design strength shall not exceed the bend capacity calculated according to [6.6.1.10]. In both cases, the material factor corresponding to the duration of the loading according to [6.3.1.8] shall be considered.

Guidance note:

A recommended value for maximum strain to be utilized is 4‰.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.12.1.12 It shall be demonstrated that forces caused by bent reinforcement are resisted. If no reinforcement is provided for transverse tension normal to the plane of the bent reinforcement, the reinforcement shall not be bent around a mandrel diameter less than determined by the equation:

$$D = \frac{0.4 \cdot \phi^2 \cdot \sigma_s}{s \cdot f_{td}} \left(1 - \frac{\phi}{s}\right) \cos\left(\frac{\beta}{2}\right)$$

where β is the opening angle of the bend.

Here is s the spacing of the reinforcement bars. For reinforcement near the free surface parallel 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.

If it is necessary to provide reinforcement for transverse tension, the total area of this reinforcement shall be at least 40% of the area of the bent bar. The transverse reinforcement shall consist of at least 2 bars placed within the curve of the bend. Transverse reinforcement may be omitted provided there are compressive stresses at least equal to f_{td} normal to the plane of the bent bar.

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:

$$D = \phi \sqrt{\frac{\phi}{s} \frac{\sigma_s}{f_{cd}}} \quad \text{for normal density concrete}$$

and

$$D = 1.5\phi \sqrt{\frac{\phi}{s} \frac{\sigma_s}{f_{cd}}} \quad \text{for lightweight aggregate concrete}$$

In this calculation, s shall not exceed 4 · ϕ.

For requirements to the mandrel diameter, see also [6.17.4].

It is not necessary to check that stirrups made in accordance with [6.17.4.8] are in accordance with the provisions of this clause.

The above requirements to minimum bending diameters are not relevant for FRP bars because these bars are formed during the production process. It shall however be demonstrated that forces caused by FRP reinforcement with bends is resisted.

6.13 Fatigue limit state

6.13.1 General

6.13.1.1 The entire stress history imposed during the life of the structure that is significant with respect to safe service life evaluation shall be taken into account when determining the long term distribution of stress cycles (see [5.5.20]).

6.13.1.2 The random nature of the loads shall be accounted for in determination of the long term distribution of stresses. Both the variation of stress ranges and mean stresses and durations shall be considered. The method of analysis shall be documented.

6.13.1.3 The effects of significant dynamic response shall be properly accounted for when determining stress ranges. Special care shall be taken to adequately determine the stress ranges in structures or members excited in the resonance range. The amount of damping assumed shall be appropriate to the design.

6.13.1.4 The geometrical layout of the structural elements and reinforcement shall be such as to minimize the possibility of fatigue failure.

6.13.1.5 Fatigue design may alternatively be undertaken utilizing 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.

6.13.1.6 For structures subject to multiple stress cycles, it shall be demonstrated that the structure will endure the expected stresses during the required design life.

6.13.1.7 Calculation of design life at varying stress amplitudes and/or mean stress should be based on cumulative linear damage theory. The stresses due to cyclic actions may be arranged in stress blocks. Each stress block should be defined by the peak stress and trough stress and a corresponding number of stress cycles. A minimum of 10 blocks is recommended. Blocks shall be evenly distributed so that each block provides a significant contribution to the total damage ratio.

6.13.1.8 If the random nature of the loads implies that the stress ranges, mean stress and durations vary, a linear damage accumulation law may be assumed:

$$D = \sum_{i=1}^{k} n_i / N_i \leq \eta$$

where k is the number of stress blocks used (≥10), n_i is the number of cycles in stress block i. and N_i is the number of uniform cycles with the same mean value and stress range which causes failure.

6.13.1.9 The characteristic fatigue strength or resistance (S-N curve) of a structural detail shall be applicable for the material, structural detail, state of stress considered and the surrounding environment. S-N curves shall take into account any relevant material thickness effects. Such S-N curves shall be documented. Alternatively, S-N curves for concrete, steel and FRP may be used together with the stresses obtained from analysis provided that these are calculated based on criteria outlined in [5.5].

6.13.1.10 Fatigue strength relationships (S-N curves) for concrete shall 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, wet, submerged).

6.13.1.11 The limit for the cumulative damage ratio (η) to be used in the design shall depend on the access for inspection and repair regardless of whether this is planned or not. Limits for cumulative damage ratios according to Table 6-7 are normally acceptable for concrete and steel reinforcement.

Table 6-7 Limit of cumulative damage ratios (η)

No access for inspection and repair	*Below or in the splash zone*[1)]	*Above splash zone* [2)]
0.33	0.5	1.0
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 limit for the cumulative damage ratios shall be reduced to 0.33. 2) Where inspection or repair is not possible the limit for the cumulative damage ratio for reinforcement above the splash zone is 0.5.		

6.13.1.12 The action effects shall be calculated according to the theory of elasticity.

6.13.1.13 The capacity may be assumed to be adequate, when calculated design life for the largest acting amplitude corresponds to at least 2.0 × 10^6 cycles if the fatigue loading is caused by randomly variable actions such as wind, waves, traffic etc.

6.13.1.14 For FRP reinforced concrete structure with a uniform load history (constant mean and stress range), the limit for the damage ratio (η = n/N) to be used in design is 0.33.

6.13.1.15 For FRP reinforced concrete structures with a non-uniform load history, if this cumulative damage theory is used, the damage ratio (η) to be used in design is 0.03. The permitted cumulative damage ratio due to exposure to variable loading is specified to account for uncertainty in the damage accumulation model and degradation of residual strength towards the end of the lifetime.

6.13.2 Fatigue strength, design life

6.13.2.1 The design life of concrete and grout subjected to cyclic stresses may be calculated from:

$$\log_{10} N = C_1 \frac{\left(1 - \frac{\sigma_{\max}}{C_5 \cdot f_{rd}}\right)}{\left(1 - \frac{\sigma_{\min}}{C_5 \cdot f_{rd}}\right)}$$

where:

f_{rd} = the compression strength for the type of failure in question
σ_{max} = the numerically largest compressive stress, calculated as the average value within each stress-block

σ_{min} = the numerically least compressive stress, calculated as the average value within each stress-block

C_5 = fatigue strength parameter. For concrete, C_5 shall be taken equal to 1.0. For grout, C_5 shall be determined by testing.

Guidance note:

In the absence of fatigue tests for grout, C_5 may be taken as 0.8.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

When σ_{min}, is tension, it shall be taken as zero when calculating the design life.

The factor C_l 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:

$$X = \frac{C_1}{1 - \frac{\sigma_{min}}{C_5 \cdot f_{rd}} + 0.1 \cdot C_1}$$

then the design life may be increased further by multiplying the value of log N by the factor C_2 where this is taken as:

$$C_2 = (1 + 0.2 (\log_{10} N - X)) > 1.0$$

6.13.2.2 The design life of steel 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 σ_{max} in the reinforcement shall be less than f_{sk}/γ_s, where γ_s is taken from Table 6-1.

6.13.2.3 For straight steel reinforcement bars and reinforcement bars bent around a mandrel with diameter not less than specified in Table 6-17 and used in a concrete structure under exposure classes X0, XC1, XC2, XC3, XC4, XF1, XA1 and XA2, the value of C_3 = 19.6 and C_4 = 6.0 shall be used. See [6.15.2] for exposure class definitions.

For reinforcement bent around a mandrel of diameter as specified in Table 6-16 and used in a structure under exposure class X0, XC1, XC2, XC3, XC4, XF1, XA1 and XA2, the value of C_3 = 15.9 and C_4 = 4.8 shall be used. See [6.15.2] for exposure class definitions.

For intermediate bending diameters to those specified in Table 6-16 and Table 6-17, interpolated values may be used. Infinite fatigue life may be assumed if the calculated value of N is greater than $2 \cdot 10^8$ cycles.

6.13.2.4 Values of C_3 and C_4 for straight steel bars in a concrete structure under exposure class XD1, XD2, XD3, XS1, XS2, XS3, XF2, XF3, XF4, XA3 and XSA are suggested in Table 6-8. For straight reinforcement bars in a concrete structure exposed to specially or severely aggressive environment, which are not included in the previous list, the influence of corrosion on the fatigue properties shall be assessed separately. See [6.15.2] for exposure class definitions.

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 [6.13.2.3].

Table 6-8 Coefficients C_3 and C_4 for calculation of fatigue life in straight bars

	Level of stress variations [MPa]		
	Δσ > 235	*235 > Δσ > 65*	*65 > Δσ > 40*
C_3	15.7	13.35	16.97
C_4	4.5	3.5	5.5

6.13.2.5 The characteristic long-term performance of FRP reinforcement shall be established from relevant tests with cyclic and constant sustained loading covering the relevant stress ranges, mean stresses and load durations according to App.F.

6.13.2.6 For FRP reinforcement a safe service life equation of the following form is used:

$$\log(N) = C\left(\frac{1-\dfrac{\sigma_{peak}}{f_F/\gamma_{F,ssa}}}{1-\dfrac{\sigma_{trough}}{f_F/\gamma_{F,ssa}}}\right)$$

where σ_{peak} is the peak stress of the stress cycle, σ_{trough} is the stress at the trough of the stress cycle and f_F is the characteristic tensile strength of the bar. The material factor $\gamma_{F,ssa}$ accounts for the duration of the loading. The coefficient C is a material dependent coefficient determined from cyclic fatigue tests to obtain a characteristic low curve.

6.13.2.7 In design, the load duration used in the damage accumulation shall not be taken less than 5 years in each stress block for FRP reinforcement. See [6.3.1.9].

6.13.2.8 Prestressed FRP reinforcement shall be checked for safe service life using the formulation in [6.13.2.6] above for non-prestressed FRP reinforcement.

6.13.3 Bending moment and axial force

6.13.3.1 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 calculating stresses.

For concrete subject to compression, f_{rd} is taken equal f_{cd}.

6.13.3.2 If a more accurate calculation is not performed, stresses in concrete and reinforcement may be calculated with a linear stress distribution in the compression zone. The calculations may be based on a Young's modulus equal to 0.8 E_{cn} for the concrete.

In such a calculation the reference strength, f_{rd} of the concrete in compression may be taken as:

$$f_{rd} = \alpha \cdot f_{cd}$$

The value of α may be calculated as $\alpha = 1.3 - 0.3\,\beta > 1.0$

where:

β = the ratio between the numerically smallest and largest stresses acting simultaneously in the local compressive concrete zone. The distance between the points used when calculating β shall not exceed 300 mm ($0 < \beta < 1.0$).

6.13.3.3 For FRP reinforcement, the stress level in the concrete defined in [6.13.3.2], shall be calculated using the design Young's modulus of FRP reinforcement, E_{Fd}. The stress level in the FRP shall be determined based on cracked sections and stress strain curves for concrete as given in [6.13.3.2]. For the FRP reinforcement a linear stress-strain curve shall be applied in the calculations.

6.13.4 Shear force

6.13.4.1 The design life at tensile failure of concrete without shear reinforcement should be calculated in accordance with [6.13.2.1].

σ_{max}/f_{rd} shall be replaced by V_{max}/V_{cd}.

σ_{min}/f_{rd} shall be replaced by V_{min}/V_{cd}.

V_{cd} shall be calculated in accordance with [6.6.2].

The factor C_l 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.

6.13.4.2 For those stress-blocks where shear forces changes sign, the denominator in the formula for log N in [6.13.2.1] shall be replaced by:

$$1 + V_{min}/V_{cd}$$

If the shear force changes sign the calculation shall, if necessary, be performed with both the positive and negative values for V_{max} and V_{min} respectively in the formulas above.

V_{cd} shall be calculated in accordance with [6.6.2].

The factor C_l shall be taken as in [6.13.4.1].

6.13.4.3 The design life at tensile failure of concrete for structures with shear reinforcement should be calculated in accordance with [6.13.2.1] by assuming the concrete at all load levels to transfer a portion of the acting shear force equal to the ratio 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 to be carried by the shear reinforcement. The design life of the concrete at tensile shear failure shall be demonstrated in accordance with [6.13.1].

6.13.4.4 The design life of the shear reinforcement should be calculated, in accordance with [6.13.2.2] to [6.13.2.4] for steel reinforcement and [6.13.2.5] to [6.13.2.6] for FRP reinforcement, by assuming the shear reinforcement at all load levels to transfer a portion of the acting shear force equal to the ratio 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°.

6.13.4.5 If the shear force changes sign, account of this shall be made when calculating the number of stress cycles in the shear reinforcement.

6.13.4.6 The design life at compression failure of concrete should be calculated in accordance with [6.13.2.1].

σ_{max}/f_{rd} shall be replaced by V_{max}/V_{ccd} .

σ_{min}/f_{rd} shall be replaced by V_{min}/V_{ccd} .

For those stress-blocks where the shear force changes sign, use V_{min} = 0.

V_{ccd} shall be calculated in accordance with [6.6.2.6].

The factor C_l shall be entered with the values given in [6.13.4.1].

6.13.4.7 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 0.9 · d. The reference stress of the concrete, f_{rd} shall be taken as f_{cd}.

6.13.5 Anchorage and splicing

6.13.5.1 Demonstration of the design life for force development should be performed in accordance with 6.13.2.1.

σ_{max}/f_{rd} shall be replaced by τ_{bmax}/f_{bd}

σ_{min}/f_{rd} shall be replaced by τ_{bmin}/f_{bd}

The bond strength, f_{bd} shall be calculated in accordance with [6.11.1.16].

The bond stress, τ_b shall be taken as:

$$\tau_b = 0.25 \cdot \phi \cdot \sigma_s / l_b$$

6.13.5.2 For structures in air C_l shall be 12.0, for structures in water C_l 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.

6.14 Accidental limit state

6.14.1 General

6.14.1.1 Structural calculations for an accidental limit state shall document the capacity of the structure. The calculations should be performed according to this clause and [6.4], [6.5], [6.6], [6.7], [6.8], [6.9], [6.10], [6.11], [6.12] and [6.16].

6.14.1.2 The material coefficients are given in [6.3.1].

6.14.1.3 Strength and strain properties are as given in [6.3.1] to [6.3.4]. The strain limits ε_{cu} and ε_{su} may however be given particular assessment.

6.14.1.4 Structures in safety classes 2 and 3 (see [2.1.3]) shall be designed in such a way that an accidental load will not cause extensive failure.

6.14.1.5 The design may permit local damage and displacements exceeding those which are normally assumed by design in the ultimate limit state. Structural models and load transferring mechanisms which are normally not permitted may be assumed. The capacity in the accidental limit state shall be documented in the following two steps:

- Resistance to abnormal actions: Accidental actions and other abnormal actions shall only cause local failure.
- Resistance in damaged condition: The structure with a local damage shall resist actual actions until repair or other precautions are taken to prevent a total loss or loss of critical serviceability requirements, e.g. oil containment..

6.14.1.6 Local regulations may require that the structure have a certain degree of serviceability after an accidental load, e.g. in case decommissioning/refloating is required at the end of the service life. In this case, it shall be demonstrated that the damage caused by the accidental load does not result in the structure losing the required degree of serviceability or that the damage can be repaired in order to regain the serviceability needed.

6.14.2 Explosion and impact

6.14.2.1 For explosion loads and impact type loads, an increased Young's modulus 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.

6.14.2.2 The structural calculations may take account of the load variation with time and the dynamic properties of the structure.

6.14.3 Fire

6.14.3.1 Required fire resistance is determined in one of the following ways:

- An offshore structure shall be designed to resist a fire in accordance with the requirements of DNVGL-OS-A101, if no other requirements for the actual structure are provided from national building dode or other national regulations
- For structures where the national building regulations give requirements to fire resistance as a function of fire loading, the fire loading is calculated, and the required fire resistance is determined in accordance with the national building dode
- Necessary fire resistance may 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 dode.

6.14.3.2 The adequacy of the fire resistance shall be documented. One of the following methods may be used:

- calculation in accordance with [6.14.3.3]
- use of other Internationally accepted methods
- testing in accordance with an accepted international standard.

6.14.3.3 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.

To verify adequate fire resistance against hydrocarbon fires in the structure, relevant temperature-time curves for hydrocarbon fires shall be used.

The strength properties of the materials as a function of the temperature are as given in [4.3.3.10] for concrete and [6.3.4.49 for steel reinforcement. Special strength properties shall be applied for concrete exposed to temperatures down to cryogenic temperature. Reference is made to DNVGL-ST-C503.

The Young's modulus of the concrete is given in [5.5.2.7]. The strain properties of the steel reinforcement are as given in [5.5.2.9].

A stress-strain diagram similar to that applicable for the ultimate limit state, with the stress ordinate reduced, should 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.

The strength properties of FRP as a function of temperature shall be derived by testing.

6.14.3.4 The structure shall be so detailed that it maintains the required load bearing ability for the required period. A geometrical configuration 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.

6.14.3.5 The temperature insulation ability and gas tightness of partitioning structures shall be demonstrated in the accidental limit state of fire.

6.15 Serviceability limit state

6.15.1 General

6.15.1.1 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.

6.15.1.2 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 shall 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 6-10.

6.15.1.3 When it is necessary to ensure tightness of compartments against leakage due to external/internal pressure difference, the concrete section shall be designed with a permanent compression zone, see [6.15.6].

6.15.1.4 Concrete structures shall 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.

6.15.1.5 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.

6.15.2 Durability

6.15.2.1 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.

6.15.2.2 Concrete structures/elements shall be classified in exposure classes according to Table 6-9. Exposure classes are related to the environmental conditions in accordance with EN 206.

Table 6-9 Exposure classes related to environmental conditions in accordance with EN 206

Class designation	*Description of the environment*	*Informative examples where exposure classes may occur*
1. No risk of corrosion attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete exposed to very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Concrete components exposed to spray containing chlorides
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spay zones	Parts of marine structures
5 Freeze/thaw attack		
XF1	Moderate water saturation without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation, with de-icing agents or seawater	Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing

Class designation	*Description of the environment*	*Informative examples where exposure classes may occur*
6 Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206, Table 2	Concrete exposed to natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206, Table 2	Concrete exposed to natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206, Table 2	Concrete exposed to natural soils and ground water
7 Special aggressive environment		
XSA	Structures exposed to strong chemical attack which are not covered by the other classes and will require additional protective measures	Structures exposed to fluids with low pH-value

For structures of exposure class XSA, 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 XS3 shall be fulfilled.

6.15.3 Crack width limitations

6.15.3.1 When calculating crack widths for comparison with the values in Table 6-10, 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.

6.15.3.2 If more accurate values are not known for short-term but frequently repeated actions such as wind, traffic and wave actions, 50% of the characteristic load as defined in Sec.5, 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.

6.15.3.3 In order to protect the steel reinforcement against corrosion and to ensure the structural performance, the reinforcement shall have a minimum concrete cover as given in [6.17.2] and the nominal characteristic crack widths calculated in accordance with [6.15.8] shall be limited as given in Table 6-10.

Table 6-10 Limiting values of nominal characteristic crack width, w_k

Exposure class	*Reinforcement sensitive to corrosion w_k*	*Reinforcement slightly sensitive to corrosion w_k*
XSA	Special consideration	Special considerations
XD1, XD2, XD3, XS1, XS3, XF2, XF3, XF4, XA3	0.20 mm	0.30 mm
XC1, XC2, XC3, XC4, XS2, XF1, XA1, XA2	0.20 mm	0.40 mm
X0	0.40 mm	-

6.15.3.4 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 should be considered as slightly sensitive to corrosion.

6.15.3.5 For structures reinforced with steel reinforcement which are permanently submerged in saline water, the crack width requirements given for exposure class XS2 in Table 6-10 apply. Exceptions are structures with water on one side and air on the opposite side, for which the requirements for XS3 apply on the air side.

6.15.3.6 For structures reinforced with steel reinforcement the crack width limitations given in Table 6-10 are related to the crack width at a distance from the reinforcement corresponding to the minimum concrete cover in accordance with Table 6-15.

When the concrete cover is larger, the nominal crack width when comparing with the values in Table 6-10 may be taken as:

$$w_{1k} = w_{ok} \cdot \frac{c_1}{c_2} > 0.7 \cdot w_{ok}$$

where:

w_{ok} = crack width calculated in accordance with [6.15.8]
c_1 = minimum concrete cover, see Table 6-15
c_2 = actual nominal concrete cover.

6.15.3.7 If reinforcement sensitive to corrosion is placed on the inside of reinforcement slightly sensitive to corrosion and with larger concrete cover than the minimum requirement, the nominal crack width when comparing with the requirements for corrosion sensitive reinforcement in Table 6-10 may be taken as:

$$W_{2k} = w_{1k} \cdot \varepsilon_{s2}/\varepsilon_{s1}$$

where:

ε_{s1} = tensile strain in reinforcement slightly sensitive to corrosion on the side with highest strain
ε_{s2} = tensile strain at the level of the reinforcement sensitive to corrosion.

6.15.3.8 For cross-sections with reinforcement sensitive to corrosion the crack limitation requirements do also apply for cracks parallel to this reinforcement.

6.15.3.9 For short periods in the construction phase, the crack width limitation for structures reinforced with steel reenforcement given in Table 6-10 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.

6.15.3.10 The strain in the reinforcement in structures reinforced with steel reenforcement shall not exceed 90% of the yield strain for 100% of characteristic loads (γ_f = 1.0 for all loads), including moments.

6.15.3.11 Crack width calculation may be avoided when the strain in the FRP reinforcement is limited to 4‰ under SLS loading for structures where the size of the crack is critical. Likewise, crack width calculations may be ignored for structures where the strain in the FRP reinforcement is less than 6‰ and the size of the crack width is not critical.

6.15.3.12 Although no specific crack width requirement is specified for FRP reinforcement due to durability considerations, the crack width shall be limited due to considerations based on appearance. This may vary based on application like offshore structures, foundations, water tight structures, oil containment structures, etc.

Guidance note:

For structures in which the concrete surface is visible w_k < 0.5 mm.

For structures not visible w_k < 0.8 mm.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

See [6.15.6] for special crack width requirements in order to ensure tightness against leakage of fluid.

6.15.3.13 For FRP reinforcement, crack width shall be calculated based on SLS loading conditions and account shall be taken of the actual concrete cover and spacing between the reinforcement.

6.15.4 Displacements

6.15.4.1 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.

6.15.4.2 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.

6.15.4.3 Action effects when calculating displacements shall be determined by use of actions and load factors in accordance with [5.4.1]. Effect of pre-stressing forces shall be taken into account in accordance with [5.3].

When calculating long-term displacements, the variation of the variable actions with time may be taken into account.

6.15.5 Vibrations

6.15.5.1 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.

6.15.6 Tightness against leakages of fluids

6.15.6.1 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
— member shape and dimensions shall be chosen so that proper placing of the concrete is possible.

6.15.6.2 Members subjected to an external/internal hydrostatic pressure difference shall be designed with a permanent compression zone not less than the larger of:

— $0.25 \cdot h$
— values as given in Table 6-11.

Table 6-11 Depth of compression zone versus pressure difference

Pressure difference [kPa]	*Depth of compression zone [mm]*
< 150	100
> 150	200

The above applies for the operating design condition using ULS combination b) (see [5.4.1]) except that a load coefficient of 0.5 is used instead of 1.3 for the environmental load (E).

6.15.6.3 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) (see [5.4.1]) 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.

6.15.6.4 In structures where requirements to tightness against leakages are specified, the reinforcement shall meet special requirements for minimum reinforcement, see [6.17.7.5] and [6.17.11.2].

6.15.7 Tightness against leakage of gas

6.15.7.1 Concrete is not gas tight and special measures shall be taken to ensure gas tight concrete structures, when this is required.

6.15.8 Crack width calculation

6.15.8.1 For structures reinforced with steel reenforcement the concrete may be considered as uncracked if the principal tensile stress σ_1 does not exceed f_{tn}/k_1.

With combined axial tensile force and bending moment the following condition applies:

$$\left(k_w \sigma_N + \sigma_M\right) < \frac{k_w f_{tn}}{k_1}$$

With combined axial compression force and bending moment the following condition applies:

$$\left(\sigma_N + \sigma_M\right) < \frac{k_w f_{tn}}{k_1}$$

where:

σ_N = stress due to axial force (tension positive)
σ_M = edge stress due to bending alone (tension positive)
f_{tn} = normalized structural tensile strength of concrete (Table 6-1 and Table 6-2)
k_1 = constant used in calculations of crack width (Table 6-12)
k_w = coefficient dependent on cross-sectional height, h
k_w = $1.5 - h/h_1 \geq 1.0$, where $h_1 = 1.0$ m.

Table 6-12 Values of constant parameter k_1

Exposure class	*Corrosion sensitive reinforcement*	*None corrosion sensitive reinforcement*
XSA	Special consideration	Special consideration
XD1, XD2, XD3, XS1, XS2, XS3, XF2, XF3, XF4, XA3	2.0	1.5

Exposure class	*Corrosion sensitive reinforcement*	*None corrosion sensitive reinforcement*
XC1, XC2, XC3, XC4, XF1, XA1, XA2	1.5	1.0
X0	1.0	1.0

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" should be used.

Stresses caused by temperatures, creep, shrinkage, deformations etc. shall be included in the evaluation provided the crack width is influenced by these parameters.

If a high predicted cracking load (cracking moment) is non-conservative, then f_{tk} shall be used in the calculations and k_1 shall be taken as 1.0.

6.15.8.2 The characteristic crack width of a steel reinforced concrete member exposed to tensile forces and shrinkage of concrete should 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

ε_{sm} = the mean principal tensile strain in the reinforcement in the crack's influence length at the outer layer of the reinforcement

ε_{cm} = mean stress dependent tensile strain in the concrete at the same layer and over the same length as ε_{sm}

ε_{cs} = the free shrinkage strain of the concrete (negative value).

The crack widths may be calculated using the methods outlined in App.E.

6.15.8.3 If no documentation of the characteristic crack widths is performed in accordance with [6.15.8.2], then the requirements for limitation of crack widths may be considered as satisfied if the actual stresses in the steel reinforcement do not exceed the values in Table 6-13.

Table 6-13 Stress limitations for simplified documentation of satisfactory state of cracking

Nominal characteristic crack width	*Type of load effect*	*Stress in reinforcement [MPa]*				
		Spacing between the bars or bundles of bars [mm]				
		100 mm	*150 mm*	*200 mm*	*250 mm*	*300 mm*
W_k = 0.4 mm	Bending	360 MPa	320 MPa	280 MPa	240 MPa	200 MPa
	Tension	300 MPa	230 MPa	210 MPa	200 MPa	190 MPa
W_k = 0.2 mm	Bending	240 MPa	200 MPa	160 MPa	120 MPa	100 MPa
	Tension	160 MPa	150 MPa	130 MPa	110 MPa	100 MPa

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.

6.15.8.4 When structures are exposed to water pressure of magnitude sufficient to influence the calculated steel reinforcement stress level or the crack width, then the impact of the water pressure in the cracks shall be included in the calculation.

6.15.8.5 In crack width calculations for concrete structures with FRP reinforcement, the load magnitude for offshore structures may be determined based on principles provided in [6.15.3]. A guideline for prediction of the characteristic crack width in FRP reinforced structures is provided in App.E.

6.15.8.6 For concrete structures with FRP reinforcement stresses and strains caused by temperatures, creep, shrinkage, deformations etc. shall be included in the calculation provided the crack width is influenced by these parameters.

The above stresses and strains shall be included in the strain value ε_{sm} when calculating crack width for the member according to App.E.

Guidance note:

This guideline should only be used for FRP reinforced concrete structures. In cases where the structural member is reinforced by both steel reinforcement and FRP reinforcement the crack width criteria for steel reinforcement structures apply.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.15.8.7 Crack width calculation may be avoided when the requirements in [6.15.3.11] are met.

6.15.8.8 For concrete structures with FRP reinforcement the same approach may be used for SLS conditions specified in EN1990 for design situations where no detailed crack width calculations have been carried out and there is no special requirement to limit the crack width for reasons of appearances.

It shall be noted that this approach generally for structural members with sufficient tension reinforcement will yield acceptable crack width. However, for structures with small cover and side mesh reinforcement the approach has so far shown to under-predict the crack width.

6.15.9 Limitation of stresses in prestressed structures

6.15.9.1 The stresses in the prestressed steel reinforcement shall for no combination of actions exceed 0.8 f_y, alternatively 0.8 · f_{01}.

During prestressing, however, stresses up to 0.85 · f_y alternatively 0.85 · f_{01}, 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.

6.15.9.2 The stress in the prestressed FRP reinforcement shall under no circumstances exceed 80% of the design strength of the FRP reinforcement for load combination type I as defined in [6.3.1.8].

6.15.9.3 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_{cckj} or 0.6 · f_{cck} in the serviceability limit state.

The outer compressive fibre stress shall be calculated assuming a linear distribution of stresses, presuming a cracked section, over the cross-section. f_{cckj} shall be taken as the strength of the concrete at the time when the load in question is applied. Creep and shrinkage of the concrete may be taken into account when calculating the stresses.

6.15.10 Freeze/thaw cycles

6.15.10.1 The general requirement to freeze/thaw resistance of concrete is given in [4.3.2.6]. Where appropriate the freeze/thaw resistance of the concrete shall be evaluated. This evaluation shall 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 shall be given to freeze/thaw of the concrete in the splash zone.

Special frost resistant concrete may be required based on this evaluation.

6.15.11 Temperature effects

6.15.11.1 Thermal stresses due to temperature effects shall be taken into account when relevant. Relevant material properties shall be used. Reference is made to [5.5.3].

6.15.12 Deflection prediction for fibre reinforced polymer reinforced concrete members

6.15.12.1 This section applies to the prediction of deflections in beam elements. Deflections of more complex structures need to be documented accordingly.

6.15.12.2 In predicting the long term deflection of a structural member reinforced by FRP due account shall be taken of creep effects in concrete and relaxation in FRP.

6.15.12.3 The displacement of the FRP reinforced member may be calculated from a combination of non-cracked and cracked concrete member.

6.15.12.4 For displacement due to bending, initially, the deflection is predicted for the un-cracked member with full bending stiffness up to the cracking load (f_{tn}, see Table 6-1 and Table 6-2). The deflection of the beam beyond the cracking load may be calculated using the cracked moment of inertia of the concrete beam.

Guidance note:

The deflection of FRP reinforced concrete structures in bending may be determined based on the following general principal:

1) Predict the cracking load, P_{cr}, of the structural element under investigation.
2) Calculate the deflection δ_E for the cracking load, P_{cr}, using elastic properties for concrete. Both the E modulus of concrete and FRP reinforcement should be modified to account for possible creep in concrete and relaxation in FRP.
3) Based on beam formulations, calculate the cracked section modulus for the structural element under investigation. The structural element may be composed of smaller structural element each with different cracked section modulus.
4) Calculate the deflection of the structural element, δ_{C1}, for the load in excess of the cracking load, i.e. $P - P_{cr}$.
5) Modify the predicted cracked deflection δ_{C1}, by the common reduction factor to k_{dB}.
6) The final deflection at a given point in the structural element may be predicted by the following formula:

$$\delta_{C1} = \delta_E + k_{dB} \cdot \delta_{C1}$$

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.16 Design by testing

6.16.1 General

6.16.1.1 Concrete structures should be designed either by testing or by a combination of calculation and testing. This applies to all limit states defined in [6.2.2.1].

6.16.1.2 Testing should 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 should cover all properties of the structure, or only certain properties, 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.

6.16.1.3 The rules of the standard with regard to dimensions, including the rules for detailing of reinforcement in [6.17] shall also apply to structures and parts of structures dimensioned by testing. Deviations from these rules may be undertaken, provided it is demonstrated by the test that such deviations are justified.

6.16.2 Test specimen

6.16.2.1 When determining the dimensions of the test specimen, tolerances which exceed those given in [6.3.6] worst case condition shall be taken into account. More stringent tolerances may be considered.

6.16.2.2 The test specimen may be produced with nominal dimensions if the specified tolerances are less than the requirements to [6.3.5]. If the accepted deviations have been accounted for in a conservative way, the reduced material factors in Table 6-1 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.

6.16.2.3 The effect of unintended eccentricity, inclination and curvature shall be taken into account as given in [6.1.3.1], [6.4.1.3] and [6.5.1.6] to [6.5.1.8].

6.16.2.4 When determining the material strength in the test specimen, characteristic strengths equal to those prescribed for production of the component should be aimed at.

6.16.2.5 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.

6.16.2.6 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.

6.16.2.7 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 is proved that smaller values are justifiable.

6.16.2.8 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 0.1 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.

6.16.2.9 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.

6.16.3 Design actions

6.16.3.1 The design actions shall be determined with the same load coefficients used when the capacity is determined by calculation, normally in accordance with [5.4].

6.16.3.2 The design actions shall be selected so that they are representative for the anticipated actions on the structure, if necessary through simulation.

6.16.4 Test procedure

6.16.4.1 A test procedure shall be made, see also [6.16.6].

6.16.4.2 Preparation and storing of the test specimen shall follow methods which are representative for the production of the components.

6.16.4.3 A test record shall be prepared, showing observations made during testing with indication of time and the corresponding action levels.

6.16.4.4 All test records shall be signed by the person responsible for the testing.

6.16.5 Processing of the test results

6.16.5.1 In general, the test shall comprise not less than three specimens. Characteristic value (R_k), mean value (R_m), and standard deviation (s) shall be determined. The characteristic value is calculated according to the formula:

$$R_k = R_m - w\ s$$

where:

w has the following values:

n	3	4	5	6	7	8	9	10	12	15	20	25	30	40	50	100
w	3.15	2.68	2.46	2.32	2.25	2.18	2.14	2.10	2.04	1.99	1.93	1.89	1.87	1.83	1.81	1.76

n = Number of specimens

If the ratio between lowest and highest test result is less than 0.5, the test results shall not be used. If the ratio is between 0.5 and 0.7, the lowest test value shall be used as characteristics value.

6.16.5.2 If the standard deviation is particularly high or some of the test results highly deviate from the others, the causes of this should be analysed.

6.16.5.3 The design value of the capacity is obtained by dividing the characteristic capacity with a material coefficient, which is dependent of the mode of failure for the capacity of the component as detailed in [6.16.5.4] below. The material coefficients given in [6.2.4] shall be used. The appropriate value of material coefficient shall be used dependent on how tolerances are accounted for in the design and in the test specimen.

6.16.5.4 The design value of the capacity shall be determined with the material coefficient for concrete for all modes of failure where the concrete is governing for the capacity. The design value of the capacity may be determined with the material coefficient for reinforcement, if the mode of failure is governed by the reinforcement, provided it is proved that a failure caused by failure of the concrete would not give a lower design value of the capacity.

6.16.5.5 For failure modes where the concrete and the reinforcement jointly contribute to the capacity, the material coefficient for concrete shall be used unless a more detailed examination is performed.

For FRP reinforced concrete structures, a higher material factor for FRP reinforcement shall be used. Unless a more detailed examination of the failure mode is carried out, the material coefficient of FRP for the appropriate load combination specified in [6.3.1.8], shall be applied.

6.16.5.6 If reinforced components have failures in an area where the reinforcement is insufficiently anchored, as may be the case with shear and bond failures in hollow core slab elements on short supports, the design value of the capacity for these failure modes shall be calculated with the material coefficient for unreinforced concrete, increased by 50%.

6.16.5.7 For unreinforced components a material coefficient of twice the value given in [6.3.1.2] shall be used, if the failure mode is governed by the tensile strength of the concrete. Such an increase of the material coefficient is not required for steel fibre reinforced elements if the volume of steel fibres exceeds 1% of the concrete volume.

Further, all the requirement of [6.3.6] shall be fulfilled.

6.16.5.8 If the characteristic crack width shall be determined, only highly strained areas shall be considered.

6.16.5.9 The component may be treated by areas, where each area is evaluated separately.

6.16.5.10 The characteristic value may be set equal to the highest measured value of crack width or displacement if the test does not give sufficient basis for a statistical calculation of the characteristic value.

6.16.6 Test report

6.16.6.1 The execution and the results from the test shall be recorded in a test report to be signed by the person in charge of the test.

6.16.6.2 The test report shall as a minimum comprise the following information:

a) aim of the test and the principles used for selection of testing object (specimen)
b) material parameters, such as class of concrete and reinforcement, type and properties of the aggregates, type and properties of additives
c) detailed geometry of the specimen, including reinforcement layout
d) result from the testing of materials, strength values for the concrete and reinforcement
e) preparation of the specimen (or component), identification number, dimensions, weight, curing conditions, storing and handling
f) instruments used during the test
g) actions
h) results of the test, test records
i) interpretation of the results, calculation of design values of capacities.

6.17 Rules for detailing of reinforcement

6.17.1 Positioning

6.17.1.1 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 is achieved.

The positions of ribbed bars may be designed in accordance with the given minimum spacing 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 is obtained in compliance with the specified tolerances.

6.17.1.2 Ribbed bars may be arranged in bundles. Bundles shall not consist of more than four bars including overlapping (see [6.17.3.3]). Normally, the bars shall be arranged so that the bundle has the least possible perimeter.

6.17.1.3 When using welded mesh fabric in accordance with approved international standard, two layers may be placed directly against each other.

6.17.1.4 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.

6.17.1.5 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_{max} + 5 mm.

Free distance between reinforcement bars in one layer, and between each reinforcement layer, if more than one layer is used, is dependent on the exposure class of the concrete structure. Table 6-14 shows the limitations for each exposure class. See [6.15.2] for exposure class definitions.

Table 6-14 Minimum distance between reinforcement bars with respect to exposure class

Exposure class	*Free distance between reinforcement bars in one layer*	*Free distance between each layer of reinforcement bars*
XSA	Special consideration	Special consideration
XD1, XD2, XD3, XS1, XS2, XS3, XF2, XF3, XF4, XA3	45 mm	35 mm
X0, XC1, XC2, XC3, XC4, XF1, XA1, XA2	40 mm	25 mm

In addition, the free distance between reinforcement shall normally be no less than the outer diameter of bundles or ducts.

6.17.1.6 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.

6.17.1.7 With regard to anchorage, the free distance between ribbed bars, bundles of ribbed bars or strands shall be no less than $2 \cdot \phi$ where ϕ is the nominal diameter for ribbed bars and strands or the equivalent diameter for bundles based on equivalent cross-sectional area.

At lapped splices of individual bars placed next to each other, the free distance to adjacent bars shall be no less than $1.5 \cdot \phi$.

6.17.2 Concrete cover

6.17.2.1 To ensure proper bond, the concrete cover shall be:

- at least equal to ϕ for ribbed bars and bundled bars
- at least equal to $2 \cdot \phi$ for pre-tensioned reinforcement
- at least equal to ϕ_{Duct} for post-tensioned reinforcement/tendons placed in ducts, but does not need to be more than 80mm.

Where:

ϕ = defined in 6.17.1.7

ϕ_{Duct} = outer diameter of the post tensioning duct.

In addition to the above, the requirements of the relevant national approval scheme (ETA or similar) and the requirements of the prestressing system's manufacturer shall be considered when establishing the minimum concrete cover.

All above limitations are excluding project defined tolerances.

6.17.2.2 Based on requirements to corrosion protection the concrete cover shall not be less than the values given in Table 6-15 for structures with steel reinforcement. See [6.15.2] for exposure class definitions.

Table 6-15 Minimum concrete cover due to corrosion protection

	Design lifetime 50 years		*Design lifetime 100 years*	
Exposure class	Reinforcement sensitive to corrosion	Reinforcement slightly sensitive to corrosion	Reinforcement sensitive to corrosion	Reinforcement slightly sensitive to corrosion
XSA	Special considerations	Special considerations	Special considerations	Special considerations
XS3, XF4	60 mm	50 mm	70 mm	60 mm
XD1, XD2, XD3, XS1, XS2, XF2, XF3, XF4, XA3	50 mm	40 mm	60 mm	50 mm
XC2, XC3, XC4, XF1, XA1, XA2	35 mm	25 mm	45 mm	35 mm
X0, XC1	25 mm	15 mm	35 mm	25 mm

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_{max} + 5 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.

End surfaces of tensioned reinforcement in precast elements in very aggressive environment, represented by XSA, XD1, XD2, XD3, XS1, XS2, XS3, XF2, XF3, XF4, XA3, shall be protected.

Adequate corrosion protection of the end anchorage system of post-tensioned reinforcement shall be documented for the actual exposure class.

Post-tension bars shall be placed in tight pipes injected with grout, grease, etc.

6.17.2.3 For structures reinforced with FRP bars, the minimum concrete cover to the longitudinal reinforcement shall be taken as the minimum of:

— the equivalent diameter, D_{eq}, of the group of FRP bars or
— 1.5 times diameter of the aggregate used in the concrete mix.

For bundled groups of FRP bars, the diameter of the bar group, shall be taken as the equivalent diameter based on area of FRP.

$$D_{eq} = \sqrt{\frac{4 \cdot n \cdot A_{F,BAR}}{\pi}}$$

where:

$A_{F,\,BAR}$ = area of each FRP bar
D_{eq} = equivalent diameter of group of bars
n = number of FRP bars in group.

6.17.2.4 For FRP reinforcement, concrete cover to the stirrups of beams and columns may be taken as minimum ½ the diameter of the FRP stirrup.

6.17.2.5 For structures exposed to fire, the requirement to minimum concrete cover shall additionally be determined from fire resistant requirements.

6.17.3 Splicing

6.17.3.1 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 [6.11].

6.17.3.2 At laps of tensile reinforcement, necessary development length shall at least be taken equal to the necessary development length calculated in accordance with [6.11]. Plain bars shall in addition have end hooks.

6.17.3.3 Bars and bundles that are spliced by lapping shall be in contact with each other.

Areas where a transfer of forces is required between adjacent bars, which are not placed against each other, should be designed in accordance with [6.9.1.3] and [6.9.1.4].

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 [6.11.1.8].

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.

6.17.4 Bending of steel reinforcing bars

6.17.4.1 All provisions within this subsection are valid for reinforcement that meets the requirements of [4.7]. Bending test criteria shall be in accordance with the applicable material standard or [K.1.1.3], whichever is strictest.

6.17.4.2 When designing bends in reinforcement bars due consideration shall be taken to avoid damage to the reinforcement, see [6.17.4.3], as well as local damage to the concrete at the bend during loading, see [6.17.4.4].

6.17.4.3 The minimum mandrel diameters to be used during production are given in Table 6-16, and shall not be less than 1.5 times the diameter of the test mandrel used when demonstrating the bending properties of the steel. Reinforcement shall not be bent at lower temperatures than the bending properties have been documented for, the temperature in the reinforcement shall be no less than -10°C during bending.

Table 6-16 Minimum mandrel diameters to be used during production

Reinforcement type	*Bar diameter [mm]*											
	5	*6*	*7*	*8*	*10*	*11*	*12*	*14*	*16*	*20*	*25*	*32*
B500C[a)]		16		20	25		32	40	50	80	125	160
B500B[a), b)]		20		32	40		50	63	80			
		32		40	50		63	89	100			

a) Warm rolled ribbed reinforcement produced with controlled cooling may be bent with temperatures down to 20°C below zero.

b) For reinforcement type B500B mandrel types in the upper line may be used for bending at temperatures above 0°C.

6.17.4.4 For normal bent reinforcement, to avoid failure of the concrete inside the bend of the bar, the mandrel diameter shall be sized in accordance with [6.12.1.12] or alternatively those mandrel diameters given in Table 6-17 may be used without further documentation. For stirrups and anchorage hooks, see [6.17.4.8].

Table 6-17 Permissible mandrel diameter for bending of reinforcement without documentation in accordance to [6.12.1.12]

Tensile strength of reinforcement (f_{sk}) MPa	*Bar diameter [mm]*											
	5	*6*	*7*	*8*	*10*	*11*	*12*	*14*	*16*	*20*	*25*	*32*
500	100	125	160	160	200	200	250	250	320	400	500	630

6.17.4.5 Bent reinforcement which will be straightened or re-bent 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. The mandrel diameters given in Table 6-18 should be used.

Table 6-18 Permissible mandrel diameter for bending of reinforcement which shall be rebent or straightened

Reinforcement type	*Bar diameter [mm]*										
	5	*6*	*7*	*8*	*10*	*11*	*12*	*14*	*16*	*20*	*25*
B500C		32		40	50		63	80	100	160	320
B500B		63		80	100		125	160	200		

Reinforcement which will be straightened or re-bent 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 re-bent shall not be used in structural members where the reinforcement will be subjected to fatigue.

6.17.4.6 Reinforcement bars of type Tempcore or similar shall not be heat treated when bending or straightening.

6.17.4.7 Stirrups and anchorage hooks shall be made of reinforcement of weldable quality.

6.17.4.8 Verification in accordance with [6.12.1.12] 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 spall by straightening the hook when the reinforcement bar is tensioned. The bend shall at least be 135°.

6.17.4.9 Welded reinforcement bars with welded attachments may be bent around mandrel diameters in accordance with [6.17.4.1] to [6.17.4.8] provided the distance between the start of the bend and the welding point is no less than four times the diameter of the bar.

6.17.4.10 For structures subjected to predominantly static loads, the bar may be bent at the welding point with a mandrel diameter as given in Table 6-17.

6.17.4.11 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 the wire if the weld is located on the inside.

6.17.4.12 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 prestressing - exceeding 95% of the yield stress or of the 0.1% 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.

6.17.5 Bentfirbre reinforced polymer bars

6.17.5.1 The relationship between strength of the FRP reinforcement and the bend in FRP bars is given in [6.6.1.10].

6.17.6 Minimum area of reinforcement - general

6.17.6.1 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.

6.17.6.2 In each individual case, the actual structure and state of stresses shall be taken into consideration when determining the minimum reinforcement.

6.17.6.3 For structures exposed to pressure from liquid or gas the numerical value of f_{tk} shall be replaced by (f_{tk} + 0.5 p_w) in the formulae for calculating the required amount of minimum reinforcement, where p_w is liquid or gas pressure.

6.17.6.4 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.

6.17.6.5 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.

6.17.6.6 In slabs the prestressing units shall not have larger spacing than six times the thickness of the slab.

6.17.7 Minimum area of reinforcement - slabs/plates

6.17.7.1 A structure or structural member shall be considered as a slab if the width of the cross-section is larger than or equal to 4 times the thickness.

6.17.7.2 In general, the total depth of the cross-section, h, shall be no less than L_i /35 where L_i is the distance between zero moment points.

6.17.7.3 For two-way slab systems, the smallest L_i for the two span directions shall apply, and for cantilever slabs:

$$L_i = 2 \cdot L$$

where L is the length of the cantilever.

6.17.7.4 Transverse to the main reinforcement and directly on this, a continuous minimum reinforcement shall be placed for steel reinforced members. The reinforcement shall have a total cross-sectional area equal to:

$$A_s \geq 0.25 \cdot k_w \cdot A_c \cdot \frac{f_{tk}}{f_{sk}}$$

where:

k_w = 1.5 – $h/h_1 \geq 1.0$
h = the total depth of the cross-section
h_1 = 1.0 m
f_{tk} = defined in [6.17.6.3].

At inner supports this reinforcement may be distributed with one half in the upper face and one half in the lower face.

Secondary reinforcement bars shall not be placed at center distance greater than three times the slab thickness or greater than 500 mm.

For FRP reinforced members, f_{sk} shall be replaced for the stress in the FRP reinforcement at:

— 4‰ strain for structural members sensitive to visual structural cracks or
— 6‰ strain for structural members not sensitive to visual structural cracks.

In order to assess the stress corresponding to this strain E_{Fd} shall be used.

6.17.7.5 In structures where water tightness is of importance, the minimum reinforcement should be at least twice the value given above and the spacing not more than 300 mm.

6.17.7.6 In the span and over the support 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.

6.17.7.7 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.

6.17.7.8 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.

6.17.7.9 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.

6.17.7.10 In two-ways slab systems, these rules apply for both directions of reinforcement.

6.17.7.11 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.

6.17.7.12 As for inner supports the transverse reinforcement which is calculated in accordance with [6.17.7.4] and [6.17.7.5] may be distributed with one half in the upper face and one half in the lower face.

6.17.7.13 Normally no stirrups or other types of shear reinforcement are required for slabs. In order to take the contribution of steel stirrups into account for shear capacity, the shear reinforcement shall have a cross-sectional area at least corresponding to (in mm^2/mm^2):

$$A_{sv} \geq 0.2 \cdot \frac{f_{tk}}{f_{sk}}$$

where f_{tk} is defined in [6.17.6.3].

For FRP reinforced members, f_{sk} corresponds to the stress in the FRP reinforcement at:

— 4‰ strain for structural members sensitive to visual structural cracks or
— 6‰ strain for structural members not sensitive to visual structural cracks.

In order to assess the stress corresponding to this strain E_{Fd} shall be used.

6.17.8 Minimum area of reinforcement - flat slabs

6.17.8.1 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.

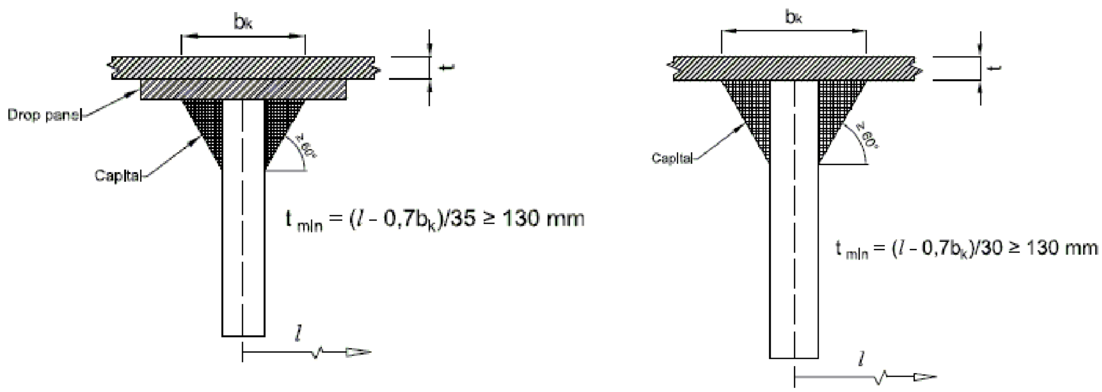


Figure 6-16 Flat slab with and without drop panel and with capital

The slab shall have a minimum thickness of:

— $(l - 0.7 \cdot b_k)/30 \geq 130$ mm for slabs without drop panel
— $(l - 0.7 \cdot b_k)/35 \geq 130$ mm for slabs with drop panel.

l 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 less than the width of the column in the span direction or larger than the value corresponding to a 60° inclination of the face of the capital to the horizontal plane.

6.17.8.2 For steel reinforced members, the slab reinforcement shall have a total cross-sectional area at least equal to:

$$A_s \geq 0.25 \cdot k_w \cdot A_c \cdot \frac{f_{tk}}{f_{sk}} \text{ , in each of the two main directions,}$$

where:

k_w is in accordance with [6.17.7.4]

f_{tk} = defined in [6.17.6.3].

For FRP reinforced members, f_{sk} corresponds to the stress in the FRP reinforcement at:

— 4‰ strain for structural members sensitive to visual structural cracks or
— 6‰ strain for structural members not sensitive to visual structural cracks.

In order to assess the stress corresponding to this strain E_{Fd} shall be used.

6.17.8.3 At the middle of the span, the spacing of bars shall not exceed 300 mm.

6.17.8.4 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.

6.17.9 Minimum area of reinforcement - beams

6.17.9.1 The cross-sectional depth h shall normally be no less than L_i /35.

L_i is the distance between points of contra-flexure. For cantilever beams, $L_i = 2 \cdot L$. and L is the length of the cantilever.

6.17.9.2 Steel reinforced rectangular beams should normally have reinforcement at the tension face, at least equal to:

$$A_s \geq 0.25 \cdot k_w \cdot b \cdot h \cdot \frac{f_{tk}}{f_{sk}}$$

where:

k_w = as given in [6.17.7.4]

f_{tk} = defined in [6.17.6.3].

At the compression side the reinforcement should not be less than half of this value, if not otherwise documented to be sufficient.

6.17.9.3 Steel reinforced 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:

$$A_s \geq A_{cf} \frac{f_{tk}}{f_{sk}}$$

where:

A_{cf} = the effective cross-section area of the flange, $h_f \cdot b_{eff}$

b_{eff} = the part of the slab width which according to [6.1.4] is assumed as effective when resisting tensile forces

h_f = the thickness of the flange (the slab)

f_{tk} = defined in [6.17.6.3].

In beams where the neutral axis is located near the flange, this quantity may be reduced to:

$$A_s \geq 0.5 \cdot h_f \cdot b_{eff} \cdot \frac{f_{tk}}{f_{sk}}$$

In flanges subjected to compression, the requirement for minimum reinforcement is:

$$A_s \geq 0.25 \cdot A_{cf} \cdot \frac{f_{tk}}{f_{sk}}$$

Longitudinal reinforcement shall always be present in the corners of the stirrups.

6.17.9.4 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.

6.17.9.5 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.

6.17.9.6 Normally stirrups shall be provided along the entire length of a beam irrespective of the magnitude of the acting shear forces. In steel reinforced members, this stirrup reinforcement shall have a cross-sectional area corresponding to:

$$A_s \geq 0.2 \cdot A_c \cdot \sin\alpha \cdot \frac{f_{tk}}{f_{sk}}$$

where:

A_c	=	the concrete area of a longitudinal section of the beam web
α	=	the angle between stirrups and the longitudinal axis of the beam. The angle shall not be taken less than 45°
f_{tk}	=	defined in [6.17.6.3].

The tensile strength f_{tk} 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 extra 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 1200 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 reinforcement, but no larger than 500 mm. In those parts of prestressed 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, see also [6.6.1].

6.17.9.7 For FRP reinforced members, the provisions of longitudinal tension and compression reinforcement in [6.17.9.2], [6.17.9.3] (web) and [6.17.9.6] (stirrups) shall be modified by replacing f_{sk} by the stress in the FRP reinforcement at:

— 4‰ strain for structural members sensitive to visual structural cracks or
— 6‰ strain for structural members not sensitive to visual structural cracks.

In order to assess the stress corresponding to this strain E_{Fd} shall be used.

6.17.9.8 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.

6.17.9.9 Compression reinforcement bars shall be braced by stirrups with spacing not exceeding 15 times the diameter of the compression reinforcement bar.

6.17.10 Minimum area of reinforcement - columns

6.17.10.1 The dimensions of columns shall be no less than:

— 40 000 mm^2 as gross cross-sectional area
— 150 mm as minimum sectional dimension for reinforced columns
— 200 mm as minimum sectional dimension for un-reinforced columns.

6.17.10.2 Steel reinforced columns shall not have less total cross-sectional area of longitudinal reinforcement than the larger of:

$$0.01 \cdot A_c \text{ and } 0.2 \cdot A_c \cdot f_{cn} / f_{sk}$$

For FRP reinforced members, f_{sk} corresponds to the stress in the FRP reinforcement at:

— 4‰ strain for structural members sensitive to visual structural cracks or
— 6‰ strain for structural members not sensitive to visual structural cracks.

6.17.10.3 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.

6.17.10.4 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.

6.17.10.5 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.

6.17.10.6 If the area of longitudinal reinforcement is larger than 2% of the cross-sectional area of the column, 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.

6.17.10.7 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.

6.17.10.8 If concrete of grade C55 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.

For FRP reinforced members, stirrups shall be FRP bars with a diameter at least equal to 10 mm. The amount of minimum stirrups (links) shall not be less than the provisions of stirrups in beams [6.17.9.6] as modified by [6.17.9.7].

6.17.10.9 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 increased stress in the core section of a spiral reinforced column shall be resisted by the foundation. Unless force transfer is documented by other means, the column spiral reinforcement shall be extended into the foundation for a height at least equal to the column core diameter.

Above requirements do not apply to FRP reinforced members. The influence of spiral FRP bars on the ductility and strength increase of columns shall be investigated separately.

6.17.11 Minimum area of reinforcement - walls

6.17.11.1 Steel reinforced walls shall have horizontal reinforcement with cross-sectional area corresponding to:

— $A_s \geq 0.6 \cdot A_c \cdot \frac{f_{tk}}{f_{sk}}$, for horizontal reinforcement in external walls
— $A_s \geq 0.3 \cdot A_c \cdot \frac{f_{tk}}{f_{sk}}$, otherwise in walls for horizontal and vertical reinforcement
— $A_s \geq 0.6 \cdot A_c \cdot \frac{f_{tk}}{f_{sk}}$, in shear walls, diaphragms and shells in both directions

where f_{tk} = defined in [6.17.6.3]

For FRP reinforced members, f_{sk} corresponds to the stress in the FRP reinforcement at:

— 4‰ strain for structural members sensitive to visual structural cracks or
— 6‰ strain for structural members not sensitive to visual structural cracks.

6.17.11.2 In structures in [6.17.11.1] where water tightness is required, the horizontal reinforcement should be at least twice the values given. The horizontal reinforcement may be reduced if the wall is free to change its length in the horizontal direction and if it is demonstrated by calculations that the chosen reinforcement resists 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.

6.17.11.3 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.

For FRP reinforced members, FRP bars shall be used instead of ribbed bars. The number and diameter of the bar shall account for the different E_F for FRP reinforcement compared with steel reinforcement.

6.17.11.4 In walls which are primarily exposed to bending caused by (local) lateral pressure load, the requirements regarding minimum reinforcement in plates in accordance with [6.17.7] shall apply.

6.17.12 Minimum area of reinforcement - reinforced foundations

6.17.12.1 Foundations shall have thickness no less than 10 times the diameter of the reinforcement bar or 200 mm, whichever is the smaller.

6.17.12.2 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.

6.17.12.3 Foundations shall be considered as beams or slabs with respect to minimum reinforcement. Reference is made to [6.17.7], [6.17.8] and [6.17.9].

6.17.13 Minimum area of reinforcement - prestressed structures

6.17.13.1 The structures shall be designed, detailed 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.

6.17.13.2 At the anchorages, the concrete dimensions shall be sufficient to ensure a satisfactory introduction and transfer of the anchorage forces. The documentation shall be based on calculations or tests for the anchorage in question.

6.17.13.3 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_{cd}, this shall be applied. The quantity of this extra reinforcement shall be documented by tests or calculations for the type of anchorage in question.

6.18 Corrosion control

6.18.1 General

6.18.1.1 This section is not applicable for structures reinforced solely by FRP reinforcement.

6.18.1.2 Requirements to corrosion protection arrangement and equipment are generally given in DNVGL-OS-C101. Special evaluations relevant for offshore concrete structures are given herein.

6.18.1.3 Fixed and floating concrete structures associated with production of oil and gas comprises permanent structural components made of carbon steel (C-steel) that require corrosion protection, both at the topsides and in the substructure. In addition, the substructure 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 [6.18].

6.18.1.4 Steel rebars and prestressing tendons shall be adequately protected by the concrete itself if:

— adequate concrete cover is provided
— due considerations have been paid to type/quality of the aggregates
— adequate crack width limitations have been set.

6.18.1.5 However, rebar portions freely exposed to seawater in case of concrete defects, embedment plates, penetration sleeves and various supports will normally require corrosion protection.

6.18.2 Corrosion zones and environmental parameters

6.18.2.1 A concrete structure will encounter different types of marine corrosion environments. These may be divided into corrosion zones as given in Table 6-19.

Some of these may not be applicable to floating structures.

Table 6-19 Corrosion zones (some may not be applicable to floating structures)

External zones	*Internal zones*
External atmospheric zone	Internal atmospheric zones
Splash zone	Intermediate zones
External submerged zone	Internal submerged zones
Buried zone	Buried zone

6.18.2.2 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/intermediate zones respectively. The buried zone includes parts of the structure buried in seabed sediments or covered by disposed solids externally or internally.

6.18.2.3 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.

6.18.2.4 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 may offer an alternative corrosion mechanism.

6.18.2.5 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.

6.18.3 Forms of corrosion and associated corrosion rates

6.18.3.1 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 may amount to 0.4 mm per year. For internally heated surfaces in the splash zone, corrosive rates are 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 attacked. Corrosion as a 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).

6.18.3.2 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 may readily cause leakage in piping by penetrating pits, or initiate fatigue cracking of components subject to cyclic loading.

6.18.3.3 Galvanic interaction (i.e. metallic plus electrolytic coupling) of C-steel to e.g. stainless steel or copper base alloys may enhance the corrosion rates given in [6.18.3.1]. 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.

6.18.3.4 Very high strength steels (f_{sk} > 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.

6.18.4 Cathodic protection

6.18.4.1 For details of design of cathodic protection systems, see DNVGL-RP-B401 *Cathodic protection design*.

6.19 Design of fibre reinforced concrete members

6.19.1 General

6.19.1.1 Short fibres are added to the concrete in small quantities to increase the tensile strength of the concrete. The fibres may be made from either steel or FRP. The amount of fibre which may effectively be added to the concrete to ensure good mixing and workability will depend on the type of fibre, its length, shape and concrete properties (slump, LWA, normal weight concrete, strength, admixtures etc.).

6.19.1.2 The properties of the fibre reinforced concrete shall be documented for the actual mix. The formulas given in this standard to determine the characteristic strength, characteristic tensile strength, Young's modulus shall be considered as guidelines only. See [4.4] for material requirements.

6.19.1.3 The impact of the increased tensile strength of fibre reinforced concrete, f_{td}, is as follows:

— [6.4] – Bending moment and axial force (ULS). Tensile strength of the fibre reinforced concrete shall not be considered in design of primary members. See [6.4.1.9] and [6.4.1.10].

— [6.6] – Shear strength. The tensile strength of the fibre reinforced concrete, f_{td}, may replace f_{td}, for concrete on its own. See [4.4.1.15] and [6.19.1.4] for documentation requirements.
— [6.8] – General design method for structural members subjected to in-plane forces. No change.
— [6.11] – Bond strength and anchorage failure – no change. The plain concrete properties are used.
— [6.15] – Serviceability limit state. No change. The crack width calculations shall be calculated based on the tensile strength of concrete, not the increased tensile strength of the fibre reinforced concrete.
— [6.16] – Design by testing. Effect of sustain loading shall be evaluated in interpretation of the short term test results.
— [6.17] – Rules for detailing of reinforcement. No change. The minimum reinforcement shall be based on f_{tk} of the concrete, not the increased tensile strength of the fibre reinforced concrete.

6.19.1.4 The impact on design of fibre reinforced concrete in accordance with this standard is by replacing the design tensile strength f_{td} in [6.6.2] by the modified f_{td} obtained for fibre reinforced concrete. It shall be documented by tests on beams that the increased shear strength predicted by the above approach actually is achieved using same concrete, type of fibres etc.

6.20 Design of structural members made of grout

6.20.1 General

6.20.1.1 Structural grout is normally used in members joining other structural members together. The connection may be of the following types:

— Type A: Steel to steel connections (e.g. tubular joints, pile sleeve connections and transition piece to monopile connections).
— Type B: Steel to concrete connections (e.g. connection of steel tubular shaft to a concrete foundation, support structure).
— Type C: Concrete to concrete connections (typically connecting concrete members using structural grout as compression/shear member in the joint).
— Type D: Connecting two precast concrete elements with in-situ cast structural grout connection.

6.20.2 Design for strength in ultimate limit state and accidental limit state

6.20.2.1 The design of the grouted connection in ULS and ALS shall be carried out by predicting the principal stress distribution in the grout, presuming the grout to be cracked when the tensile stresses exceed the tensile design strength, f_{td}, for the grout.

6.20.2.2 Assuming cracking means that an alternative load carrying mechanism shall be derived, where no tensile stresses are carried by the grout.

Guidance note:

A truss analogy in accordance with [6.6.3] describes such a method. E.g. in a tubular connection the tubular member may be considered to carry the tensile forces provided sufficient bond between the tubular steel member and the grout is documented.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.2.3 The compression capacity of the grout shall be determined based on the design compression strength, f_{cd}, as modified to f_{c2d} by relevant clauses in [6.8] and [6.9] for principal compressive stresses with perpendicular principal tensile strains.

Guidance note:

Generally, the assumption that the grout carries no tension except for shear forces (requires equations defining shear capacity for detail under design), means that the tensile forces caused by cracking have to be carried by alternative load response paths. The truss analogy is such an approach. Hence two approaches are available; either to document the shear capacity of the connection or presume that the grout carries no tension and prepare a load carrying model in accordance with the truss analogy.

It should be noted that the location where grout is applied in most cases should be considered as a region with discontinuity in geometry or loads and should be designed in accordance with [6.8] and [6.9]. Reference is especially made to the limited compression stress, f_{c2d}, which limits the principal compression strength when the principal tensile strains are acting perpendicular to the direction of the principal compression.

In the same way as a principal tensile strain reduces the compression capacity, principal compression stresses will increase the compression capacity. The maximum strength increase in biaxial compression should be 30%.

The maximum compressive strength under a triaxial state of stress is increased even more. When the equation for strength increase considers the compressive confining stresses σ_2 and σ_3, then both stresses, σ_2 and σ_3, should be equal in magnitude to obtain the full triaxial strength increase in the third direction. If one of the stresses is zero, then the state of stress becomes biaxial.

Confining pressure may result from internal stresses in the grout caused by response to external forces, by friction due to different material (load is transferred to grout through a steel plate) or by activation the tensile reinforcement in the grout member (e.g. by steel reinforcement).

Generally, confinement pressure in the grout created from tensile reinforcement should be considered a passive confinement pressure. Passive confinement pressures caused by equilibrium of stresses in the cross-section will in most cases create a principal tensile strain perpendicular (causes the tensile stress in reinforcement) to the main principal compression stress. Accordingly, the compression strength should technically be reduced for this condition in accordance with the compression field theory in [6.8].

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.3 Design for fatigue life

6.20.3.1 The design for fatigue life of the structural member made of grout (plain or fibre reinforced) shall be carried out in accordance with the general provisions in [6.13].

6.20.3.2 The design fatigue strength of the grout shall be derived as specified in [6.13.2.1]. The factor C_5 defining the design Wöhler curve for the grout in [6.13.2.1] shall be derived by experimental testing of the actual structural grout, unless the provision in the guidance note to [6.13.2.1] is applied. The value of C_5 shall be documented in the certificate for the grout.

Guidance note:

When the principal stress axes rotate on load reversal the stress range may, as a guideline, be calculated based on the minimum numerical stress in the same direction as the maximum principal compressive stress (numerically largest compressive stress).

Compressive stress in the formula [6.13.2] is taken as positive. When σ_{min} is tensile, then the stress is taken as zero in the Wöhler curve for the grout in [6.13.2].

It should be noted that in [6.13.2], the compression force is positive and defined as the maximum stress while the minimum stress on load reversal is defined as σ_{min} in the same direction of the max principal compression direction. If the stress on load reversal, above defined as σ_{min}, then σ_{min}, may be taken as zero.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.3.3 In regions with discontinuity in geometry or loads, i.e. in areas where [6/9] applies for design of concrete structures, the same design principal applies for grouted members. The fatigue reference strength shall be taken in accordance with [6.20.3.2].

6.20.4 FE Analyses of grouted connections

6.20.4.1 Non-linear FE analyses may be used in determining the stress situation in the grout.

Guidance note:

A non-linear FEM may differ from case to case. However, the following general principles are considered important:

— The boundary conditions in the model should be representative

— Representative boundary conditions also mean that slippage and contact element should be used to ensure that tensile stresses are not transferred beyond its tensile/friction capacity

— In order to obtain reliable design results the tensile stresses in the FEM should not exceed the design tensile strength of the grout, f_{td}. It shall be noted that material factors should be included in defining the material strength used in the FE model, when design capacity is determined by the FE analysis

— For a stress situation with combined tension-compression, the compression stress should not exceed f_{c2d} defined in Chapter [6.8] as part of the compression field theory. In non-linear FE analyses this is also covered by a comprehensive biaxial and triaxial failure envelope. The failure envelope should be realistic and shown to be so by comparing with outputs with experimental test results

— A failure envelope which considers strength increase due to biaxial and triaxial state of stress is acceptable, but the strength increase should be documented taking into account the principal stresses in the grout in the other directions. The increased strength should in general be related to f_{cd} as the basic uniaxial strength of the grout.

In most analysis, the failure occurs when the compression stress reaches the compressive strength provided tensile stresses in the grout have been transferred to adjacent steel members. If tensile failure occurs either by cracking (unable to transfer the tensile stresses to nearby steel member) or by boundary slippage, then instability of the non-linear analyses may occur suddenly. This is a general sign of failure.

Non-linear analyses may be sensitive to failure of, for example, small pieces of grout from the structural member. If such failures are encountered in the analytical FE model, then instability of the analyses will be noted. In some cases this may be the failure load and in other cases the model will still have remaining capacity, but observes instability in the iterations.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.5 Fibre reinforced grout

6.20.5.1 The design of fibre reinforced grouted members may follow the principles described in [6.22], [6.23] and [6.24] except that the tensile strength, f_{td}, may be increased. For requirements for documentation of tensile strength of fibre reinforced grout, see [4.6.1.11].

6.20.6 Type A steel to steel connections with grout

6.20.6.1 This may describe typical pile sleeve connection or grouted connections between tubular members. The diameter change between the inner and outer tubular members with grout in between, will initiate compression stresses in the grout. The magnitude of these compression stresses depends on the diameters and thicknesses of the connecting members.

6.20.6.2 The capacity both in ULS and FLS depends on the surface roughness, the diameter of the tubular joint as well as the thickness and strength of the steel and grout elements.

Guidance note:

For a detailed design approach for the design of grouted tubular monopile connections see DNVGL-ST-0126.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.6.3 The structural connection may be designed with shear keys mounted on the tubular sections. The shear keys may be welds on both tubular members to be joined together.

Guidance note:

For a detailed design approach for the design of grouted tubular monopile connections see DNVGL-ST-0126.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.6.4 The connection shall be designed taking into account the material and geometric properties of the grout as well as those of the shear keys.

Guidance note:

The shear keys may be designed in accordance with [6.1.5]. The grout design strength f_{cd} should be in accordance with [6.3.1] as modified by [6.9.1.5] to [6.9.1.7]. The strength may be evaluated using a truss model in which the capacity is provided by principal compression stresses. The strength of the compression strut is limited by f_{c2d} as provided in [6.9.1.7] due to tensile strains perpendicular to the compressive strength under investigations.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.6.5 The grout material shall be documented in accordance with requirements in [4.5] and [4.6] and be delivered with a product certificate, see App.H.

Guidance note:

In fatigue life predictions according to [6.13], f_{rd} should be replaced with f_{c2d}. The compression stress under consideration should be computed in the main compression direction for the major load response in the joint. For simplicity no rotation of the principal axis is assumed.

The contact pressure between the shear keys, if applicable, and the grout should also be checked for fatigue life.

If the grouted connection is submerged in water, in splash zone or if rain water may accumulate in/on the connection, then pumping action may occur due to the dynamic behaviour of the structure and the joint; hence the factor C_1 for fatigue strength evaluation should be taken as 8 for submerged concrete presuming cracking.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.7 Type B steel to concrete connection

6.20.7.1 This often describes a connection in which the steel support plate of a steel structure is connected to a concrete structure. For mounting and aligning purposes, the volume between the steel flange and the concrete member is filled with structural grout to transfer the load. The layer of grout has in most cases a limited thickness. The compressive force on the steel/grout interface will be transferred through the grout and into the concrete member where the effect of partially loaded area may be taken into account. See [6.12].

6.20.7.2 The static strength in ULS of the structural grout will increase due to restrain by the steel flange; hence the design strength in ULS may be increased with a factor.

Guidance note:

The maximum restraint from the steel plate under static load may be taken as 1.2 (e.g. the ratio between compressive strength for a cube and a cylinder). The cube strength is known to be affect by the restraining effect of the steel plate. The grout is considered as unreinforced and the material factors in [6.3.1] apply.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.7.3 The strength under fatigue loading may also be affected by the friction, but the friction effects may be reduced under fatigue loading. If water can assemble and wet the grout then the factor C_1 = 10 on the Wöhler curves in [6.13.2] applies.

Guidance note:

Until more data is available, the fatigue strength of the structural grout and the fibre reinforced grout should be taken as defined in [4.5] and [4.6] with no strength increase due to confinement. The grout is considered to be unreinforced with the material factors for unreinforced grout defined in [6.3.1.2].

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.7.4 The local strength under the load application point during fatigue loading may also be affected by the load spreading according to [6.12]. The magnitude of this influence is currently not known. If water can assemble and wet the concrete then the factor C_1 = 10 on the Wöhler curves in [6.13.2] applies.

Guidance note:

Until more data is available; the increase in fatigue strength of the concrete and grout (depending on geometry of the connection) due to confinement in partially loaded areas should be limited to a factor of 1.3 and the material factors in [6.3.1.2] should apply.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.8 Type C concrete to concrete connection

6.20.8.1 This often describes a connection in which two concrete structural elements are connected together. For mounting and aligning purposes, the volume between the elements is filled with structural grout to transfer the load. The layer of grout has in most cases a limited thickness. The force through the grout will be transferred into the concrete member as a partially loaded area, see [6.12].

6.20.8.2 As the Poisson's ratio and the Young's modulus of concrete and grout are of the same order of magnitude, no additional restraint from the interface between grout and concrete shall be considered in design strength in ULS.

Guidance note:

Reinforcement perpendicular to the load action will partly restrain the concrete. This confinement is dependent on tensile strains perpendicular to the principal compression direction in order to be activated. Technically, the provisions of [6.8.1] and [6.9.1] apply for this condition. The confining action of the reinforcement and the compressive strength reductions in accordance with [6.8.1] and [6.9.1] are considered to oppose each other when transverse reinforcement perpendicular to the load direction is included; hence no strength increase. The concrete is reinforced and the material factors in [6.3.1.2] apply.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.8.3 The local strength under the load application point during fatigue loading may also be affected by the load spreading according to [6.12]. The magnitude of this influence is currently not known. If water can assemble and wet the concrete then the factor C_1 = 10 on the Wöhler curves in [6.13.2] applies.

Guidance note:

Until more data is available; the increase in fatigue strength of the concrete and grout (depending on geometry of the connection) due to confinement in partially loaded areas should be limited to a factor of 1.3 and the material factors in [6.3.1.2] should apply.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.9 Type D connecting two precast concrete elements with in-situ cast structural grout connection

6.20.9.1 This often describes a connection in which a concrete precast element is connected to another precast concrete element through an in-situ cast grout.

6.20.9.2 The grouted connection shall be reinforced by steel reinforcement from both connected precast elements.

Guidance note:

For concrete pre-cast tower structures subject to alternating bending moments, compression in the grouted connections should be maintained by the use of a post tensioning system.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.9.3 As the Poisson's ratio and Young's modulus of concrete and grout is of the same order of magnitude, no additional restraint from the interface between grout and concrete shall be considered in design strength in ULS for the grout.

Guidance note:

Reinforcement perpendicular to the load may partly restrain the concrete. This confinement is dependent on tensile strains perpendicular to the principal compression direction in order to be activated. Technically, the provisions of [6.8.1] and [6.9.1] also apply for this condition. The confining action of the reinforcement and the compressive strength reductions in accordance with [6.8.1] and [6.9.1] are considered to oppose each other when transverse reinforcement perpendicular to the load direction is included; hence no strength increase. The grout is reinforced and the material factors in [6.3.1.2] apply.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

6.20.9.4 For fatigue assessment, due consideration shall be taken of water in or on the grout surface as well as the interaction of the grout with adjacent surfaces.

Guidance note:

For design of the grout under fatigue loading no local strength increase should be implemented in the fatigue design strength. If water can assemble and wet the concrete then the factor C_1 = 10 on the Wöhler curves in [6.13.2] applies. If the interface between the grout and the concrete may be exposed to stress variations between tensile stress and compressive stress and the grout is exposed to rainwater or otherwise exposed to water which may assemble, the factor C_1= 8 on the Wöhler curves in [6.13.2] applies.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

SECTION 7 CONSTRUCTION

7.1 General

7.1.1 Application

7.1.1.1 This section applies to the fabrication and construction of reinforced and prestressed concrete structures and structural parts or assemblies in concrete or grout.

7.1.1.2 Fabrication and construction of assemblies not adequately covered by this standard shall be specially considered.

7.1.2 Codes and standards

7.1.2.1 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.

7.1.3 Scope

7.1.3.1 The requirements of this section apply to material testing, formwork, reinforcement, concrete production, concrete coating, prestressing systems and repairs during construction of concrete structures.

7.2 Definitions

7.2.1 Terms

7.2.1.1 In the context of this standard, 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.

7.2.1.2 The term site used within the context of this standard shall be defined as 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).

7.3 Documentation

7.3.1 General

7.3.1.1 As the basis for fabrication and construction activities the following documentation, as applicable, shall be approved explicitly by the designer and other relevant parties for construction:

— 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.

7.3.1.2 Assumptions made during the design of the structure influencing the fabrication and construction activities shall be documented and shall be realistic in respect of allowing a safe construction process. Design and construction risk assessments may be required to achieve this.

7.3.1.3 Relevant documentation from the fabrication and construction required for safe operation of the structure shall be readily available on the installation. Such documentation shall give sufficient information to evaluate damages and subsequent possible repairs and modifications.

7.4 Quality control - inspection, testing and corrective actions

7.4.1 General

7.4.1.1 Supervision and inspection shall ensure that the works are completed in accordance with this standard and the provisions of the project specification.

7.4.1.2 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.

7.4.2 Inspection classes

7.4.2.1 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.

7.4.2.2 The inspection class to be used shall be stated in the project specification.

7.4.2.3 Inspection class may refer to the complete structure, to certain members of the structure or to certain operations of execution.

7.4.2.4 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.

7.4.3 Inspection of materials and products

7.4.3.1 Inspection shall be witnessed and signed by a qualified department different from the production department. Note in the case of formwork such inspections may be carried out by qualified members of the temporary design discipline or specialists from within the formwork department.

7.4.3.2 The inspection of the properties of the materials and products to be used in the works shall be as given in Table 7-1.

Table 7-1 Inspection of materials and products

Subject	*Inspection class 1* *simplified*	*Inspection class 2* *normal*	*Inspection class 3* *extended*
Materials for formwork[1)]	Not required	In accordance with project specification	
Reinforcing steel	In accordance with ISO 6935 and relevant national standards		
Prestressing steel	Not applicable	In accordance with ISO 6934	
FRP reinforcement	In accordance with product or type approval certificate		
Prestressing FRP reinforcement	In accordance with product or type approval certificate		
Fresh concrete: ready mixed or site mixed	In accordance with this standard		
Other items [2)]	In accordance with project specification and this standard		
Precast elements	In accordance with this standard		
Pre-blended structural grout	In accordance with the type approval certificate		
Inspection report	Not required	In accordance with this standard	
1) Inspections may be carried out by qualified members of the temporary design discipline or specialists from the formwork department 2) Items such as embedded steel components, repair material, epoxy for coating and injection			

7.4.3.3 In addition, FRP reinforcement shall be inspected to verify that the bars show no visible signs of handling damage.

7.4.3.4 The FRP bars shall be adequately marked for identification upon arrival. The marking shall be maintained to establish traceability until actual use in the structure.

7.4.3.5 FRP reinforcement shall be stored in a manner which prevents harmful exposure to UV light and erasure of marking. Reinforcement of different grades and dimensions shall be stored separately.

7.4.4 Inspection of execution

7.4.4.1 General

Inspection of execution according to this standard shall be carried out as given in Table 7-2 unless otherwise stated in the project specification.

Table 7-2 Inspection of execution

Subject	*Inspection class 1*	*Inspection class 2*	*Inspection class 3*
Scaffolding, formwork and falsework	Random checking	Major scaffolding and formwork to be inspected before concreting	All scaffolding and formwork shall be inspected before concreting
Reinforcement (steel and FRP)	Random checking	Major reinforcement shall be inspected before concreting	All reinforcement shall be inspected before concreting
Prestressing reinforcement (steel and FRP)	N/A	All prestressing components shall be inspected before concreting, threading, stressing. Materials to be identified by appropriate documentation	
Embedded items	According to project specification		
Erection of precast elements	N/A	Prior to and at completion of erection	
Site transport and casting of concrete	Occasional checks	Basic and random inspection	Detailed inspection of entire process
Curing and finishing of concrete	Occasional checks	Occasional checks	Regular inspection
Stressing and grouting of prestressing reinforcement	N/A	Detailed inspection of entire process, including evaluation of stressing records prior to cutting permission, grouting in accordance with this standard	
Grouting of structural connections	Occasional checks	Detailed inspection of entire process	
Application of repair and coating materials	Occasional checks	Regular inspection	
As-built geometry	N/A	According to project specification	
Documentation of inspection	N/A	Required	

7.4.4.2 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.

7.4.4.3 Inspection of reinforcement

Before casting operations start, inspections according to the relevant inspection class, shall confirm that:

— The reinforcement shown on the drawings is in place, at the specified spacing.
— Reinforcement is not contaminated by oil, grease, paint or other deleterious substances.
— 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.
— No tie wire shall protrude into the cover.

After concreting, the starter bars at construction joints shall be checked to ensure that they are correctly located. For structures of inspection class 2 and 3, all FRP bars shall be inspected before concreting. Materials shall be identified by appropriate documentation as specified in [4.9].

7.4.4.4 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.

7.4.4.5 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 7-3.

The inspection class for concreting operations shall be inspection class 3, 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.

Table 7-3 Requirements for planning, inspection and documentation

Subject	*Inspection class 1*	*Inspection class 2*	*Inspection class 3*
Planning of inspection	N/A	Inspection plan, procedures and work instructions program Actions in the event of non-conformities.	
Inspection	N/A	Frequent but random inspection.	Continuous inspection of each casting.
Documentation	N/A	All planning documents. Records from all inspections. All non-conformities and corrective action reports.	

7.4.4.6 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
— onditions of element, any protruding reinforcement bars, connection details, position of the element, etc. prior to joining and execution of other completion works.

7.4.4.7 Inspection of works for grouting of structural connections.

Before operations start, inspections shall verify:

— all documentation is in place regarding required certification, execution procedures, QC procedures etc.
— the position and fixity of piping, inlets, vents, overflows with respect to that shown on the design drawings
— seals or temporary formwork are undamaged, as flexible or rigid as required, clean and correctly installed
— compartments are cleaned of unwanted material and faces against which grout will be placed are suitable to achieve the required interaction
— all piping is clear of obstruction
— contingency procedures and equipment is developed and available
— equipment is working and has been testing for functionality.

During grouting, the inspection shall include:

— conformity of the fresh grout tests, e.g. fluidity and air
— the characteristics of the equipment and of the grout
— the actual pumping speed and pressures during grouting
— the actual pumped volume
— mixing times and water dosage
— testing and/or visual verification of returning material
— actions in the event of incidents and harmful climatic conditions.

7.4.4.8 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 prior to any further work which may make remediation or further investigation impossible:

— 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.

7.5 Construction planning

7.5.1 General

7.5.1.1 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.

7.5.1.2 Procedures detailing the construction sequences, testing and inspection activities shall be prepared. Sufficient delivery of materials and storage capacity shall be ensured to accommodate the anticipated demand for any continuous period of casting.

7.5.1.3 The planning for all construction stages shall ensure that there is adequate time for the concrete to harden sufficiently to support the loads imposed.

7.5.1.4 Due consideration shall be given to access and time required for adequate survey and inspection as the construction proceeds.

7.5.1.5 Constructional operations concerning transportation and installation operations shall be detailed in special procedures prepared in accordance with the requirements given in Sec.3.

7.5.1.6 For FRP reinforced structures special care in the construction planning is required because all bars are delivered in its final shape and dimensions to the construction site. Only the straight bars should be modified at site, in this case by reducing the length. The bars shall not generally be bent, welded etc. at the construction site when installing the bars in the casting forms.

For complex structural members, special planning not normally carried out in construction should be required.

7.6 Materials and material testing

7.6.1 General

7.6.1.1 Constituent materials, reinforcement and prestressing systems used in construction, as well as fresh and hardened concrete and grout shall satisfy the relevant requirements given in Sec.4.

7.6.1.2 Testing of materials shall be performed prior to and during construction to confirm quality of the materials and to ensure that the specified properties are obtained.

7.6.1.3 Testing of materials shall be performed in accordance with the requirements of Sec.4. The testing shall be conducted with calibrated and tested instruments and equipment.

7.6.1.4 Testing at independent, recognized laboratories may be required.

7.6.1.5 Records of all performed testing shall be kept for later inclusion in the construction records.

7.6.2 Constituent materials

7.6.2.1 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.

7.6.2.2 Cement shall be delivered with works certificate (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 are referred to specific batches.

7.6.2.3 Testing of cement on site shall be performed on a random basis during the construction period. The frequency of the sampling shall be specified based on experience and shall be approved by client/verification authority prior to start of construction. The sampling shall 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.

7.6.2.4 Testing of cement is at least to be performed to establish the following properties:

— fineness
— initial and final set
— oxide composition
— mortar strength.

Testing shall be performed as specified in Sec.4, and the test results shall satisfy the requirements in Sec.4. Cement failing to meet the requirements shall not be used.

7.6.2.5 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
— petro-graphical composition and properties that may affect the durability of the concrete
— water content.

7.6.2.6 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.

7.6.2.7 Testing of aggregates shall be performed on a regular basis during the construction period. The frequency of the sampling shall be specified based on the quality and consistency of the supply as well the concrete production volume, and shall 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.

7.6.2.8 The water source(s) shall be investigated for the suitability and dependability of the water supply. The water shall not 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 shall be sufficient and dependable enough to ensure adequate supply during any foreseen extensive production period.

7.6.2.9 The quality of mixing water shall be documented by testing at intervals adjusted in each case to type of water supply (public or other) as agreed between the relevant parties.

7.6.2.10 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.

7.6.2.11 The effect of the admixtures on concrete shall be tested at intervals on site in terms of the following properties:

— 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 7, 28 and 91 days.

7.6.3 Reinforcement and prestressing system components

7.6.3.1 All reinforcement shall be delivered to the construction site with appropriate certificates confirming compliance with the specified requirements (see Sec.4). The steel shall be adequately marked for identification upon arrival. The marking shall be maintained to establish traceability until actual use in the structure.

7.6.3.2 Reinforcement shall be stored in a manner which prevents harmful corrosion and erasure of marking. Reinforcement of different grades and dimensions shall be stored separately.

7.6.3.3 Components of the prestressing system shall be delivered with appropriate certificates confirming compliance with the specified requirements (see Sec.4). The marking shall be maintained to establish traceability until actual use in the structure.

7.6.3.4 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 shall be documented not to adversely affect the bond to the grout. Alternately the cables shall be cleaned prior to use.

7.6.3.5 Regular spot checks shall be performed on site to ensure:

— proper traceability, marking and stocking of reinforcement and components of prestressing system
— that bending of bars is performed within approved diameters and temperatures.

7.6.3.6 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.

7.6.3.7 Testing of mechanical splices in reinforcement shall comprise:

— prior to construction; 3 tensile tests of the splices
— during construction; tensile tests of 1% of all splices performed.

7.6.3.8 Testing of prestressing steel shall be performed at regular intervals prior to its use. The intervals shall be part of the procedure and the result of the testing shall be documented.

7.6.3.9 Testing of components for the prestressing system and testing and calibration of stressing equipment may be required and shall be documented.

7.6.3.10 Testing of components for the FRP prestressing system shall be performed at regular intervals prior to its use. The intervals shall be part of the procedure and the testing shall be documented.

7.6.4 Production and on-site quality control testing

7.6.4.1 Prior to start of construction, the properties of the intended concrete mix shall be verified by testing of samples from a series of trial mixes. The testing and test method shall be in accordance with the requirements of Sec.4.

7.6.4.2 The following properties shall be documented:

— mix proportions and the resulting consistence, bleeding and air content
— compressive strength
— setting times and strength development
— Young's modulus in compression
— permeability of hardened concrete
— durability in accordance with the approved specification requirements
— effect of admixtures.

7.6.4.3 During production, the concrete shall be tested regularly for strength, air content, consistency, temperature and density, as given in Table 7-4.

Table 7-4 Frequency of production testing of concrete

Parameter	*Frequency*
Strength	One sample per shift, and normally not less than one sample for every commenced 100 m^3 or at least one sample per change of constituent materials or mix proportion whichever gives the largest number of samples
Air content, Temperature and consistency	Three times per shift, or whenever a strength sample is taken
Density	Once per shift

Each sample for strength testing, taken from one batch at the form after transportation, shall comprise of at least 4 test specimens unambiguously marked for identification. The collection, curing and testing shall be performed in accordance with an approved specification.

7.6.4.4 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.

7.6.4.5 The properties of a grout shall be tested through on-site quality controls at regular intervals during the production and placement of the grout.

7.6.4.6 Records shall be kept of all testing including references to mix design, date and time of sampling as well as identify sections/parts which were grouted.

7.6.4.7 The frequency of on-site QC testing of grout to be used to fill post tensioning ducts after tensioning of the strands or bars shall be as a minimum as given in Table 7-5.

Table 7-5 Frequency of QC testing for grouting of post tensioning ducts using cement grout or pre-blended grout

Parameter	*Frequency*[1), 2), 3)]
Compressive strength	Five test specimens shall be taken; once per shift, for every commenced 100 m^3 or once per change of constituent materials/mix proportion
Fluidity	Once per batch, once after each duct
Temperature	Once per fluidity test
Density	Once per sampling for strength test or every 3 hours[4)]
Expansion and bleeding	Once per sampling for strength test or every 3 hours[4)]
Grouting speed	Monitored and recorded on the first duct per shift, and at commencement of each group of similar ducts
Grouting pressure[5)]	Continuously monitored, recorded once for each duct

Notes:

1) Test frequency in this table assumes the grout is designed to transfer prestress loads or compression loads within the section. If grout is not assumed to contribute to the structural system a reduced QC regime may be developed.
2) Test frequency in this table assumes the use of a batch mixer for grout production whereby a fixed quantity of constituents is mixed with a fixed dosage of water in distinct batches. If a continuous mixer is used a suitable QC plan shall be established. Density and/or the water powder ratio of the produced grout shall be monitored continuously using suitable densitometers or water content meters. Frequent calibration of meters shall form part of the QC testing regime.
3) Where different frequencies are provided for a given parameter the one resulting in the largest number of tests shall be selected.
4) Frequency may be reduced for pre-blended pre-packed grout at a specific batching location to once per strength test if consistency with the prequalification tests can be established after the first 100 ducts, or the first month of continuous production.
5) The pressure in the duct during grouting shall be controlled by calculating the combined effect of hydrostatic and dynamic pressure exerted by the grout. A pressure gauge should be mounted as close to the inlet of the duct as practicable. A maximum dynamic pressure shall be included in the grouting procedure and the gauge pressure monitored during pumping to ensure this is not exceeded.

7.6.4.8 The frequency of on-site QC testing of grout to be used for underbase grouting shall be as a minimum as given in Table 7-6.

Table 7-6 Frequency of QC testing for underbase grouting using cement grout

Parameter	*Frequency*[1), 2),]
Compressive strength	Five test specimens shall be taken; once per shift, for every commenced 100 m^3, once per change of constituent materials/mix proportion or for every compartment grouted
Fluidity or viscosity	Once on the first batch and; once per sampling for strength tests or every 3 hours
Temperature	Once on the first batch and; once per sampling for strength tests or every 3 hours
Density	Once on the first batch and; once per sampling for strength tests or every 3 hours
Expansion and bleeding	Once per sampling for strength tests or every 3 hours
Grouting speed	Once at the start of pumping and; for every compartment grouted, after adjusting the pump rate, or every 3 hours

Parameter	Frequency[1), 2),]
Grouting pressure	Continuously monitored, recorded once at the start of pumping, for every compartment grouted, or every 3 hours

Notes:

1) Test frequency in this table assumes the use of a continuous mixer for grout production whereby cement and water is continuously fed into the mixer. Density and/or the water cement ratio of the produced grout shall be monitored continuously using suitable densitometers or water content meters. Frequent calibration of meters shall form part of the QC testing regime.
2) Where different frequencies are provided for a given parameter the one resulting in the largest number of tests shall be selected.

7.6.4.9 The frequency of on-site QC testing of grout to be used to fill grouted structural connections shall be as a minimum as given in Table 7-7.

Table 7-7 Frequency of QC testing for grouting of structural connections using pre-packed or cement based grout

Parameter	Frequency[1), 2), 3), 4)]
Compressive strength	Five test specimens shall be taken; once per shift, for every commenced 15 m^3, once per change of constituent materials / mix proportion or for every compartment grouted
Bleeding/homogeneity (visual inspection)	Once on the first batch and once per sampling for strength tests
Spread/flow	Once on the first batch and once per sampling for strength tests
Temperature	Once on the first batch and once per sampling for strength tests
Grouting speed	Once at the start of pumping, for every compartment grouted, or after adjusting the pump rate
Grouting pressure	Continuously monitored, recorded once at the start of pumping, for every compartment grouted.

Notes:

1) Test parameters and frequencies specified in this table assumes that the grout is made from a pre-blended grout mix, where the constituents are mixed together in dry powder form under a QA/QC system in a factory and transported/stored in bags or silo. For requirements for on-site QC testing of site mixed cement grout when applied on structural connections see Table 7-6 with the following modifications to the testing requirements; (i) Compressive strength sampling frequency regarding volumes shall be for every commenced 15m^3 (ii) Fluidity or viscosity testing frequency shall be once for every commenced 5m^3. A suitable QC plan shall be established. It is assumed that the mixing of cement grout for structural connections is undertaken using batch mixers, therefore note 1 in Table 7-6 is not valid for this case. If continuous mixing is used see note 2 of this table.
2) Test frequency in this table assumes the use of a batch mixer for grout production whereby a fixed quantity of constituents is mixed with a fixed dosage of water in distinct batches. If a continuous mixer is used a suitable QC plan shall be established. Density and/or the water powder ratio of the produced grout shall be monitored continuously using suitable densitometers or water content meters. Frequent calibration of meters shall form part of the QC testing regime.
3) Where different frequencies are provided for a given parameter the one resulting in the largest number of tests shall be selected.
4) This table may be taken as applicable for grouting operations including annulus grouting of cylindrical/conical connections commonly found on jacket, monopile, jetty and tri-pod foundations, piles for offshore loading systems, grouted clamps/repairs and under flange grouting.

7.6.4.10 Until uniform quality of the grout has been demonstrated, higher frequencies of testing may be required.

7.6.4.11 Testing of grout shall be performed on specimens taken from samples collected during grout production. The collection, curing and testing shall be performed in accordance with an approved specification.

7.6.4.12 Samples for testing of fresh and hardened grout shall be, whenever possible, collected from evacuation points of the compartments being grouted and the samples taken from the emerging, surplus grout.

Guidance note:

Subsea connections or connections where retrieval of specimens from the overflow material is not possible may have the QC specimens taken at the pump where measures are taken to ensure that good grout fills the entire connection. These measures may include close visual inspection of the overflow via ROV, pumping additional volumes of material or verification of density of the overflow using densomiters etc. Effectiveness of such measures should be established through testing prior to operations.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

7.6.5 Testing of concrete in the structure

7.6.5.1 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 shall be agreed upon by client/verification authority in each case. Increased examination of concrete in the structure shall be considered if one of the following conditions occurs:

— 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.

7.6.5.2 The procedures to be followed together with calibration methods and criteria for non-destructive examination shall be approved in each case.

7.6.5.3 When test results are compared, a relationship shall 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.

7.6.6 Non-cementitious materials

7.6.6.1 Non-cementitious materials are materials such as epoxies and polyurethane which are specially made for use in combination with structural concrete to either improve the concrete properties or supplement, repair or replace the concrete.

7.6.6.2 Non-cementitious materials shall be delivered with test reports specifying the composition and properties of the material. The material shall be handled and stored in accordance with the supplier's recommendations.

7.6.6.3 Non-cementitious materials shall not be used unless a careful evaluation and testing has been performed prior to their use and procedures for the use/application have been approved.

7.7 Formwork

7.7.1 Design, materials and erection

7.7.1.1 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.

7.7.1.2 Formwork shall have sufficient strength, stiffness and dimensional stability to withstand the loadings from casting, compaction and vibration of fresh concrete. When casting concrete against non-vertical and nearly vertical formwork faces the pressure from wet concrete may cause significant uplift and shall be taken into consideration. In addition the support conditions for the formwork and possible live and environmental loads prior to, during and after the casting shall be considered.

7.7.1.3 For special and critical casting operations it may be required to submit design calculations for the formwork for advance approval.

7.7.1.4 Special care shall be taken when designing formwork for concrete with long setting time where large heights of fresh concrete may exert significant loading on the formwork.

7.7.1.5 Slip-forming operations shall be described in a slip-forming 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.

7.7.1.6 Feasibility tests on site may be required for complicated slip-form operations.

7.7.1.7 Slip-forms with variable dimensions shall be specially considered.

7.7.1.8 Materials for formwork shall accommodate the requirements to strength, stiffness and low water absorption. Formwork shall be erected by experienced personnel working in accordance with detailed drawings. Wooden spacers shall not be used.

7.7.1.9 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.

7.7.1.10 The materials employed shall be consistent with any special requirements for the surface finish or later surface treatment.

7.7.1.11 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 pre-cambering, loading, unkeying, striking and dismantling.

7.7.1.12 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.

7.7.1.13 Formwork shall keep the concrete in its required shape until it is hardened.

7.7.1.14 Formwork and the joints between boards or panels shall be sufficiently tight against loss of water and fines.

7.7.1.15 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.

7.7.1.16 The internal surface of the formwork shall be clean. When slip-forming 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.

7.7.1.17 Special care shall be taken when designing formwork for structural grout or concrete with high or altered flow characteristics where the hydrostatic pressure from concrete may be more than expected from normal concrete.

7.7.1.18 Where formwork is designed to be tight, when casting with structural grout for example, special care shall be taken to connections, penetrations and fixity to the ground. If required a test using water may be considered prior to the casting operation.

7.7.2 Slip-form systems

7.7.2.1 When using the slip-forming 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 slip-form 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 slip-forming period.

7.7.2.2 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.

7.7.2.3 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 slip-forming rate and other conditions affecting the hardening process of the concrete shall be such as to reduce or eliminate the tendency for lifting cracks.

7.7.2.4 The slip-form 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 slip-form.

7.7.2.5 The concrete cover to the reinforcement shall be kept within the tolerances using sufficiently long and stiff guides between the reinforcement and the form, adequately distributed around the form.

7.7.2.6 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.

7.7.2.7 Due consideration should be given to the cleaning of slip-form panels to minimise build-up of concrete lumps. This is especially true when slip-forming at very low climbing rates is planned. The ability to replace individual panels during slip-forming where persistent lumps appear may be considered.

7.7.2.8 The temperature of the slip-form panels on both sides of the wall should be considered during planning. Measures to control the temperature to avoid excessively high temperatures or significant variation in temperatures on both sides of the wall should be provided where required , e.g. limit the effects from temperature increase in small cells, exposure to direct sunlight or strong winds. Such methods include running potable water (soaker) systems, air conditioners, hoarding, heating elements etc

7.7.3 Jump-forming systems

7.7.3.1 Jump-forming 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.

7.7.3.2 The jump-form, when installed, shall allow the necessary preparation and cleaning of construction joints. The jump-form system shall accommodate the necessary walkways and access platforms to ensure that the concreting works are performed in an appropriate manner.

7.7.4 Inserts in formwork, recesses and blockouts

7.7.4.1 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.

7.7.4.2 Any embedded item shall have sufficient strength and stiffness to preserve its shape during the concreting operation and be free of contaminates that would affect them, the concrete or the reinforcement.

7.7.4.3 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.

7.7.5 Removal of formwork and falsework

7.7.5.1 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.

7.7.5.2 Striking shall be made in a manner that will not subject the structure to overload or damage.

7.7.5.3 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.

7.7.5.4 If formwork is part of the curing system, the time of its removal shall take into account the requirements [7.10.3].

7.7.6 Surface treatment and final preparation

7.7.6.1 At completion of formwork erection and during slip-forming operations it shall 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.

7.7.6.2 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.

7.7.6.3 The surface treatment and final preparation of formwork shall be described in a special procedure.

7.7.6.4 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 not 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.

7.7.6.5 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.

7.8 Reinforcement and embedded steel

7.8.1 Reinforcement

7.8.1.1 Reinforcement shall be of the type, grade and dimensions given in the approved specification/ drawings (see also requirements in Sec.4) and shall be placed with the spacing, splices and concrete covers stated in the same documents.

7.8.1.2 The surface of the reinforcement shall be free of substances that may be harmful to the reinforcement or the bond between reinforcement and concrete at the time of installation and shall be protected from such substances until casting of concrete commences.

7.8.1.3 Steel 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.

7.8.1.4 Bending of reinforcement with temperature below 0°C shall only be performed on steel of given quality specified in Sec.4.

7.8.1.5 FRP bars may be cut to specified length but shall otherwise be used in the as delivered shapes. FRP bars shall not be bent to shape.

7.8.1.6 Welding of steel reinforcement shall 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 welds of importance. The Production tests and quality of the welding procedures shall be documented.

7.8.1.7 Steel welding is only permitted on reinforcing steel that is classified as weldable in the relevant standard as required in [4.7].

7.8.1.8 Steel welding shall be used and performed in accordance with specifications by design, and shall conform to special requirements in international standards as relevant.

7.8.1.9 Steel welding should not be executed at or near bends in a bar, unless specifically approved by the design.

7.8.1.10 Steel welding of galvanized or epoxy-coated reinforcement is only permitted when a procedure for repair is specified and approved.

7.8.1.11 For steel 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 takes without cracking or damage.

7.8.1.12 In-place bending of steel in the formwork may be allowed if it is demonstrated that the prescribed bending radius is obtained, and the work is performed without misplacing the reinforcement.

7.8.1.13 The straightening of bent steel reinforcing bars is prohibited unless the bars are originally bent over a mandrel with a diameter at least 1.5 times greater than the test mandrel used to document ductility of the particular bar. A procedure for such bar straightening work shall be prepared and approved.

7.8.1.14 Steel 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.

7.8.1.15 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 placed accurate so that they meet all the requirements regarding placing tolerances for reinforcement.

7.8.1.16 Steel deformed, high bond bars may be bundled in contact to ensure adequate concrete penetration into areas with congested reinforcement. Special attention shall 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 [6.17.3.3]), are allowed to be in the same bundle at any section.

7.8.1.17 The reinforcement shall 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.

7.8.1.18 The specified concrete cover shall be ensured by securely fixed, sturdy spacers. Wooden spacers are prohibited. No tie wire shall protrude into the cover.

7.8.1.19 Attention shall be paid to the execution and detailing of reinforcement at construction joints and the areas around prestressing anchorages.

7.8.1.20 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 for steel reinforcement 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.

7.8.1.21 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.

7.8.1.22 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.6.

7.8.1.23 Assembly of steel 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 reinforcement bar at the weld.

7.8.1.24 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 [6.17.1] and [6.17.2].

7.8.1.25 In areas of congested reinforcement, measures shall be taken to ascertain that the concrete flows and fill all voids without segregation and that it is possible to adequately compact it.

7.8.1.26 FRP reinforcement shall be handled with care. FRP bars which are damaged in storage and handling prior to, during installation and prior to casting shall be replaced.

7.8.1.27 FRP reinforcement has a density of same magnitude as that of concrete. The consequence is that the reinforcement may float up during vibration. The fixing of the FRP reinforcement shall be done considering this consequence.

7.8.2 Prestressing ducts and anchorages

7.8.2.1 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.

7.8.2.2 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 prestressing assembly or system consisting e.g. of prestressing reinforcement, 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.

7.8.2.3 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.

7.8.2.4 Documentation of prestressing steel of different deliveries shall be made in the as-built records.

7.8.2.5 Cutting shall be done by an appropriate method in a way that is not harmful.

7.8.2.6 Prestressing steel shall not be subject to welding. Steel in the vicinity of prestressing steel shall not be subject to oxygen cutting or welding except when sufficient precaution have been taken to avoid damage to prestressing steel and ducts.

7.8.2.7 The prestressing 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, compaction and vibration of concrete.

7.8.2.8 The straight entry into anchorages and couplers as well as the co-axiality of tendon and anchorage shall be as specified by the supplier's specifications or system approval documents.

7.8.2.9 Care shall 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.

7.8.2.10 Vents and drains on the sheaths shall be provided at both ends, and at all points where air or water may 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.

7.8.2.11 Inlets, vents and drains shall be properly marked to identify the cable.

7.8.2.12 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.

7.8.2.13 Sheaths shall be checked after pouring of concrete to ensure sufficient passage for the tendons.

7.8.2.14 Sheaths shall be cleared of any water immediately prior to tendon threading.

7.8.3 Embedded steel

7.8.3.1 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.

7.8.3.2 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.

7.8.3.3 Embedments shall be securely fixed at their location to prevent any accidental movement during succeeding construction stages.

7.8.3.4 Due consideration shall be given, where relevant, to heat transfer into the concrete during welding, and the corresponding effects on concrete quality, anchoring bond as well as the quality of the welding.

7.8.3.5 Adequate sealing shall 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 is demonstrated that their corrosion will not cause concrete spalling endangering the reinforcement.

7.8.4 Inspection and survey

7.8.4.1 During and after installation of reinforcement, ducts, anchorages and embedments, survey and inspection shall 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
— type and location of embedments
— compliance with installation/operation procedures.

7.9 Production of concrete and grout

7.9.1 General

7.9.1.1 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.

7.9.1.2 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.

7.9.1.3 The various mix designs shall be approved for their intended applications and the mix proportions recorded, again see Sec.4. Each approved mix design shall 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.

7.9.1.4 The lay-out, and mixing procedures to be used at the mixing plant, shall be described and approved prior to start of construction. The description shall 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.

7.9.1.5 The constituent materials shall be weighed; volumetric batching shall not be used unless adequate accuracy is documented regularly. The quantity of water used in the mixes shall be adjusted according to the water content of the aggregates.

7.9.1.6 In special cases, it may be required to maintain the temperature 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 shall be investigated, documented and approved before such methods are used.

7.9.1.7 Survey and inspection shall be performed during production of concrete and grout, and should as a minimum include:

— compliance with mix design and mixing procedures
— compliance with sampling and test intervals
— compliance with specified method statements
— review of the contractors internal QC controls for casting operations.

7.10 Transport, casting, compaction and curing of concrete

7.10.1 Transport

7.10.1.1 Transport of concrete from the mixing plant to the place of casting shall be performed in a manner that provides optimum quality concrete at the place of casting. Segregation in the fresh concrete shall be avoided, and in cases where early setting may represent a problem the maximum time allowed between emergence from the mixer and completed casting shall be specified and approved.

7.10.1.2 Rotating truck mixers shall be used for road transport from the mixing plant. Transport in a non-rotating vessel should be avoided, except for very short distances. Pumping or skips should be used for placing the concrete in the forms. Other methods for placement may also be considered.

7.10.1.3 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.

7.10.1.4 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.

7.10.1.5 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.

7.10.1.6 The maximum amount of water that may be added to the concrete during the transport shall be specified and be in accordance with the pretesting documentation.

7.10.1.7 When pumping is used for the casting of large sections, a sufficient number of back-up units shall be provided.

7.10.2 Casting and compaction

7.10.2.1 A procedure for the casting process shall 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 re-vibration
— contingency measures in case of form stop, blockage, equipment failure etc.

7.10.2.2 Before casting commences examination of the formwork, reinforcement, ducts, anchorages and embedments shall be completed with acceptable results. Immediately before placing of the concrete the formwork shall be examined for debris and foreign matters detrimental to concrete quality. The form shall be free of detritus, ice, snow and standing water.

7.10.2.3 Construction joints shall be prepared and roughened in accordance with project specifications. In monolithic structures an adequately roughened surface may be obtained by the one of the following methods:

— application of a surface retarder on the fresh concrete, and later cleaning by water jetting
— abrasive blasting or sandblasting
— high-pressure water jetting.

Construction joints shall be clean, free of laitance and thoroughly saturated with water, but surface dry. The roughened surface shall be prepared so to expose the coarse aggregate without cracking or dislodging it. 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 shall be exercised in the preparation of construction joints in sections of the structure that shall remain watertight in temporary or operational phases.

7.10.2.4 During casting care shall be exercised when placing the concrete in the forms so that accidental displacement of reinforcement, embedments etc. will not occur.

7.10.2.5 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 shall ensure thorough compaction, penetration of concrete into voids and homogeneous concrete. Concrete layer placed onto a construction joint should be well vibrated to ensure a dense concrete. Direct contact between vibrators and reinforcement shall be avoided.

7.10.2.6 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 re-vibration.

7.10.2.7 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.

7.10.2.8 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 re-vibration 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.

7.10.2.9 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 be permitted to occur unless the mix is demonstrated to allow this without segregation.

7.10.2.10 Concrete should 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.

7.10.2.11 Alternative methods to the use of internal vibrators in order to obtain an adequately compacted concrete may be permitted provided this is documented for the relevant type of conditions by trial casting.

7.10.2.12 Concrete which does not require the use of vibrators in order to obtain an adequately compacted state due to the makeup of its mix design shall have its adequacy documented prior to its specification.

7.10.2.13 Low temperature concreting may require special procedures to ensure that the concrete reaches adequate maturity. Necessary precautions to be specified in the procedures should include, where applicable:

— heating the concrete mix
— use of accelerators in the concrete mix

— heated and/or insulated formwork
— hoarding and heating of the environment
— heating of the concrete part adjacent to the material to be cast.

7.10.2.14 Hot weather concreting shall 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 forecast 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.

7.10.2.15 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.

7.10.2.16 For underwater concreting, special procedures shall be prepared and their adequacy documented.

7.10.2.17 Records shall be kept during the casting operations. Each batch shall 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.

7.10.2.18 During casting of concrete survey and inspection shall be performed to ensure compliance with the approved procedure.

7.10.2.19 Special concreting methods shall be stated in the project specification or agreed.

7.10.2.20 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.

7.10.2.21 Concrete for slip-forming shall have an appropriate setting time. Slip-forming 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.

7.10.3 Curing

7.10.3.1 Concreting procedures shall 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.

7.10.3.2 During curing the concrete surface is, as far as practicable, to be kept wet with fresh water. Care shall 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 shall be used.

7.10.3.3 Whenever there is a possibility that the concrete temperature may fall below the freezing point during curing, adequate insulation shall be provided.

7.10.3.4 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.

7.10.3.5 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.

7.10.3.6 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.

7.10.3.7 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.

7.10.3.8 The peak temperature of the concrete within an element shall not exceed 70°C unless data are documenting that higher temperatures will have no significant adverse effect.

7.10.3.9 The set concrete shall be protected from vibrations and impacts that may damage the concrete or its bond to reinforcement.

7.10.3.10 The surface shall be protected from damage by heavy rain, flowing water or other mechanical influences.

7.10.4 Completion

7.10.4.1 Formwork shall not 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.

7.10.4.2 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.

7.10.4.3 The concrete surface shall be examined and areas subject to repair marked out. If any areas show visible signs of inferior quality, the area shall be marked for possible testing of concrete quality.

7.11 Completion of prestressing systems

7.11.1 Threading and stressing of tendons

7.11.1.1 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 water tightness. All ducts shall be cleared by compressed air or similar means prior to threading of tendons.

Guidance note:

When post tensioning works are performed on a floating structure the ingress of seawater into ducts during temporary construction phases should be avoided. Prior to threading, if standing water is found in post tensioning ducts it should be tested for salinity. If required the ducts should be flushed with potable water to reduce salt concentrations to agreed levels, in line with that commonly allowed in grout for instance. Water should be removed prior to the grouting of ducts. If this is not possible a Non-Conformance Report should be created where the extent and effect are documented. Mitigating procedures should be produced.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

7.11.1.2 Tendons shall be examined for damages, corrosion, dimension and identification before they are threaded.

7.11.1.3 Stressing of tendons shall be carried out according to the system manufacturer's or other approved procedure which as a minimum shall specify:

— the sequence of stressing for multiple cables
— the number of stressing steps
— elongation versus load
— amount of overstressing to compensate for creep
— requirements to equipment.

7.11.1.4 Stressing of tendons shall be carried out by personnel with documented qualification, e.g. previous experience or adequate training.

7.11.1.5 On completion of stressing operations, protruding ends of tendons shall be protected.

7.11.1.6 The final stress in each tendon shall be recorded.

7.11.1.7 During threading and stressing of tendons, survey and inspection shall be performed to ensure compliance with the approved procedure.

7.11.2 Tensioning of tendons

7.11.2.1 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.

7.11.2.2 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.

7.11.2.3 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 minimum compressive strength stated in the approval documents of the prestressing system. Special attention in this respect shall be paid to the anchorage areas.

7.11.3 Pre-tensioning

7.11.3.1 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 is not 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.

7.11.3.2 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.

7.11.3.3 If the fresh concrete is not 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 reinforcement and/or the concrete.

7.11.3.4 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.

7.11.3.5 Only qualified methods of prestressing of FRP shall be used.

7.11.4 Post-tensioning

7.11.4.1 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.

7.11.4.2 If, during the stressing operation, the calculated elongation is not 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.

7.11.4.3 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 impair re-tensioning shall not be carried out. No tendons shall be cut if the obtained elongations deviate from the theoretical by more than the following without design approval:

— 7% for short tendons of less than 12m length
— 5% for tendons of 12m or greater length.

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 may be necessary.

---e-n-d---o-f---n-o-t-e---

7.11.4.4 The prestressing tendons shall be protected from corrosion in the period from threading to prestressing. 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.

7.11.5 Protective measures, grouting, greasing, concreting

7.11.5.1 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 grout, grease or wax. Anchorage areas and end caps shall be protected as well as the tendons. In the case of vertical ducts due consideration shall be given in design to ensure that standing water does not accumulate during temporary construction phases by providing means of drainage at the bottom wherever possible.

7.11.5.2 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. In case of a delay protective measures shall be considered to prevent corrosion in accordance with national regulations or recommendations by the supplier.

7.11.5.3 A grouting procedure 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.

7.11.5.4 Grouting devices shall be as permitted for the prestressing system.

7.11.6 Unbonded tendons

7.11.6.1 Anchorage areas of un-bonded 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.

7.11.6.2 Sheathed un-bonded tendons shall be adequately sealed against penetration of moisture at their ends.

7.11.7 Grouting of post-tensioning ducts

7.11.7.1 For general requirements to grouting operations see [7.17].

7.11.7.2 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.

7.11.7.3 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.

7.11.7.4 Provision for vacuum grouting or reinjection shall be made in case of discovery of a blockage in a post tensioning duct. Ducts shall under no circumstances be left empty and un-grouted without specific approval by design.

7.11.7.5 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.

7.11.7.6 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.

7.11.7.7 If grouting of a duct is interrupted or a duct is incompletely filled, corrective actions shall be taken. No ducts shall be left with incomplete filling of grout.

7.11.7.8 For vertical ducts, or ducts with significant elevation change along their length, due consideration shall be paid to the effect of pressure on the bleed potential of the grout material during its fluid phase. ASTM C1741 shall be included in the prequalification test programme for grout when an elevation change of 20 m or greater is present in the post-tensioning ducts in the structure, see [7.17.1.1].

7.11.8 Greasing operations

7.11.8.1 Greasing shall be carried out at continuous and steady rate. After completion of greasing, unintended loss of grease from the ducts shall be prevented by sealing them under pressure.

7.11.8.2 The volume of the injected grease shall be checked against the theoretical free volume in the duct. The change of volume of the grease with change in temperature shall be taken into account.

7.12 Repairs

7.12.1 General

7.12.1.1 Procedures for the execution of repairs shall be prepared. General procedures, appropriate for the most common types of repairs, are normally to be available at the start of construction. Further procedures shall be prepared if repairs, not covered by the initial procedures, shall be performed. The procedures are as a minimum to contain the following information:

— criteria and authority for deciding implementation of repairs
— necessary equipment
— qualification of personnel
— required ambient conditions (e.g. temperature)
— repair material specification
— repair execution description
— procedure testing
— inspection and testing.

7.12.1.2 Materials for repair during construction shall be approved for use in advance. Documentation of relevant properties shall be submitted and include:

— strength and strength development
— deformation characteristics
— thermal properties
— bond to concrete
— chemical compatibility with concrete
— stability/durability in future environment
— pot life.

7.12.1.3 Execution of repairs shall be performed by experienced personnel with documented capabilities. Prior to the actual execution procedure testing may be required to document:

— feasibility of repair
— in-place strength
— special requirements.

7.12.1.4 Execution and testing of repairs shall be surveyed and inspected for compliance with approved procedures.

7.13 Corrosion protection

7.13.1 General

7.13.1.1 Survey and inspection and execution of corrosion protection systems shall be in accordance with the requirements in Sec.5 and Sec.6 as relevant.

7.14 Site records and as-built documentation

7.14.1 General

7.14.1.1 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 shall include details of 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.

7.14.1.2 As a minimum the construction records shall contain:

— quality assurance/quality control manual
— relevant material certification and test reports
— summary testing reports 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 theoretical shape and out of alignment), placing of prestressing ducts and anchorages and location of embedments
— inspection summary reports
— as-built drawings
— information with regard to any non-conformances
— information with regard to any waivers or modifications from the specified requirements
— information with regard to storage, handling, installation, testing and operation of items shipped with the structure.

7.15 Precast concrete elements

7.15.1 General

7.15.1.1 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.

7.15.1.2 When precast elements are used in offshore concrete structures, their manufacture and design are covered by this standard. Therefore they shall meet all requirements to materials, strength and durability as if they were cast in-situ.

7.15.1.3 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 reinforcement)
— 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.

7.15.1.4 Precast elements shall be clearly marked and identified with their intended position, and in case of any ambiguity due to visual symmetry, also marked and identified with their lateral and vertical orientation in the final structure. As built information and records of conformity testing and control shall be available.

7.15.1.5 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.

7.15.2 Handling and storage

7.15.2.1 Instructions shall be available giving the procedures for the handling, storage and protection of the precast elements.

7.15.2.2 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.

7.15.2.3 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.

7.15.3 Placing and adjustment

7.15.3.1 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.

7.15.3.2 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.

7.15.3.3 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.

7.15.4 Jointing and completion works

7.15.4.1 The completion works are executed on the basis of the requirements given in the erection program and taking climatic conditions into account.

7.15.4.2 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.

7.15.4.3 Connectors of any type shall be undamaged, correctly placed and properly executed to ensure an effective structural behaviour.

7.15.4.4 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.

7.15.4.5 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.

7.16 Geometrical tolerances

7.16.1 General

7.16.1.1 Design tolerances are specified in [6.3.1]. The design assumption is based on an alternative approach, either:

- design and construct in accordance with the tolerances in [6.3.1] with high material factors, or
- design and construct 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.

7.16.1.2 This clause defines the types of geometrical deviations relevant to offshore structures. see [7.16.3], [7.16.4] and [7.16.5]. 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.

7.16.1.3 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.

7.16.1.4 All these factors shall be considered when tolerances are specified. Tolerances assumed in design (See [6.3.1]) may be greater than the tolerances actually found to be acceptable for other reasons.

7.16.1.5 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.

7.16.2 Reference system

7.16.2.1 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.

7.16.2.2 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.

7.16.3 Member tolerances (guidelines)

7.16.3.1 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 best fit inner radius from intended radius
— 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.

7.16.4 Cross-sectional tolerances (guidelines)

7.16.4.1 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 reinforcement bar layers same face
— tolerance on distance between rebar layers opposite faces
— tolerances on spacing of rebars in same layer
— tolerances on lap lengths.

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.

7.16.5 Embedments and penetrations (guidelines)

7.16.5.1 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
— blockouts deviations from intended position and dimensions.

7.17 Grouting operations

7.17.1 General

7.17.1.1 The grouting operation shall be conducted with strict adherence to the approved procedures, see [7.17.1.7]. The grout mix shall be prequalified to document compliance with this standard, relevant national regulations and project specific performance criteria through a laboratory test programme.

The grouting procedures and prequalified grout mix shall be documented to be adequate through full scale testing onshore prior to application in the structure. The testing shall consider the limitations imposed on the operations by the work site, structure layout and likely environmental conditions. Failure criteria, for instance for voids with respect to size and location in a post tensioning duct mock up, should be established in advance of testing.

7.17.1.2 Prior to start of operation it shall be ensured that the grouting system is operable, and that air and surplus grout may be evacuated from the volume at a rate exceeding the filling rate, Means shall be provided to observe the emergence of grout from the various emergence points.

7.17.1.3 Grouting shall only be conducted if the temperature in the structure immediately adjacent to the grouted part ranges from +5°C to +30°C, and will continue to be within this range for not less than 72 hours after grouting, unless a low or elevated temperature prequalification test programme has been completed.

Guidance note:

Note that additional precautions such as conditioning of materials and/or equipment may be required to ensure that the desired fresh properties are met when installing the material close to the extremes of the qualified temperature range.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

7.17.1.4 If a low or elevated temperature prequalification test program has been completed grouting outside the specified range may be permitted. Such testing shall as a minimum document the relevant fresh and hardened properties of the material at the intended application temperature. Where low temperature grouting is planned it shall be ensured that the surfaces of the compartment to be grouted are kept free from frost, taking effects from wind chill into account, prior to and during placement.

Guidance note:

Where grouting of post tensioning ducts with significant rise is planned the prequalification test programme should be designed to capture the effect of pressure at normal and low temperatures (if applicable). Note that low temperatures may delay initial set significantly thereby increasing the total bleed potential of the material.

The acceptance criteria for bleeding of fresh grout using the method defined in ASTM C1741 may be taken as zero bleed after 10 minutes subjected to a pressure corresponding to the hydrostatic pressure experienced by the grout at the bottom of the duct in question. It is recommended to repeat the test at two or more intervals up to the attainment of initial set.

Note that ASTM C1741 should not replace the standard tests such as ASTM C940, EN445 (*Inclined Tube Test or Wick Induced Test*) or similar in the prequalification and field test programmes. Rather it should be in addition to evaluate the effect of pressure specifically.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

7.17.1.5 The recorded grout temperature during production should not be less than + 10°C nor above +25°C during placement without due consideration of pumpability. Pumpability of the grout at elevated temperatures should be verified by means of full scale testing.

7.17.1.6 If the temperature in the structure is above +30°C grouting may be permitted provided special precautions, including an elevated temperature prequalification test programme documenting material properties as well as operational activities , ensure a successful grouting operation.

7.17.1.7 A grouting procedure shall be prepared and submitted for approval. The procedure shall, as a minimum, contain the following information:

— requirements to fresh grout properties; bleeding, fluidity, density, etc.
— requirements to hardened grout
— batching and mixing requirements
— means of transportation of fresh grout
— requirements to pumps and other equipment
— grouting pressure
— limitations on grouting speed
— 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.

7.17.1.8 Grouting shall be carried out at a continuous and steady rate from the lowest inlet until the emerging grout has the appropriate quality, not affected by evacuated water, or in the case of ducts, preservation oil.

7.17.1.9 Non-retarded grout and grout with an expanding admixture shall be placed into the structure within 30 min after mixing unless otherwise proven by testing.

7.17.1.10 Records shall be kept during the grouting operation. Each batch shall 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.

7.17.1.11 During the grouting operation, survey and inspection shall be performed to ensure compliance with the approved procedure.

7.17.1.12 For grouting of post tensioning ducts see [7.11.7].

SECTION 8 IN-SERVICE INSPECTION, MAINTENANCE AND CONDITIONAL MONITORING

8.1 General

8.1.1 Application

8.1.1.1 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 should be achieved. Alternative methods may also fulfil the intent of these provisions and may be applied provided they are demonstrated and documented to provide the same level of safety and confidence.

8.1.1.2 Requirements for in-service inspection, maintenance and condition monitoring for concrete offshore structures in general are given under this subsection.

8.1.2 Scope

8.1.2.1 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 shall 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.

8.1.3 Personnel qualifications

8.1.3.1 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.

8.1.3.2 Inspectors shall have adequate training appropriate for supervisors, divers, ROV-operators as specified in accordance with national requirements where applicable.

8.1.4 Planning

8.1.4.1 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.

8.1.4.2 The condition of the load-bearing 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.

8.1.4.3 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.

8.1.4.4 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 economic consequences. Forecasting the occurrence of these events is needed to allow sufficient lead-time for corrective action (e.g. to repair) or abandonment.

8.1.4.5 The structure should also be monitored to detect small damages and defects, which may develop to a critical situation. Particular emphasis should be placed on identifying the likelihood of small failures, which may 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.

8.1.5 Programme for inspection and condition monitoring

8.1.5.1 The first programme for inspection and condition monitoring should provide an initial assessment, as described in [8.1.6.2] 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.

8.1.6 Inspection and condition monitoring milestones and intervals

8.1.6.1 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.

8.1.6.2 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.

8.1.6.3 During in-service, more information will become available and the knowledge about the initial condition may be updated.

8.1.6.4 Inspection and condition monitoring of the structure shall be carried out regularly in accordance with the programme for inspection and condition monitoring established.

8.1.6.5 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 [8.1.7]. 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.

8.1.6.6 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.).

8.1.6.7 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.

8.1.6.8 Based on 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.

8.1.7 Documentation

8.1.7.1 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.

8.1.7.2 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).

8.1.7.3 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, video-recordings
— 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.

8.1.8 Important items related to inspection and condition monitoring

8.1.8.1 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.

8.1.8.2 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.

8.1.8.3 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, should 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 should 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

8.1.8.4 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 de-laminations
— local impact damage.

8.1.8.5 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.

8.1.8.6 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).

8.1.8.7 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 actions against possible harmful effect of bacterial activity shall be evaluated.

8.1.8.8 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.

8.1.8.9 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 should be used for estimating the time to initiation of reinforcement corrosion attack in the structure.

8.1.9 Corrosion protection

8.1.9.1 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.

8.1.9.2 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.

8.1.9.3 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.

8.1.9.4 Inspection of corrosion control based on use of corrosion resistant materials should 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 measurements of the electro potential of the reinforcement may 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

8.1.9.5 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 may 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 cathodic protected any corrosion should be prevented. In cases where cathodic protection of the reinforcement is limited, the absence of spalling and rust staining at cracks in the concrete cover should not be taken as evidence for no corrosion.

8.1.10 Inspection and condition monitoring types

8.1.10.1 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.

8.1.10.2 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.

8.1.10.3 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 may be performed at a distance, without direct access to the inspected areas, for instance by use of binoculars. 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 may be provided by use of instrumentation based condition monitoring. The instrumentation may 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.

8.1.10.4 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.

8.1.10.5 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.

8.1.11 Marking

8.1.11.1 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.

8.1.12 Guidance for inspection of special areas

8.1.12.1 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, de-laminations and surface corrosion staining.

8.1.12.2 The emphasis for the monitoring will be to detect and monitor damage caused by overstressing, abrasion, impact damage, and environmental exposure.

8.1.12.3 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 may 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.

8.1.12.4 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 crack widths 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 crack width. The investigation should identify the cause or causes, the changes with time, and the likely effect on the structure.

8.1.12.5 Concrete crushing, spalling and de-lamination also require careful determination of cause. Crushing is generally associated with either flexural overload, axial compression or impact. Delamination and spalling may 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.

8.1.12.6 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 may be used to detect ongoing corrosion that is not visible by visual inspection alone.

8.1.12.7 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 de-laminations
— joint cracking or separation.

8.1.12.8 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.

8.1.12.9 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.

8.1.12.10 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.

8.1.12.11 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 may take place. Continuous flow of saline and oxygenated water may 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.

8.1.12.12 The splash zone may experience damage from impact of supply vessels, etc. and may 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 may 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.

8.1.12.13 Debris. Drill cuttings may 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 may 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.

8.1.12.14 Scour is the loss of foundation supporting soil material and may be induced by current acceleration round the base of the structure or by pumping effects caused by wave induced dynamic rocking motion. It may lead to partial loss of base support and ensuring unfavourable redistribution of loads.

8.1.12.15 Differential hydrostatic pressure (drawdown). Structural damage, or equipment failure, may lead to ingress of water and affect the hydrostatic differential pressure (see [8.1.10.5]). This might call for special inspection before and during drawdown.

8.1.12.16 Temperature of oil sent 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.

8.1.12.17 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_2S (Hydrogen sulphide) which, in large enough amounts, will cause a lowering of pH value of the cement paste in the concrete. Favourable 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 may 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 may be obtained by adequate monitoring of SRB activity and pH levels.

8.1.12.18 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 may 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 may 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 de-laminations 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

APPENDIX A ENVIRONMENTAL LOADING

A.1 General

A.1.1 Environmental loads

A.1.1.1 Wind, wave, tide and current are important sources of environmental loads (E) on many structures located offshore. In addition, depending on location, earthquake or ice loads or both may be significant environmental loads.

A.1.1.2 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.

A.1.1.3 For fixed concrete structures, static analyses may 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 may 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.

A.1.1.4 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.

A.1.1.5 Environmental loads due to wind, wave and current relate particularly to the ultimate limit state requirements. In addition, these loads may 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.

A.1.1.6 The estimation of loads due to wind, wave and current requires an appropriate description of the physical environment in the form of sea state 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 DNVGL-OS-C101 Ch.2 Sec.2.

A.1.1.7 Procedures for the estimation of seismic loads are provided in DNVGL-OS-C101 Ch.2 Sec.2.

A.1.1.8 The computation of ice loads is highly specialized and location dependent and is not covered in detailed 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 DNVGL-OS-C101 Ch.2 Sec.2.

A.1.2 Wave loads

A.1.2.1 Wave loads 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.

A.1.2.2 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 are 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.

A.1.2.3 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.1.3 Diffraction analysis

A.1.3.1 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.

A.1.3.2 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 may be predicted based on linearized three-dimensional potential theory.

A.1.3.3 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.

A.1.3.4 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.

A.1.3.5 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.

A.1.3.6 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.

A.1.3.7 Structures with significantly varying cross-section near the waterline, within 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 may be significantly non-linear relative to the magnitude of the sea state. Linear diffraction theory assumes wall-sided geometry at the waterline.

A.1.3.8 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.

A.1.3.9 The hydrodynamic pressure at the seabed and within the seabed soil being different at either side of the GBS causes a net horizontal force onto the soil. This shall be accounted for in the foundation stability analyses, as outlined in DNVGL-RP-C212.

A.1.3.10 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.1.4 Additional requirements for dynamic analysis under wave load

A.1.4.1 In cases where the structure responds 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.

A.1.4.2 Ringing may control the 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.

A.1.4.3 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.1.5 Model testing

A.1.5.1 The necessity of model tests to determine 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 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.

A.1.5.2 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.

A.1.5.3 Where possible, model test loads shall be validated by comparison with analytical solutions or the results of prior appropriate test programs.

A.1.5.4 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.

A.1.5.5 Model test data shall be converted to full scale by appropriate factors consistent with the physical scaling procedures applied in the test program.

A.1.5.6 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 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.

A.1.5.7 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.1.6 Current load

A.1.6.1 Currents through the depth, including directionality, shall be combined with the design wave conditions. The Characteristic current load shall be determined in accordance with DNVGL-OS-C101 Ch.2 Sec.2.

A.1.6.2 The disturbance in the incident current field due to the presence of the fixed structure shall be accounted for.

A.1.6.3 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.

A.1.6.4 Numerical procedures based on computational fluid dynamics (CFD) may be considered in the evaluation of current loads or other effects associated with current. These 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 may provide a more economic and reliable procedure for predicting drag forces than physical modelling techniques.

A.1.6.5 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.

A.1.6.6 The presence of water motions in the vicinity of the base of a structure may 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.

A.1.6.7 If found necessary scour protection shall be provided around the base of the structure. See DNVGL-OS-C101 Ch.2 Sec.10.

A.1.7 Wind loads

A.1.7.1 Wind loads may be determined in accordance with DNVGL-OS-C101 Ch.2 Sec.2.

A.1.7.2 Wind forces on an offshore concrete structure will consist of two parts:

— wind forces on topside structure
— wind forces on concrete structure above sea level.

A.1.7.3 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.

A.1.7.4 Global mean wind loads on the exposed part of a 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.

A.1.7.5 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 may be susceptible to wind dynamics. An appropriate description of wind dynamics such as a wind spectrum shall be included in wind load estimation.

A.1.7.6 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

B.1 General

B.1.1 Physical representation

B.1.1.1 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.

B.1.1.2 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.

B.1.1.3 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 their magnitude (imperfection bending in walls, implosion of shafts, etc.).

B.1.1.4 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.

B.1.1.5 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.

B.1.1.6 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.

B.1.1.7 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.

B.1.1.8 It is normally acceptable to simulate the concrete by equivalent linear elastic properties in most limit states. Unless a different value is justified, the Young's modulus of plain concrete, taken as the secant value between σ_c = 0 and 0.4 f_{cck}, 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 Young's modulus should be considered in the analyses of the corresponding load effects.

B.1.1.9 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.

B.1.1.10 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.

B.1.1.11 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.

B.1.1.12 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.

B.1.1.13 Unless another value is shown to be more appropriate, a Poisson's ratio of $v = 0.2$ shall be assumed for un-cracked concrete. For cracked concrete, a value of $v = 0$, may be used. A coefficient of thermal expansion of 1.0×10^{-5} /°C 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.

B.1.1.14 The representation of soil-structure interaction for any type of analysis and load situation shall take into account the non-linearity of the soil, regardless whether the interaction is represented by distributed reactions or by distributed or lumped springs. For further discussions see DNVGL-RP-C212.

B.1.1.15 Reactions on the structure from its foundation/anchorage shall be based on general principles of soil mechanics in accordance with DNVGL-RP-C212. 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.

B.1.1.16 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. For more discussion see DNVGL-RP-C212.

B.1.1.17 Consideration shall also be given to the unevenness of the seabed, which may potentially cause high local reactions.

B.1.1.18 Upper limits of soil resistance should be considered during analysis of platform removal.

B.1.1.19 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 calculating subsequent foundation pressures.

B.1.1.20 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.

B.1.1.21 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.

B.1.1.22 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.

B.1.1.23 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.

B.1.1.24 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.

B.1.1.25 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.

B.1.1.26 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.

B.1.2 Loads

B.1.2.1 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.

B.1.2.2 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.

B.1.2.3 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.

B.1.2.4 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.

B.1.2.5 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.

B.1.2.6 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 load effects. Pseudo-static or dynamic analyses shall be used.

B.1.2.7 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.

B.1.2.8 Structures designed to contain cryogenic gas (LNG) shall additionally be designed in accordance with the provisions made in DNVGL-ST-C503.

B.1.3 Mass simulation

B.1.3.1 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.

B.1.3.2 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.

B.1.3.3 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 full mass 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.

---e-n-d---o-f---n-o-t-e---

B.1.4 Damping

B.1.4.1 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

C.1 General

C.1.1 Linear elastic static analysis

C.1.1.1 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 [C.1.2] to [C.1.4].

C.1.1.2 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 [C.1.2] and static analysis shall only be permitted when dynamic effects are small.

C.1.1.3 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 is normally required for accidental limit states, abnormal level earthquake and local analysis.

C.1.1.4 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 deformational 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.

C.1.1.5 Reduction of the stiffness of components should be considered if it is 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.

C.1.1.6 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 should 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 load cases that afterwards may be scaled in magnitude and added to represent complete load combinations i.e. loading scenarios is very effective.

---e-n-d---o-f---n-o-t-e---

C.1.2 Dynamic analysis

C.1.2.1 Fixed structures with natural periods of the global structure greater than 2.5s may be susceptible to dynamic response due to wave action during in-service conditions, at least for fatigue assessment. Structures in shallow water subject to large waves may exhibit significant dynamic response at lower periods due to the higher frequency content of shallow water or particularly steep waves.

C.1.2.2 Other load conditions to which the structure may be subjected, such as sea tow, wind turbulence, vibration, impact and explosion may 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.

C.1.2.3 Earthquakes are a particularly severe form of oscillatory loading that shall always require detailed dynamic analysis in moderate and high seismicity areas.

C.1.2.4 Where dynamic effects are significant, dynamic response may 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.

C.1.2.5 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.

C.1.2.6 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 [C.1.3].

C.1.2.7 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.

C.1.2.8 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 apply for seismic analysis. For more discussion see DNVGL-RP-C212.

C.1.2.9 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.

C.1.2.10 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.

C.1.2.11 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.

C.1.2.12 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 should be provided.

C.1.3 Pseudo-static analysis

C.1.3.1 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 essential similar response pattern within the structure but differ in magnitude.

C.1.3.2 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.

C.1.3.3 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.

C.1.3.4 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.

C.1.4 Non-linear analysis

C.1.4.1 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 abnormal level seismic events.
— For slender members in compression, where deflection effects are significant (imperfection, bending or buckling).

C.1.4.2 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 shall be applied in steps or increments, and at each loading step, iterations for equilibrium shall be carried out.

C.1.4.3 Non-linear solutions shall not be superimposed. This implies that a non-linear analysis shall be carried out for every load case or load combination, for which a solution is requested.

C.1.4.4 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.

C.1.4.5 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.

C.1.4.6 Only linear elastic stress-strain curves for FRP reinforcement shall be included in the analyses. This will limit redistribution of forces in the concrete structure.

APPENDIX D USE OF ALTERNATIVE DETAILED DESIGN STANDARD

D.1 General

D.1.1 Introduction

D.1.1.1 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 provisions in this appendix.

D.1.1.2 Other recognised codes or standards may be applied provided it is documented that they meet or exceed the level of safety of this DNV GL standard.

D.1.1.3 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 appendix 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 [D.1.2] 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

D.1.2 Conditions

D.1.2.1 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.

D.1.2.2 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.

D.1.2.3 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.

D.1.2.4 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 S-N curves, representing the 2.5% fractile, should be prepared for reinforcement bars, and in particular for items that have stress concentrations such as couplers, end anchors and T-heads.

D.1.2.5 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 [6.17.2]
— limitation of crack-widths under SLS conditions, see [6.15.3].

D.1.2.6 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 under-pressure 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.

D.1.2.7 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.

D.1.2.8 The reference standard shall give the design principles required for design of prestressed concrete, including principles for partial prestressing, when appropriate.

D.1.2.9 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.

D.1.2.10 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.

D.1.2.11 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 are dependent of how water pressure is applied in the initial calculation of action effects.

D.1.2.12 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.

D.1.2.13 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.

D.1.2.14 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.

D.1.2.15 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.

D.1.2.16 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 conventional linear elastic strength analyses, employing normal design and detailing rules for the reinforcement design.

D.1.2.17 In cases where the seismic action cause large amplitude cyclic deformations which may 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.

D.1.2.18 The material factors shall be such that a total safety level consistent with this standard is obtained. This shall be documented.

APPENDIX E CRACK WIDTH CALCULATION (INFORMATIVE)

E.1 Steel reinforced structures

E.1.1 Introduction

E.1.1.1 The general basis for calculation of crack width in an offshore structure is provided in [6.15.8].

E.1.1.2 This appendix provides recommendations for calculation of crack width for stabilized crack pattern. Stabilized crack pattern is defined as a crack pattern 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.

E.1.1.3 Normally, a stabilized crack pattern is used in evaluation of crack width as the provision of minimum reinforcement in the structure is intended to ensure a well-spaced developed crack pattern.

E.1.2 Stabilized crack pattern

E.1.2.1 Influence length, l_{sk}

For stabilized crack pattern, 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:

$$s_{rk} = 1.7 s_m = 1.7\left(s_{ro} + \frac{k_c A_{cef}}{\sum \dfrac{\pi \phi}{f_{tk} k_b / \tau_{bk}}} \right)$$

where the summation, Σ, covers tensile reinforcement within the concrete area influencing the transfer of tensile stresses between concrete and tensile reinforcement between cracks, A_{cef}.

E.1.2.2 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:

$$s_{rk} = 1.7 s_m = 1.7\left(s_{ro} + \frac{(f_{tk}/\tau_{bk}) k_b k_c h_{cef} s_b}{\pi \cdot n \cdot \phi} \right)$$

where:

s_{ro}	=	20 mm (a constant length with presumed loss of bond)
f_{tk}/τ_{bk}	=	the effective ration 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 · h_{cef}, 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	=	the width of the effective concrete section considered [mm]
h_{cef}	=	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} 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	=	a coefficient which accounts for the strain distribution within the cross-section $k_c = (1 + \varepsilon_{II}/\varepsilon_I)/2$ where $\varepsilon_{II}/\varepsilon_I$ 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 n + 0.85, a coefficient which accounts for reduced bond of bundled reinforcement
c	=	the concrete cover for the reinforcement under investigation
ϕ	=	the diameter of the reinforcement bar
s_b	=	the distance between reinforcement bars or bundles of bars, maximum value in the calculation 15φ (for bundles of reinforcement $15\phi\sqrt{n}$)
n	=	number of bars in a bundle.

E.1.2.3 Characteristic distance between cracks, s_{rk}, shall not be larger than 2.5 (h - x) and not less than 2.5 c, where c < (h-x).

E.1.2.4 Should the reinforcement be distributed unevenly between different parts of the cross-section, then the characteristic distance between the cracks, s_{rk}, shall be predicted individually for groups with similar intensity of reinforcement.

E.1.2.5 For reinforcement with perpendicular reinforcement bars spaced at a distance, s, then the characteristic distance between the cracks should be taken as n · s, where n is a whole number, and when the predicted distance between the cracks is greater than n · s and less than (n + 0.3) s.

E.1.3 Distance between cracks with deviations between the principle strain directions and the direction of the reinforcement

E.1.3.1 When the principal strain deviate from the direction of the reinforcement, then the distance between the crack width in the direction of the main reinforcement may be predicted from:

$$s_m = \frac{1}{\dfrac{\sin\nu}{s_{mx}} + \dfrac{\cos\nu}{s_{my}}}$$

where:

ν	=	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}	=	the predicted distance between the cracks in the x-direction
s_{my}	=	the predicted distance between the cracks in the y-direction.

E.1.4 General Method

E.1.4.1 The mean tensile strain, ε_{sm}, may be calculated using the principles outlined in [6.8]. The mean strain may be calculated based on the assumption that the concrete contribute between the cracks with an average tensile stress, β_s f_{tk}, and a corresponding strain, $\varepsilon_{cm} = \beta_s$ f_{tk} E_{cn},

where:

β_s is the ratio between the mean tensile stress and the tensile strength of the concrete in the influence area of the characteristic crack.

β_s = 0.6 for short duration one time loadings
= 0.4 for long duration or repeated loads at actual load level

For values of the Young's modulus E_{cn} reference is made to 4.3.3.

E.1.5 simplified approach

E.1.5.1 The crack width may be calculated by the following simplified equation:

$$w_k = s_{rk}\left(\left\{1-\beta_s\frac{\sigma_{sr2}}{\sigma_{s2}}\right\}\frac{\sigma_{s2}}{E_{sk}}-\varepsilon_{cs}\right)$$

where:

σ_{s2} = stress in the reinforcement in the crack for the actual cross-sectional forces.
σ_{sr2} = 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} = see simplified approach above.

σ_{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, σ_{s2}, and shall not be larger than σ_{s2}.

For structures exposed to water pressure, the reinforcement stress, σ_{s2}, shall include the effect of full water pressure, p_w, on the crack surface. Additional simplification may be made by presuming β_s = 0, thus neglecting the shrinkage strain.

E.2 FRP reinforced structures

These guidelines predict the crack width in structural elements which are reinforced by FRP surface reinforcement.
For structures reinforced by a mixture of steel reinforcement and FRP reinforcement, the provisions of [E.1]. applies.

For prestressed reinforcement, the prestressing force should be considered as an applied normal force and moment. If steel tendons are used, then the crack width criteria for sensitive reinforcement in [6.15.3.3] applies.

FRP reinforced concrete members only:

The characteristic crack width, for beams and slabs, is taken to be equal to:

$$w_k = 1.2\, w_m$$

and for pre-stress beam using FRP reinforcement, it is taken as:

$$w_k = 1.4\, w_m$$

where w_m denotes the mean crack width calculated for the mean elongation ε_{sm} which is produced along the average distance S_{rm} between cracks.

$$w_m = S_{rm} \,.\, \varepsilon_{sm}$$

If more accurate data are not available, the parameters S_{rm} and ε_{sm} of the previous equation should be assessed as follows, provided that the reinforcement is distributed in a sensibly uniform manner in the effective embedment section of the concrete:

a) after cracking has stabilized, the final average distance between cracks in the effective embedment section (see Figure E-1) is:

$$S_{rm} = 2(c + \frac{s}{10}) + \kappa_1 \kappa_2 \frac{\phi}{\rho_r}$$

where:

c = the concrete cover, for beam with side net of reinforcement and for deep beams the side's cover should be used

s = the spacing of the reinforcing bars, S ≤ 15 ϕ

ϕ = the bar diameter

k_1 = coefficient which characterizes the bond properties of the bars

k_1 = 0.4 for high bond bars

k_1 = 0.8 for plain bars

k_2 = coefficient representing the influence on the form of stress diagram

k_2 = 0.125 in bending

k_2 = 0.25 for pure tension

ρ_r = $A_s/A_{c,eff.}$

where:

A_s = the area of reinforcement contained in $A_{c,eff}$

$A_{c,eff}$ = the effective concrete area (effective embedment zone) where the reinforcing bars may effectively influence the crack widths:

$$A_{c,eff} = b \,.\, h_{c,eff}$$

where:

$h_{c,eff}$ = $\beta_{c,eff}$ (h-d)

$\beta_{c,eff}$ = the coefficient for effective height, for beams, it should be calculated using Figure E-2. For slabs (where t ≤ 0.3 m) $\beta_{c,eff}$ = 2.5.

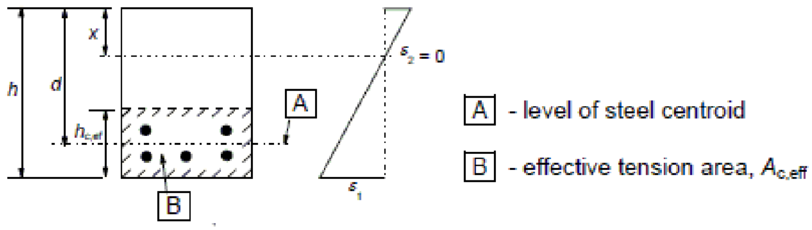


Figure E-1 Effective concrete area

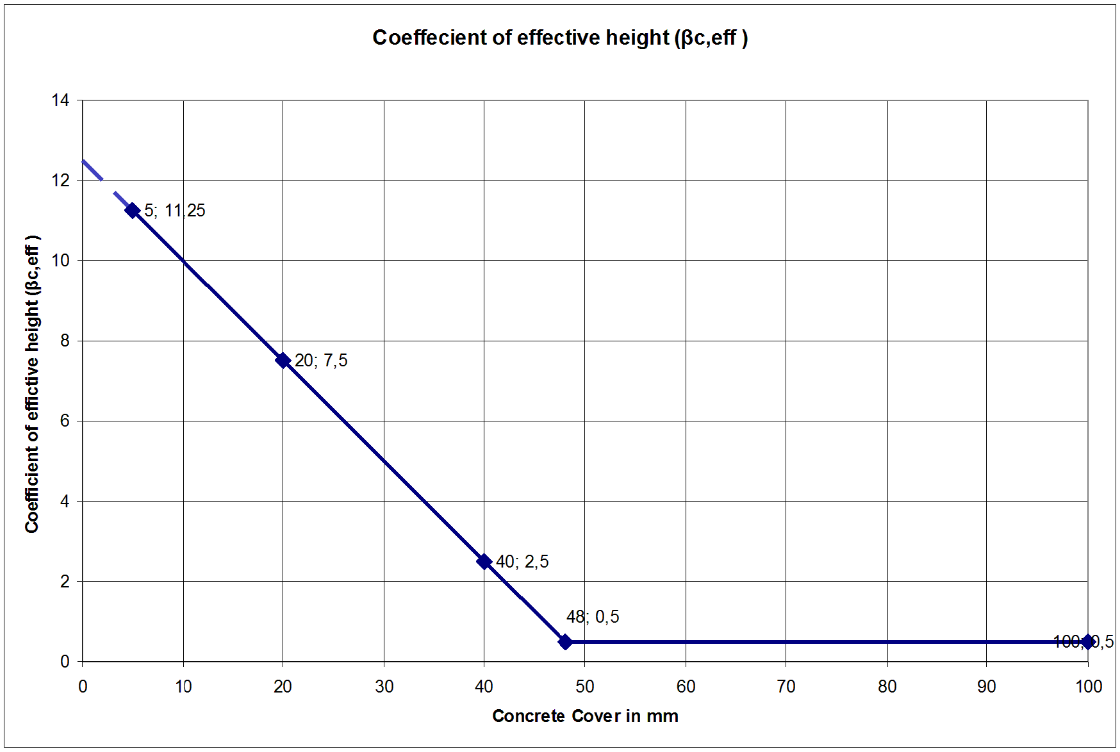


Figure E-2 Coefficient of effective height

b) The mean elongation of the reinforcement situated in the effective embedment section, taking account of the contribution of the concrete in tension should be taken as being equal to:

$$\varepsilon_{sm} = \frac{\sigma_s}{E_s}\left[1 - \beta_1\beta_2\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right] \geq 0{,}4\frac{\sigma_s}{E_s}$$

where:

σ_s = the stress in the reinforcement in the cracked section, under combination of actions under consideration

σ_{sr} = stress in the reinforcement calculated on assumption of a cracked section, where the maximum tensile stress in the concrete (un-cracked section) is taken equal to f_{tk}.

β_1 = coefficient which characterises the bond properties of the bars $\beta_1 = 1/(2,5\ k_1)$

where:

β_1 = 1.0 for high bond bars

β_1 = 0.5 for plain bars

and:

β_2 = coefficient representing the influence of the duration of application or repetition of loads

where:

β_2 = 1.0 at the first loading

β_2 = 0.5 for loads applied in a sustained manner or for a large number of load cycles.

APPENDIX F REQUIREMENTS TO CONTENT IN CERTIFICATES FOR FRP BARS

F.1 General

F.1.1 Minimum requirements

F.1.1.1 This standard opens for designing structural elements using FRP reinforcement bars of carbon, glass, aramid or basalt fibre reinforced composites.

F.1.1.2 In order to use this standard in evaluation of the structural capacity of structures using FRP reinforcement and in order to achieve comparative safety levels as required for steel reinforced concrete members, test results shall be included in a product or type approval certificate.

F.1.1.3 A product or type approval certificate shall be provided with each bar delivery. The certificate shall state, as a minimum, the information listed in Table F-1.

Table F-1 Information to be stated on FRP bar certificates

Reference to the relevant product specification	
Batch number and production dates	
Permissible temperature range	
$A_{F, BAR}$	Cross-sectional area of bar
ρ_F	Density of FRP bar [kg/m^3]
D_B	Nominal bar diameter
m_{tex}	Amount of fibres in the bar in tex []g/km] (alternative tow size in tex and number of tows)
m_f	Fibre fraction by weight
E_F	Characteristic value of the Young’s modulus of FRP reinforcement bar at qualified temperatures
$f_{F, bend}$	Characteristic strength of bent part of FRP bar
f_F	Characteristic value of short term tensile strength
$f_{F, TTR}$	Characteristic tensile strength (force per area) in FRP bar until failure in TTR tests at reference durations.
Characteristic strengths, $f_{F, bend}$, f_F, $f_{F, TTR}$, documented for elevated temperature testing.	
Characteristic strengths, f_F, $f_{F, TTR}$, documented for alkali degradation testing	
γ_F	Material factor to account for variation in strength, placement and manufacturing control, used to find γ_{FI}, γ_{FII}, γ_{FIII} for design, see [6.3.1.13].
$\gamma_{F,ssa}$	Material factor to be used for long term safe service life assessment.
γ_{FA}	Material factor to be used in accidental limit states.
γ_{FE}	Material factor on Young’s modulus of FRP bars accounting for long term creep effects in the bars.
α_F	Thermal expansion coefficient of FRP reinforcement

$\eta_{F, TTRI}$	Conversion factor for loads of duration 50 years corresponding to load combination type I.
$\eta_{F, TTRII}$	Conversion factor for loads of duration 1 year corresponding to load combination type II.
$\eta_{F, TTRIII}$	Conversion factor for loads of duration 1 week corresponding to load combination type III.
η_T	Temperature conversion factors for qualified temperatures, outside -20 to +20°C
η_b	Conversion factor for bends for the bend radiuses covered
C	Coefficient of characteristic safe service life formula (or parameters of other documented fatigue formulation)
k_1	Bond strength factor for FRP reinforcement relative to values in Table 6-6, [6.11.1.16].
k_{dB}	Coefficient in deflection calculation in [6.15.12.4].
Reference to test reports for pull-out bond strength testing at each qualified temperature	
Reference to fatigue testing test reports.at qualified application temperatures	
Reference to full scale elements test reports	
Reference to applicable standards	
Quality management system and manufacturing service arrangement (or similar) ref. nos.	
Reference to material and supplier quality control documents/certificates	

F.2 Testing of materials

F.2.1 Recommended testing

F.2.1.1 Laboratory testing of the FRP material and bar products shall be carried out as part of a complete qualification programme to document its properties for use in design.

F.2.1.2 Each of the parameters in Table F-1 shall be documented though a pre-qualified test programme. The testing required for each is given in Table F-2.

Table F-2 Critical parameters and corresponding recommended tests

E_F	E-modulus testing (instant elongation in response to tension) bar at qualified temperatures
$f_{F, bend}$	Embedded tensile strength of bent bars
f_F	Embedded static tensile strength testing
$f_{F, TTR}$	Embedded time to rupture tests at qualified temperatures.
$\gamma_{F,ssa}$	Embedded cyclic fatigue time to rupture tests at qualified temperatures
γ_{FA}	Embedded static tensile strength testing.
γ_{FE}	Creep testing (elongation due to sustained tension).
α_F	Thermal expansion testing (elongation of bars due to temperature)
η_T	Embedded static tensile strength testing, outside -20 to +20°C

η_b	Bend testing of bars embedded in concrete
C	Embedded cyclic fatigue time to rupture tests at qualified temperatures
$k1$	Pull-out bond strength.
k_{dB}	Full scale beam testing

F.2.1.3 Recommended tests for FRP bar products are tabulated in Table F-3.

Table F-3 Recommended tests methods – FRP bars

Parameter	*Test method*			*Comment*
	ISO 10406-1	*CSA S806*	*ACI 440.3R*	
Tensile strength in air	Sec.6	Ann.C	B2	Embedment conversion factor needed.
Embedded tensile strength	N.A.	N.A.	N.A.	No standard tests are available for bars embedded in concrete.
Pull-out bond strength	Sec.7	Ann.F Ann.D	B3	
Tensile strength of bent bars	N.A.	Ann.E	B5	
Alkali resistance	Sec.11	Ann.O	B6	Standard methods permit alkali exposure without loading. Effect of sustained and cyclic stress on alkali degradation needs to be documented in addition. Embedment conversion factor needed.
Cyclic fatigue in air	Sec.10	Ann.L	B7	Standards allow test frequencies of 1 – 10 Hz. The lower range is recommended. Anchor failures should not be counted as bar failure. Embedment conversion factor needed in addition for structural design.
Embedded cyclic fatigue time to rupture	N.A.	N.A.	N.A.	No standard tests are available for bars embedded in concrete.
Time to rupture in air	Sec.12	Ann.J	B8	Anchor failures should not be counted as bar failure. Embedment conversion factor needed in addition for structural design.
Embedded time to rupture	N.A.	N.A.	N.A.	No standard tests are available for bars embedded in concrete.
Long term relaxation in air	Sec.9		B9	
Long term creep		Ann.J		
Coefficient of thermal expansion		Ann.M		

F.2.2 Requirements of testing

F.2.2.1 Each bar dimension of each bar type and grade shall be characterised prior to use. The properties of each bar configuration and size shall be referred to the cross-section area for that bar size in the bar data sheet (product specification) provided by the manufacturer.

F.2.2.2 The testing described in simplified approach shall be carried out for each bar diameter to be used in design. After the first bar diameter has been successfully qualified subsequent bars diameters may require a less complete testing schedule for qualification, this may be decided upon review of the bar specific test data.

F.2.2.3 Testing is normally conducted at one particular reference temperature, typically room temperature (20 to 23°C). The properties of FRP bars may however be subject to change under different ambient operational temperatures. It is thought that the performance of the bars will not be detrimentally affected, to require additional testing, if operation is restricted to temperatures down to -20°C. However the performance of the bars at elevated temperatures (above +20°C), if required for likely application, shall be proven and documented by relevant testing.

F.2.2.4 Reinforcing FRP bars may be tested according to relevant international standards or guidelines such as ISO 10406-1, CSA 806-02, ACI 440.3R-04. However, additional characterisation shall be performed to characterise critical parameters not covered by those standards and guidelines. In particular, the performance of the FRP bars as embedded in concrete shall be documented by testing. Any effects of mechanical stress on alkali degradation shall also be documented by relevant tests.

F.2.2.5 Bar tensile strength shall be characterized in terms of the rupture strength due to tension that increases at a constant rate till rupture, hereafter denoted short term tensile strength for test durations of 2 to 5 minutes. If tests of bars in air are used to obtain the tensile strength of the bars (e.g. according to ISO 10406-1 Sec.6 or ACI 440.3R-04 Sec.B2) these tests shall be complemented with tests of the bars embedded in concrete to determine the conversion factor from strength in air to embedded strength.

F.2.2.6 Fatigue performance of the bars shall be documented by tests with cyclically varying tension loading where the number of cycles to failure is recorded. Tests shall be performed at mean stress levels and stress cycle magnitudes representative of the intended use of the bar. If tests of bars in air are used to obtain the fatigue performance of the bars (e.g. according to ISO 10406-1 Sec.6 or ACI 440.3R-04 Sec.B2) these tests shall be complemented with fatigue tests of the bars embedded in concrete to determine the conversion factor from fatigue performance in air to embedded fatigue performance.

F.2.2.7 Sustained load performance of the bars shall be documented by tests with constant sustained tension where the time to rupture (TTR) is recorded. If tests of bars in air are used to obtain the TTR of the bars (e.g. according to ISO 10406-1 Sec.6 or ACI 440.3R-04 Sec.B2) these tests shall be complemented with TTR tests of the bars embedded in concrete to determine the conversion factor from sustained load performance in air to sustained load performance as embedded in concrete.

F.2.2.8 The value of, $f_{F,\ TTR}$, the characteristic tensile strength (force per area) in the FRP bar until failure during TTR testing shall be documented for durations of loading ranging from 1 hour to 1 year.

F.2.2.9 The effect of exposure to the alkali environment within moist concrete on the static tensile strength, fatigue and sustained load performance shall be established by testing where the bars are exposed to a realistic environment. This should be done at least for the smallest bar dimension of each bar configuration.

F.2.2.10 Adequate bonding of the bars to the concrete shall be documented by relevant tests. The pull-out strength measured according to standardised tests (e.g. ISO 10406-1 Sec.7 or ACI 440.3R-04 Sec.B3) is well suited to compare bond strength of different bar configurations. For documenting the actual bonding performance of a specific bar in concrete, such pull-out tests shall be complemented with representative tests of structural elements showing adequate performance with regard to crack distribution and width, debonding failures, spalling, anchorage of the bars and overlap splicing of the bars.

F.2.2.11 The performance of the bars at bends, e.g. in stirrups, shall account for reduced tensile strength at the bend. The value of this reduction factor shall be documented by tests. As a minimum, the strength of bends should be determined experimentally for the largest cross-section and smallest bend radius of each bar configuration, in which case this bend strength may be applied to all bar dimension of that configuration. If the strength of bends is established for more than one bar dimension and bend radius, interpolation may be used to obtain strength values for intermediate cases. Extrapolation shall not be performed to more favourable strength values than documented by testing.

APPENDIX G QA/QC SYSTEM FOR MANUFACTURE OF FRP BARS

G.1 General

G.1.1 Minimum documentation

G.1.1.1 This appendix provides guidelines for QA/QC systems for manufacturing of FRP bars.

G.1.1.2 The method and documentation of verification of incoming raw materials by the bar manufacturer and the bar manufacturer’s own acceptance criteria shall be specified in the quality system. As a minimum a works certificate issued by the raw material suppliers shall be verified against the bar manufacturer’s acceptance criteria and filed. If type approved materials are specified for the production, this shall be verified. Testing carried out shall be described covering test equipment, test methods, test samples and reference to the test standards used.

G.1.1.3 The works certificate from the fibre supplier should state all information considered relevant by the bar manufacturer, not to be limited by the minimum information listed in Table G-1.

Table G-1 Information to be stated by fibre supplier in works certificate

Type designation, i.e. product name (grade) with list of tow weight (variants)
Name and address of the manufacturer
Batch number and production date(s)
Manufacturer's product specification/data sheet, including: — fibre type: designation, sizing (coating) and sizing content — fibre diameter with tolerances — chemical composition of the actual minerals with tolerances — type and application of coupling agents (if any) — powder or emulsion bounded — tow size (tex) with tolerances — moisture content — specified minimum fibre strength with reference to the test standard used — specified minimum fibre modulus with reference to the test standard used — specified maximum alkali degradation of bare fibre with reference to the test standard and conditions used (this serves as a means to control uniformity of material quality and is not used in design).
Fields of application and special limitations of the product. The suitability for service in the alkali environment as embedded in concrete should be addressed and whether this warrants any particular requirements for bar production.
Reference to specification of fabrication processes
Reference to specification of quality control arrangement
Quality system certification
Description of packing of the product
Information regarding marking of the product
Relevant service experience, if available
Type approvals of the product from relevant certifying agents

G.1.1.4 The works certificate from the resin supplier should state all relevant information, not to be limited by the minimum information listed in Table G-2.

Table G-2 Information to be stated by resin supplier in works certificate

Type designation, i.e. product name
Name and address of manufacturer
Product description (type of base resin, etc.)
Field of application and special limitations of the product (curing procedure, laminating procedure, shelf life, compatibility/non-compatibility with other materials, etc.) considering specifically the intended service in the alkali environment as embedded in concrete and measures needed to ensure bonding to concrete.
Reference to product specification, data sheet (mechanical properties, health data sheets, etc.), stating at least: Specified maximum alkali degradation of neat cured resin with reference the test standard used — Specified minimum elongation at break with reference the test standard used. — Temperature of deflection or glass transition temperature for the cure cycle specified for the bar manufacturing with reference the test standard used.
Test results with reference the test standard used.
Reference to specification of production processes
Reference to specification of quality control arrangement
Quality system certification
Information regarding marking of the product and packaging
Type approvals of the product from relevant certifying agents

G.1.1.5 Other incoming material shall have a marking that shall at least include the following information listed in Table G-3:

Table G-3 Required marking of incoming material

Manufacturer’s name
Production plant
Product name (grade)
Storage instruction (if applicable)
Production date

G.1.1.6 The conditions under which raw materials are stored shall be described. As a minimum the allowable range of temperature and relative humidity shall be specified as well as the method for controlling and logging these conditions. Cleanliness of the storage area shall be addressed as well as precautions if original packaging on stored material is broken. The control of shelf-life of products shall also be described.

Guidance note:

The storage area should be free from dust and other types of contamination that may have an adverse effect on the quality of the finished product.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

G.1.1.7 The FRP bar manufacturer shall completely describe each step in the production process of the bars from the production of each raw material input used to the delivery of the bar product. It shall also provide an overview of the production in general. For each step in the production process, aspects of particular importance shall be identified and how these aspects are taken care of by the techniques of manufacturing and quality control shall be described. The production parameters used for this control shall be identified and their target values and tolerances specified. The quality system including quality procedures and manufacturing instructions shall account for these aspects

G.1.1.8 A specification shall be made describing all relevant production parameters including details of how each shall be recorded and logged.

G.1.1.9 Special attention shall be given to the cleanliness of the fabrication area. The fabrication area shall be free from dust and other types of contamination that may have an adverse effect on the quality of the finished product.

G.1.1.10 The equipment used for curing and procedure for verification of the cure cycle shall be described.

G.1.1.11 The method and equipment used for cutting of the bars to length shall be described.

G.1.1.12 The extent of the manufacturer's quality control after production shall be documented.

G.1.1.13 During bar production, the characteristic values of strength and stiffness stated on the certificate or data sheet shall be confirmed. This shall be accomplished by means of tests of bars produced for delivery. The plan for the tests during production shall be specified by the bar manufacturer and included in the QA system in operation. The extent of testing shall be sufficient to confirm conformity of the as produced bars with the product data sheet.

The test plan shall be so designed as to provide data for the variability of bar strength from continuous production at the facility. It shall be verified that these estimates do not fall short of the characteristic values used in design.

A particular test plan for QC, in combination with the QA measures as implemented in the quality system, applies to one set of production parameters for one manufacturing machine at one site.

In case a nonconforming result is obtained from these tests, all bars produced since the previous conforming test result shall be treated as non-conforming.

G.1.1.14 Each FRP bar product shall be given a unique product name and a product specification uniquely identifying the bar product. Each bar product may be provided in a range of bar sizes. A cross-sectional area shall be specified for each bar size. A nominal area based on the specified cross-sectional fibre content (mass) of the bar is recommended. Alternatively, the area may be based on size measurements on produced bars. In that case, special care shall be taken to ensure that the cross-sectional areas used in processing of bar test results and in stress calculations are the same. The product specification for each bar product should include the information listed in Table G-4.

Table G-4 Basic information to identify a FRP bar

Designation of bar type (grade)
Constituent materials:
Fibre type, diameter and designation
Tow size
Resin type (e.g. epoxy, polyester)
Specific resin type (trade name, full designation)

Bar properties:
Cross-sectional area(s)
Net fibre area in a FRP reinforcement bar (A_f)
Fibre mass per unit length (tex)
Net fibre area of tow ($a_{f,tow}$)
Bar diameter(s)
Cross-sectional irregularities (e.g. waviness, ribs) with tolerances
Reference to technical datasheet with design data for mechanical properties
Process parameters:
Unique reference to processing specification for the specific bar type and grade
Processing temperature
Surface finish (e.g. sand cover)
Fibre volume fraction
Max content of voids, porosities and dry areas
Permissible environmental conditions for use of the bars:
Temperature range
Humidity conditions
Chemical environment (incl. pH)
For each parameter:
Measured values
Guaranteed minimum values
Estimated standard deviation, based on tests
Number of specimens tested
Other
Reference to applicable rules and standards the product complies with

G.1.1.15 The product or package shall be marked. The marking shall be carried out in such a way that it is visible, legible and indelible. The marking shall at least include the following information:

— manufacturer’s name
— production plant
— product name (type and grade)
— storage instruction (as applicable)
— production date
— batch number
— bar size (e.g. diameter).

G.1.1.16 Packaging, spooling and other handling shall be according to procedures specified by the manufacturer.

G.1.1.17 The procedure for handling and installation shall contain the necessary instructions and limitations set to protect the integrity of the bars during construction and in the installed condition. This should in particular consider required measures to prevent damage from exposure to UV radiation, solar heating, local bending, crushing and contamination of the bars that may compromise bonding to the concrete.

G.1.2 Physical properties of bars

G.1.2.1 Cross sectional properties should be defined as follows. The net fibre area in a FRP cross section is the sum of the cross section areas of all the fibres in the cross section. It should be computed from the specified tex mass value as follows: $\alpha_{f,tow} = \frac{m_{tex}}{\rho_f}$

where ρ_f is the density of the fibre.

In the above equation the following units shall be applied; $\alpha_{f,tow}$ in mm^2, m_{tex} in g/km; ρ_f in kg/m^3

The variability of this area is usually small. The volume fraction of fibres is obtained from the average mass fraction by:

$$v_f = \frac{\frac{m_f}{\rho_f}}{\frac{m_f}{\rho_f} + \frac{m_m}{\rho_m}}$$

where m_f is the average mass fraction of fibres from production records and m_m is the average mass fraction of matrix resin ($m_m = 1 - m_f$). The nominal bar cross sectional area is given by the volume fraction of fibres and the net fibre area:

$$A_B = \frac{A_f}{v_f}$$

where the fibre area $A_f = N \cdot a_{f,tow}$ and N is the number of tows in the bar. All bar stresses are defined in terms of the nominal bar section area:

$$f_B = \frac{F_B}{A_B}$$

Although the cross section may be intentionally irregular, one may for convenience define the nominal bar diameter assuming a circular cross section:

$$D_B = 2\sqrt{\frac{A_B}{\pi}}$$

This nominal diameter can be used to calculate the bar's surface area for design calculations.

APPENDIX H REQUIREMENTS TO CONTENT IN CERTIFICATE FOR STRUCTURAL GROUT

H.1 General

H.1.1 Minimum requirements

H.1.1.1 This standard opens for designing structural details using grout, or grout material reinforced by fibre reinforcements. The fibre may be made from either steel of FRP.

H.1.1.2 Pre-blended grout materials or systems shall be delivered to site ready for application; only mixing activities in accordance to the manufacturer's approved procedures shall be undertaken.

H.1.1.3 In order to use this standard in evaluation of the structural capacity of the grout and in order to achieve comparative safety levels as required for reinforced concrete members, test results shall be included in a certificate.

H.1.1.4 The certificate shall contain documentation specific to the type and means of application of the grout material, see [H.3].

H.1.1.5 For structural grout a type approval certificate (TAC) shall as a minimum contain the parameters and information outlined in Table H-1.

Table H-1 Minimum contents of TAC for structural grout

Content	*To be included on certificate*
Details of producer/owner of certificate	X
Maximum aggregate size	(as applicable)
Weight of dry grout (per packaged quantity)	(as applicable)
Weight of fresh water (per packaged quantity of grout)	(as applicable)
W/C ratio	(as applicable)
Range of qualified application temperatures	X
Workability over an applicable duration – efflux time or flow test	X
Density – fresh and hardened	X
Target densities during mixing	(as applicable)
Air content – fresh grout	X
Stability (separation and bleeding)	X
Setting time (initial and final)	X
Mean compressive strength (150 mm × 300 mm cylinders) at 3 days at $t_{test,\ min}$ and 20°C	X
Mean compressive strength (150 mm × 300 mm cylinders) at 7 days at $t_{test,\ min}$ and 20°C	X
Mean compressive strength (150 mm × 300 mm cylinders) at 28 days $t_{test,\ min}$ and 20°C	X
Characteristic compression strength (150 mm × 300 mm cylinders) at 28 days $t_{test,\ min}$ and 20°C	X

Content	*To be included on certificate*
Mean compressive strength (150 mm × 300 mm cylinders) at 90 days $t_{test,\ min}$ and 20°C	X
Mean compressive strength (75 mm cubes) at 28 days 20°C	X
Characteristic compression strength (75 mm cubes) at 28 days 20°C	X
Ratio between standard cylinder strength and control specimens to be used at site	X
Mean flexural strength (40 mm × 40 mm × 160 mm prisms) at 28 days at $t_{test,\ min}$ and 20°C	X
Characteristic flexural strength (40 mm × 40 mm × 160 mm prisms) at 28 days at $t_{test,\ min}$ and 20°C	X
Creep properties	X
Autogenous shrinkage/total shrinkage/expansion properties	(as applicable)
Young's modulus at 28 days	X
Poisson's ratio at 28 days	X
Fatigue parameter – C_5	X
Pumpability (with reference to approved mock-up test and test temperature)	X
Maximum hose length [m] with minimum nominal bore	X
Grout free fall distance in water	X
Compression strength development at elevated temperature	(as applicable)
Reference of approved grouting procedures	X
Reference to approved production facilities	X
Note: where a parameter is only relevant to certain applications or materials it has been marked as applicable:	

H.1.1.6 For fibre reinforced structural grout, the product certificate shall as a minimum contain the following parameters and information:

Table H-2 Minimum contents of product certificate for fibre reinforced structural grout

Content	*To be included on certificate*
Details of producer/owner of certificate	X
Maximum aggregate size	(as applicable)
Weight of dry grout (per packaged quantity)	(as applicable)
Weight of fresh water (per packaged quantity of grout)	(as applicable)
W/C ratio	(as applicable)
Works certificate for fibre and resin raw materials	X
Volumetric content of fibres	X
Fibre type	X
Fibre length	X

Content	*To be included on certificate*
Volumetric content of fibres	X
Weight of fibres/m3 grout	X
Range of qualified application temperatures	X
Workability over an applicable duration – flow test	X
Density – fresh and hardened	X
Target densities during mixing	(as applicable)
Air content – fresh grout	X
Stability (separation and bleeding)	X
Setting time (initial and final)	X
Mean compressive strength (150 mm × 300 mm cylinders) at 3 days at $t_{test, min}$ and 20°C	X
Mean compressive strength (150 mm × 300 mm cylinders) at 7 days at $t_{test, min}$ and 20°C	X
Mean compressive strength (150 mm × 300 mm cylinders) at 28 days $t_{test, min}$ and 20°C	X
Characteristic compression strength (150 mm × 300 mm cylinders) at 28 days $t_{test, min}$ and 20°C	X
Mean compressive strength (150 mm × 300 mm cylinders) at 90 days $t_{test, min}$ and 20°C	X
Mean compressive strength (75 mm cubes) at 28 days 20°C	X
Characteristic compression strength (75 mm cubes) at 28 days 20°C	X
Ratio between standard cylinder strength and control specimens to be used at site	X
Mean flexural strength (40 mm × 40 mm × 160 mm prisms) at 28 days at $t_{test, min}$ and 20°C	X
Characteristic flexural strength (40 mm × 40 mm × 160 mm prisms) at 28 days at $t_{test, min}$ and 20°C	X
Long term load effects relating to sustained load fracture in FRP fibre reinforced material	X
Creep properties	X
Autogenous shrinkage/total shrinkage/expansion properties	(as applicable)
Young’s modulus at 28 days	X
Poisson’s ratio at 28 days	X
Fatigue parameter – C_5	X
Pumpability (with reference to approved mock-up test and test temperature)	X
Maximum hose length [m] with minimum nominal bore	X
Grout free fall distance in water	X
Compression strength development at elevated temperature	(as applicable)
Reference of approved procedures	X
Reference to approved production facilities	X
Note: where a parameter is only relevant to certain applications or materials it has been marked as applicable:	

H.2 Testing of materials

H.2.1 Recommended testing

H.2.1.1 Laboratory testing of the fresh and hardened grout material shall be carried out to document its properties for use in design.

H.2.1.2 The testing, specified in this subsection should be carried out by an independent laboratory holding relevant ISO 17025, or similar accreditation, as well as ISO 9001 certification.

H.2.1.3 Recommended test methods for high performance blended grout to document the fresh grout parameters are defined in Table H-3.

Table H-3 Recommended test methods - fresh grout

Test ID	*Type of test*	*Test method*	*Testing time (age)*	*Suggested no. of tests*	*Specified testing/ curing temp.*	
					$t_{test,\ min}$	*20°C*
FG1	Flow test	ASTM C1437 (C230)1)	As soon as practicable after mixing and then at 30, 60, 90 and 120 minutes2)	1 no. test specimen from each batch at each specified testing temperature	X	X
FG2	Density	EN 12350-6 or EN 1015-6	As soon as practicable after mixing	1 no. test specimen from each batch at each specified testing temperature	X	X
FG3	Bleeding/ segregation	ASTM C940	As soon as practicable after mixing and periodically thereafter	1 no. test specimen from each batch at each specified testing temperature	X	X
FG4	Air content	EN 12350-7 or EN 1015-7	As soon as practicable after mixing	1 no. test specimen from each batch at each specified testing temperature	X	X
FG5	Setting time (initial andd final)	ASTM C 953 (C191)3),4) or EN 196-33),4)	At regular time intervals after mixing until final set has been observed to produce a satisfactory penetration curve	1 no. test specimen from each batch at each specified testing temperature	X	X

1) No shock or agitation shall be applied to the flow table.
2) The material shall not be vibrated or excessively agitated between mixing and the test age.
3) The total mass of the moving parts, including the stopping device, shall be (1000 ± 2) g rather than the (300 ± 0.5) g as specified in ASTM C191 and EN 196-3.
4) Manual Vicat apparatus shall be used for determination of the setting time unless it has been documented that the automatic Vicat apparatus provides similar results at relevant testing temperatures.

H.2.1.4 The following test methods are recommended to document the hardened grout material parameters of high performance blended grout.

Table H-4 Recommended test methods - hardened grout

Test ID	*Type of test*	*Test method*	*Testing time (age)*	*Suggested no. of tests*	*Specified testing/ curing temp.*	
					$t_{test, min}$	*20°C*
HG1	Density	EN 12390-7	28 days	3 no. specimens from each batch	X	X
HG2	Compressive strength - 150 × 300 mm Cylinders	EN 12390-3	3, 7, 28, 90 days	4 no. cylinders from each batch at 3, 7 and 90 days at each specified testing temperature. 4 no. cylinders from each batch at 28 days at $t_{test, min}$. Sufficient no. of cylinders to compute characteristic strength value at 28 days at 20°C.	X	X
HG3	Compressive strength 75 mm cubes	EN 12390-3	28 days	Sufficient no. of cube specimens to compute characteristic strength value.		X
HG4	Flexural strength	ASTM C348 or EN 196-1	28 days	4 no. prisms from each batch at each specified testing temperature.	X	X
HG5	Creep	ASTM C512	28 days[1)]	2[2)] no. specimens from each batch		X
HG6	Shrinkage/ expansion[3)]	ASTM C157[4)] (ASTM C490)	24 hours, 28 days 8, 16, 32 and 64 weeks	2 no. specimens from each batch		X
	Autogenous shrinkage[3)]	ASTM C1698[5)]	As per ASTM C1698, Sec. 7.7 and regularly thereafter until 56 days	2 no. specimens from each batch		X
	Restrained shrinkage	ASTM C1581	As per ASTM C1581	2 no. specimens from each batch		X
HG7	Static Young's Modulus and Poisson's ratio – 150 × 300 mm cylinders	ASTM C469 or EN 12390-13	28 days	3 no. cylinders from each batch		X

Test ID	Type of test	Test method	Testing time (age)	Suggested no. of tests	Specified testing/ curing temp.	
					$t_{test, min}$	20°C

1) Curing period of 28 days prior to initial loading, followed by regular monitoring of relative deformation according to ASTM C512. ASTM C512 specifies to take strain readings immediately before and after initial loading, 2 to 6 h later, then daily for 1 week, weekly until the end of 1 month, and monthly until the end of 1 year. Since in practice the material is sometimes loaded before 28 days (i.e. pre-stressing of bolts at a specified minimum compressive strength) the test method may, after due consideration, be adjusted to capture this by loading the material before the specified 28 days curing age. For such cases the compressive strength shall be determined for the curing age under consideration. This should be clearly stated in the test report.
2) Two specimens from each batch represents the minimum number of specimen for the long term loading. Additional specimens shall be cast to determine the compressive strength and deformations due to causes other than load, as per ASTM C512. Test specimens for monitoring of deformations shall be sealed to prevent loss of moisture throughout the period of storage and testing.
3) Depending on the likely application of the material the most applicable test in this category may be chosen.
4) Storage method of specimens between comparator readings shall reflect the likely in-service environmental conditions.
5) A record of the specimen weight shall be made at regular intervals during the test, no fewer than an average of 3 per week.

H.2.2 Requirements of testing

H.2.2.1 To document the material properties of the grout a minimum of three production batches shall be represented in the samples for each of the tests specified in Table H-3 and Table H-4 to capture variability in the raw materials and the production process.

Guidance note:

For the purpose of documenting the characteristic compressive and flexural strength of the material it is recommended that approximately twenty test specimens, taken from as many distinct production batches as practical, are included in the sample. It is recommended that the statistical evaluation of the characteristic strength is undertaken in accordance with Annex D of EN 1990.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

H.2.2.2 All testing described in Table H-3 and Table H-4 shall be carried out at a reference room temperature of 20°C. For grout intended for low temperature service (<5°C) testing shall also be conducted at the minimum test temperature, $t_{test,\,min}$, see H.2.2.4.

H.2.2.3 Testing in accordance Table H-3 and Table H-4 carried out at 20°C documents the properties of the grout for normal application between a minimum application temperature, $t_{app,\,min}$ of 5°C and upper limit $t_{app,\,max}$ of 30°C.

H.2.2.4 For grout material intended for application below 5°C additional testing is required at the minimum test temperature, $t_{test,\,min}$. This is derived from the minimum application temperature, $t_{app,\,min}$, minus a constant, η_{temp}, to account for variability in the conditioning, testing and curing temperatures during the testing programme. $t_{test,\,min}$ is therefore defined as:

$$t_{test,\,\min} = t_{app,\,\min} - \eta_{temp}$$

where:

η_{temp} = 1°C, for normal control conditions.

H.2.2.5 For grout intended for use in regions or environments where the curing or application temperature, $t_{app,\ max}$, is expected to be greater than 30°C, additional testing shall be conducted similar in scope to that required for the minimum test temperature. Additionally, an elevated temperature mock-up test, including pump test, shall be conducted.

H.2.2.6 Constituent materials, mixing and testing equipment as well as the testing environment shall be pre-conditioned at the testing temperature for at least 24 hours prior to mixing. This is highly important for testing the grout at cold and/or elevated temperatures. Metallic testing equipment and moulds dissipate the heat out of the grout material when testing is conducted at low temperature, $t_{test,\ min}$.

H.2.2.7 .

The temperature of the low temperature curing environment shall be recorded and logged while testing is ongoing.

H.2.2.8 Curing of specimens shall be conducted in accordance with EN 12390-2. For testing at extremely low temperatures ($t_{test,\ min} \leq 0°C$) alternative methods of curing may be considered. This may include the use of salt in the curing water or similar.

H.2.2.9 Suitable calibrated moulds in accordance with EN 12390-1 shall be used for casting of test specimens. Note that dimensions of the cast specimens shall be recorded prior to testing

Guidance note:

In the case that 75 mm cubical moulds are manufactured, the fabrication tolerances in EN 12390-1, Section 5.2.4 should be governing.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

H.2.2.10 Test cubes and prisms for testing hardened grout should, in the absence of specific requirements in the applicable referenced standards, be initially cured in moulds covered with non-absorptive and non-reactive plates or sheets of tough, durable impervious plastic at the specified test temperature. The initial curing temperature shall be recorded.

H.2.2.11 The time elapsed between grout mixing and the commencement of grout testing shall be recorded. The tests shall commence at a specified grout age. The age shall be recorded within the following time accuracy:

— Specified grout age within 24 hours after mixing: ± 15 min.
— Specified grout age within 48 hours after mixing: ± 30 min.
— Specified grout age within 72 hours after mixing: ± 45 min.
— Specified grout age within 7 days after mixing: ± 2 hrs.
— Specified grout age within 28 days after mixing: ± 8 hrs.
— Specified grout age within 90 days after mixing: ± 1 day.

H.2.2.12 Temperature logging of the materials, testing equipment and environment during low/elevated temperature material testing shall be carried out throughout the conditioning, testing and curing of the specimens (where relevant). A representative sample of the grout should also be monitored for temperature development after mixing. Time that specimens spend outside the temperature controlled environment during strength testing shall be limited to maximum 30 minutes.

H.2.2.13 Fatigue testing has not been included in the above specified testing although it is strongly recommended that these tests are carried out. Fatigue testing is required to determine C_5, the fatigue strength factor, see [6.13.2]. However, provision is made in [6.13.2.1] for the use of C_5 = 0.8 in the absence of witnessed testing. This figure is thought to be conservative.

Guidance note:

If the material is likely to be exposed to ponding water or if it shall be applied subsea then the conditioning of the specimens during fatigue testing should reflect the realistic environmental conditions. The test frequencies should reflect those expected during normal operation of the structure which the material will likely be applied in.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

H.2.2.14 It is recommended to perform additional testing to determine tensile splitting strength and direct tensile strength for comparison towards the flexural strength specified as test id. HG4 in Table H-4. Tensile splitting strength should be determined according to EN 12390-6 while the test method for direct tensile strength should be agreed in advance.

H.2.2.15 If the grout material shall considered to be frost (freeze thaw) resistant the requirements of a suitable testing norm shall be satisfied. Testing may be conducted in accordance with EN 13687-1 which tests adhesion after cyclical freeze/thaw exposure or the Borås method which assesses salt scaling of the material. Additional microscopic analysis of the hardened material in accordance with ASTM C457 should be used to verify the pore distribution.

H.2.2.16 If early age compressive strength development data, i.e. less than three days, is required additional compressive tests of cylinders shall be carried out.

H.2.2.17 If it is required to document the complete stress strain curve of the material including the descending portion, for instance when non-linear material behaviour is required for analysis, a testing machine capable of operating under displacement control should be used.

Guidance note:

The test conducted using displacement control, should continue until a strain of 6‰ is recorded. Strains may be measured using optical, mechanical or electrical extensometers or stereo-photo equipment.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

H.3 Supporting documentation

H.3.1 Minimum requirements for inclusion of certificate

H.3.1.1 Test results and supporting documentation shall be summarised and evaluated in a consolidated test report. The fulfilment of the requirements specified in Simplified Approach shall be documented for the test programme undertaken.

H.3.1.2 Material and supplier quality certificates for aggregates, cement, mineral and chemical admixtures shall be provided in accordance with Sec.4.

H.3.1.3 Details of a valid manufacturing plant quality management system in accordance with ISO 9001 and preferably ISO 9004 shall be included.

H.3.1.4 Details of a valid manufacturing plant survey scheme shall be included.

H.3.1.5 The production, method of application, as well as the quality control of the mixing, curing and placement process offshore has a significant impact on the final as-built performance of the material. The following documentation shall therefore be approved and referenced in the certificate:

— Grouting procedures for standardised grouting operations offshore for each of the applications to be qualified. These shall include contingency procedures.
— Procedure for large scale mock-up test. The mock-up test shall directly correspond to a grouting procedure for a specific application. The test-setup shall reflect the actual conditions and equipment to be used at the site including the grout mixer and pump, pumping height and hose with a representative

nominal bore diameter and length to assess pumpability of the material. The mock-up test shall demonstrate that the material maintains pumpability over the likely duration of the operation including possible pauses due to blockages or equipment failures. The most challenging placement configuration expected offshore shall be reflected in the test plan including contingency procedures. Appropriate material testing shall be conducted during the test and complete filling of the intended volume shall be demonstrated after hardening.

The precise requirements with regard to the mock-up test depends on the grouting operation (and procedure) under consideration. See App.J for detailed requirements.

— Procedures for all QC testing during offshore operations. Hardened grout sampling as well as details of all tests to be carried out on constituent materials, water and fresh grout shall be documented with regard to suitable standards.
— Procedures for casting, curing, transport of the offshore QC specimens. The curing conditions should be maintained during transport to as great a degree as is practical. Transport between controlled curing environments (i.e. from curing tank on board the installation/vessel to the curing tank in the testing facility) should be limited to a maximum of 72 hours.
— Details of the qualification program used to appoint third party grouting contractors (if applicable).

APPENDIX I QA/QC SYSTEM FOR MANUFACTURE OF STRUCTURAL GROUT

I.1 General

I.1.1 Minimum requirements

I.1.1.1 This appendix provides guidelines for QA/QC systems for manufacturing and batching structural grout products.

I.1.1.2 Documentation of the verification of the incoming raw materials' properties by the grout manufacturer and the manufacturer's own acceptance criteria shall be specified in the quality system. As a minimum test reports or works certificates, where applicable, issued by the raw material suppliers shall be verified against the grout manufacturer's acceptance criteria and filed. Testing carried out shall be described covering test equipment, test methods, test samples and reference to the test standards used.

I.1.1.3 The conditions under which raw materials are stored shall be described. As a minimum the allowable range of temperature and relative humidity shall be specified as well as the method for controlling and logging these conditions. Cleanliness of the storage area shall be addressed as well as precautions if original packaging on stored material is broken. The control of shelf-life of products shall also be described.

Guidance note:

The storage area should be free from contamination that may have an adverse effect on the quality of the finished product.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

I.1.1.4 The grout manufacturer shall completely describe each step in the production process of the grout from the sourcing of each raw materials used to the delivery of the final product. It shall also provide an overview of the production in general. For each step in the production process, aspects of particular importance shall be identified and how these aspects are taken care of by the techniques of manufacturing quality control shall be described. The production parameters used for this control shall be identified and their target values and tolerances specified. The quality system including quality procedures and manufacturing instructions shall account for these aspects.

I.1.1.5 A specification shall be made describing all relevant production parameters including details of how each shall be recorded and logged.

I.1.1.6 The method and equipment used for proportioning and batching the raw materials shall be described, including weighing tolerances.

I.1.1.7 During continuous production, the values for fresh and hardened grout properties stated on the type approval certificate shall be confirmed. This shall be accomplished by means of testing of material produced for delivery. The plan for the tests during production shall be specified by the grout manufacturer and included in the QA system in operation. The extent of testing shall be sufficient to confirm conformity.

I.1.1.8 The test plan shall be designed to capture sufficient data, including the variability of material quality, from continuous production at the facility. It shall be continuously verified that the test results do not fall short of the characteristic values used in design.

I.1.1.9 The basis of the produced material conformity criteria shall be documented by the manufacturer and presented in the QA plan. The following properties should be considered as a minimum:

— flow spread
— fresh density
— air content

— setting time
— stability
— mean and characteristic compressive strength (correlated to values used in design)
— Flexural strength (40x40x160 mm prisms at 28 days).

Guidance note:

The conformity criteria in EN 206 for compressive strength may be used to monitor conformity during continuous production to the values stated in the type approval certificate.

The relationship between compressive strength gained from production control specimens and 75 mm cubes should be established. In this way production control testing may be correlated to the strength of the material established during certification testing. Periodic testing of 75mm cubes may be incorporated into the QC Plan confirm the continued validity of the assumed relationship towards the production testing specimens.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

I.1.1.10 A particular test plan for QC, in combination with the QA measures as implemented in the quality system, applies to one set of production parameters for one proportioning and batching line at one site.

I.1.1.11 The QA system shall specify how to handle non-conformities.

I.1.1.12 Each delivered package shall be marked. The marking shall be carried out in such a way that it is visible, legible and indelible. The marking shall at least include the following information:

— manufacturer's name
— production plant
— product name (type and grade)
— storage instruction (as applicable)
— production date
— batch number
— expiry date.

I.1.1.13 The procedure for transport, handling, storage and installation shall contain the necessary instructions and limitations set to protect the integrity of the grout material prior to and during construction. It shall be according to procedures specified by the manufacturer.

I.1.1.14 At the point of placement both pre-packed and bulk stored blended grouts shall be traceable back to an individual production day and an associated set of QC test results from the production facility.

APPENDIX J MOCK-UP TEST REQUIREMENTS

J.1 General

J.1.1 Introduction

J.1.1.1 This appendix aims to define the minimum requirements for mock-up testing carried out during certification of structural grouts. Note that it is not the aim of this appendix to address mock up testing of grout for injection into post tensioning ducts.

J.1.1.2 The requirements provided within this appendix are given for annulus grouting of typical cylindrical/conical grouted connections, i.e. grouting operations displacing water in open top connections. The contents of these requirements are not exhaustive and there may be applications from a material, operational or structural perspective which are not covered.

Guidance note:

The requirements provided within this appendix may be taken as valid to a large extent of mock-up tests relating to annulus grouting of cylindrical/conical connections commonly found on jacket-, monopile-, jetty- and tri-pod foundations, piles for offshore loading systems, etc. In case of mock-up tests for grouted clamps and repair work, under flange grouting, etc. the basic principles may be adopted however the detailed requirements have to be assessed on a case by case basis.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.1.3 The mock-up test should be sufficient to prove the applicability of the grout material, offshore grouting procedure and equipment to be used for the particular application in question.

J.1.1.4 The mock-up test should aim to replicate the most critical configuration from either normal or contingency grouting operation offshore as specified by the grouting (sub)contractor.

J.1.2 Mock-Up test procedure

J.1.2.1 A mock-up test procedure should be prepared prior to execution of the test. The mock-up test procedure should address the following items:

— Description of the form including annotated drawings (Dimensions of wall sections, annulus thickness, location of inlet(s), surface preparation of forms).
— Acceptance criteria for the mock-up test corresponding to specified QC testing and visual inspection (both quantitative and qualitative).
— Preparation work (pre-calibration check of water meter, flooding of form, etc.).
— Procedure for feeding/filling mixers/silos.
— Starting procedure (lubrication of hoses, flushing of form, mixing and pumping of lubricant mix and initial pumping of grout).
— Mixing procedure (automated, manual, charging sequence, mixing time, water demand, etc.).
— Pumping procedures including pump rates, pump pressure and duration of pumping.
— Stopping procedures (means of verifying full filling of theoretical volume and grout quality).
— Identification of QC tests to be carried out including frequencies, number of tests and test methods.
— Method of assessing and verifying pumpability over the duration of the test.
— Mixing and pumping equipment to be used including technical data sheets.
— Lengths of hoses to be used including nominal bore diameters.
— Vertical head of grout to be simulated by means of lifting the grout hose..
— Redundancy/contingency procedure to be simulated
— Brief plan for inspection of hardened grout after initial curing and removal of formwork.

Guidance note:

For safety reasons the steel panel formwork should be provided by a recognised formwork supplier. Due attention should be paid to the stability and stiffness of the form during the test and to the safety of any personnel required to work on or access cantilevered gangways.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.3 Equipment

J.1.3.1 Equipment to be used offshore should be utilised in the test including mixer(s), pump(s), hoses (of same nominal bore diameter to be used during operations) as well as any pressure gauges and flow meters which are planned to be used.

J.1.3.2 For the mock-up test the dry powder shall be stored and transported following the same principles as are planned during offshore operations i.e. big bags or bulk storage/silo.

J.1.4 Personnel

J.1.4.1 The mock-up test shall be carried out with equivalent number of personnel to that specified during normal operation offshore. The roles and responsibilities shall be clearly defined in the mock-up test procedure.

J.1.5 Requirements and recommendations for test setup

J.1.5.1 The mock-up test for an annulus of a cylindrical connection may be conducted by demonstration on a straight wall. The size shall be maximized within practical limitations to realistically represent the typical grouted connection for which the test shall be taken as valid. As a general requirement, the width and the height of the wall shall not be taken as less than six (6) and four (4) metres respectively.

J.1.5.2 The thickness of the simulated annulus shall be conservatively dimensioned to be representative of the thinner annulus which will be encountered offshore. Various thicknesses may be covered in the same test setup. However the minimum thickness shall be represented over a significant portion of the width.

Guidance note:

For the case of mock-up tests for pre-piled jackets or similar applications with large annulus thickness, simulated annuluses representative of the larger grout thicknesses should be considered. Detailed requirements for mock-up tests for large annulus connections should be agreed in advance.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.5.3 The form shall be partially or wholly filled with water prior to filling with grout to simulate displacement of water in the offshore connection. It shall be ensured that a head of water is maintained above the grout during the whole operation. If a leakage of water from the form work is a risk special care shall be taken to not disturb the material when topping up the water.

J.1.5.4 Clean steel formwork shall be used to provide a realistic coefficient of friction between the grout and the walls of the form. The steel surfaces shall be prepared with a specified surface roughness to represent realistic conditions offshore.

J.1.5.5 Mock-up tests to be considered representative of grouted connections with shear keys shall include a number of horizontal shear keys during the test. The shear keys may be simulated by attaching a number of rows of 12-20 mm square shaped shear keys at a spacing of approximately 200 mm at the inside of the formwork. The shear keys shall be made of a non-absorbent material.

Guidance note:

A slight inclination (5-10°) of the shear key edges are recommended in order to ease formwork stripping without damaging the surrounding grout (prevent potential wedge effects).

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.5.6 A realistic hose length and pumping height expected for the particular application shall be simulated during the test. Minimum required hose length and pumping height should be taken as 100 metres and 20 meters respectively. The minimum nominal bore of the grout hoses should be tested as part of the mock-up tests.

Guidance note:

During hoisting with a crane to obtain the required pumping height a yoke should be used to avoid folding or kinking of the grout hoses during the test.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.5.7 The grout should enter the form at one single location. The length of lateral travel for the grout shall be maximized within practical limitations to simulate the most critical offshore installation case.

Guidance note:

In cases where a single point stinger operation is specified as the contingency procedure this will in most cases be the governing situation.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.5.8 If the formwork is prepared with varying annulus thickness, the grout inlet shall be positioned perpendicularly to the wall at a location where the simulated annulus thickness is not greater than 100 mm.

J.1.5.9 The centre of the grout inlet shall be positioned a minimum of 350 mm above the bottom of the formwork to simulate a realistic grout inlet found offshore. The location of the inlet is important to verify that limited washout of the material occurs during placement.

Guidance note:

In cases where a sealing material is used at the bottom of the form to prevent leakage this should be taken into consideration for the location of the inlet.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

J.1.5.10 The mixing and pumping of the lubrication mix and grout material should be conducted, within practical limitations, as it would be offshore with reference to temperature, mixing times, pumping duration and pressure. Pumping of lubrication mix into the connection or a skip container shall be as described in the generic offshore grouting procedure in question.

J.1.5.11 Pumping shall be suspended midway through the test for at least thirty (30) minutes to simulate conditions where a blockage, loss of power or equipment breakdown would be encountered offshore. This is to verify that pumping can recommence without any adverse effects to the pumpability or to the quality of the grouted connection.

J.1.5.12 The duration of pumping during the test should reflect the time taken to fill a typical grouted connection. The volume of the form as well as pump rates will affect the total pumping time, therefore due attention should be paid to this in the planning stage. The total duration, including the time for the stoppage, is important input for the overall assessment.

J.1.5.13 The lubrication and water mixture being pushed ahead of the grout material shall be evacuated prior to the pumping being halted.

J.1.5.14 In cases where the mock-up test is undertaken as part of certification of grout materials to be used at elevated temperatures (>30°C), in addition to standard and low (if applicable) temperatures, the maximum temperature shall be considered replicated during the test.

J.1.6 Required testing during mock-up test

J.1.6.1 QC testing during the mock-up test shall as a minimum be the same as that conducted offshore. The consistency/quality of the mixed grout should be assessed more aggressively during the mock-up test than one would consider for normal offshore specification.

J.1.6.2 As a minimum the following additional tests shall be conducted on the grout going into the pump and again on the grout taken from the top of the form after full filling has been achieved:

— flow (ASTM C230/C1437, refer to FG1 in App.H)
— density (EN 1015-6, refer to FG2 in App.H)
— bleeding (ASTM C940, refer to FG3 in App.H)
— air content (EN 1015-7, refer to FG4 in App.H)
— samples for strength testing (EN 12390-3 for compressive strength, refer to HG3 in App.H).

J.1.6.3 Shape and size of compressive strength specimens taken during the test shall be the same as those to be used offshore for normal quality control (typically 75 mm cubes).

J.1.6.4 Temperature of grout constituents (powder, mixing water), mixed grout and ambient temperature shall be recorded and documented.

J.1.6.5 The angle of flow of the material inside the form should be monitored periodically during the test. The elevation of the grout may be recorded manually with tape measures at a number of locations along the form.

J.1.6.6 Pump pressure and pump rate shall be monitored and documented periodically during the test.

J.1.6.7 In cases where the temperature development during initial curing is of interest, thermocouples may be installed in the annulus prior to grouting in order to monitor the temperature of the grout after placement.

J.1.7 Inspection after the mock-up test

J.1.7.1 After a period of hardening, determined by the material manufacturer, the form shall be removed for detailed inspection to verify full filling of grout. During the visual inspection of the wall after the formwork has been removed the following will be evaluated:

— flow pattern
— layering
— inclusions
— honey combing
— bleeding and segregation
— air and sand pockets
— shrinkage
— washout.

J.1.7.2 To assess compressive strength core specimens shall be taken from a number of heights throughout the wall. Additional core specimens shall be taken from areas with interesting surface anomalies and/or obvious layering of the grout.

J.1.7.3 Close visual examination of the grout in the vicinity of simulated shear keys shall be undertaken to verify full filling around the shear keys.

J.1.7.4 A sample of critical observations that may lead to non-satisfactorily test is listed below:

- Extensive through thickness cracking due to e.g. shrinkage.
- Significant washout from the bottom and upwards over a height greater than the smallest grout thickness (reference is made to effective length of grouted connection in DNVGL-ST-0126, [C.1.1.2]).
- Low compressive strength determined from core specimens.
- Inconsistent compressive strength determined from QC specimens.
- Large discrepancy between the compressive strength of QC specimens and the material in the wall.
- Blockage in hoses or inlet arrangement during grouting, or unable to recommence grouting after the thirty minutes pause..
- Accumulation of bleed water and/or voids around shear keys
- Improper filling based on items listed in [J.1.7.1].
- Lack of density control during continuous mixing of grout (if relevant) and/or the need for retrospective calibration of density logging system.
- Significant deviations between test execution and agreed test procedure.

J.1.8 Special requirements for continuous mixing

J.1.8.1 In cases where continuous mixing is used there are special requirements to be considered in addition to the other items described within this document. These are presented in the following paragraphs.

J.1.8.2 It shall be demonstrated that the mixer operator is able to control the density of the grout being pumped to the structure by adjustments on water/powder flow rates given the density readings at the mixing tank. The density of the grout being pumped to the structure shall be continuously recorded by a densitometer located after the pump. Recordings from densitometers shall be supported by and calibrated against fresh property testing undertaken in accordance with EN 1015-6.

J.1.8.3 It shall be shown that the operator at the mixer has full overview of the calibrated density control system and the ability to control the grout density within the acceptable range. During the calibration period, it shall be demonstrated that recirculation and/or mixing will not lead to over-mixing of the grout resulting in too high temperatures. It shall however be ensured that the grout density is calibrated at the mixer before material is released.

J.1.8.4 When dry powder is transported in silos and/or pneumatically transferred between silos, the stability of the dry powder shall be confirmed by material testing for the fresh and hardened properties. The samples for the material testing shall as a minimum be taken at the production facility and end of a transportation/ transfer operation representative of the upper limit expected for the material.

APPENDIX K SUPPLEMENTAL REQUIREMENTS FOR STEEL REINFORCEMENT SPECIFIED FOR USE WITH THIS STANDARD

K.1 General

K.1.1 Minimum requirements

K.1.1.1 In order to use this standard in design of offshore concrete structures, where steel reinforcement bars shall be used, the requirements of ISO 6935-2 shall be satisfied along with the supplemental provisions defined in this appendix.

K.1.1.2 The testing defined in [K.2.4] shall be undertaken during initial type testing of steel reinforcement and/or prior to the execution of the project.

K.1.1.3 For requirements related to evaluation of conformity and factory production control testing ISO 6935-2 shall be followed along with applicable national regulations. The mandrel diameters specified in [K.2.4] should be implemented in the evaluation of conformity and factory production control testing programmes. However, if bend and rebend testing use mandrel diameters larger than that specified in [K.2.4], in line with less strict national regulations for instance, then the production bend mandrel shall be minimum 1.5 times the test mandrel diameter for any given bar size.

Guidance note:

Suitable national standards, for instance EN 10080, may be used when specifying methodologies and programmes for evaluation of conformity and production control testing.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

K.1.1.4 For requirements related to testing for low temperature application of steel ribbed bars at temperatures less than -20°C an assessment shall be conducted.

Guidance note:

EN 14620-3 may be used for defining a low temperature testing programme.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

K.2 Requirements supplemental to ISO 6935-2

K.2.1 Reinforcement bar ribs

K.2.1.1 Transverse ribs shall be in accordance with the requirements of crescent-shaped ribs in ISO 6935-2 with the exception of requirements in [K.2.1].

K.2.1.2 Transverse rib height shall be minimum 0.05d and maximum 0.1d, where d is the nominal bar diameter.

K.2.1.3 Transverse rib spacing along the length of the bar shall be minimum 0.5d and maximum 0.8d, where d is the nominal bar diameter.

K.2.1.4 Relative rib area shall be minimum 0.056 for bars of 12 mm and larger nominal diameter.

K.2.1.5 The projected length of the transverse ribs shall be minimum 75% of the bar circumferential length of the bar. The circumference of the bar shall be taken as 3.14 times the nominal bar diameter.

K.2.1.6 Longitudinal ribs height shall not exceed 0.1d, where d is the nominal bar diameter.

K.2.2 Chemical composition

K.2.2.1 Chemical composition shall be in accordance with the requirements of ISO 6935-2 with the exception of requirements in [K.2.2].

K.2.2.2 The requirements for chemical composition (% by mass) based on cast analysis shall be supplemented with:

— maximum Copper (Cu) content of 0.80
— maximum Carbon (C) content may be taken as 0.25 if the maximum carbon equivalent (CEV) does not exceed 0.48
— aximum CEV for all bar sizes of 0.50
— maximum Nitrogen (N) content may be 0.019 if sufficient quantities of nitrogen binding elements are present.

K.2.2.3 The requirements for chemical composition (% by mass) based on product analysis shall be:

— maximum carbon (C) content of 0.24
— maximum silicon (Si) content of 0.65
— maximum manganese (Mg) content of 1.70
— maximum phosphorus (P) content of 0.055
— maximum nitrogen (N) content of 0.014
— maximum copper (Cu) content of 0.85
— maximum carbon (C) content may be taken as 0.27 if the CEV does not exceed 0.50
— maximum CEV for all bar sizes of 0.52
— maximum nitrogen (N) content may be 0.019 if sufficient quantities of nitrogen binding elements are present.

K.2.3 Mechanical properties

K.2.3.1 Reinforcing steel for use with this standard shall be of grade B500BWR or B500CWR. Typically B500CWR is specified for offshore concrete structures.

K.2.3.2 The requirements for tensile properties shall be in accordance with Table H-1.

Table K-1 Mechanical properties of steel reinforcement bars

Ductility class	*Steel grade*	*Specified characteristic value of upper yield strength (R_{eH})$^{1)}$ [MPa]*		*Ductility properties*		
				Specified characteristic value of (R_m/R_{eH})$^{1)2)}$	*Specified characteristic value at elongation [%]*	
		Min	*Max*	*Min*	*A, Min*	*$A_{gt}$$^{3)}$, Min*
B	B500BWR	500	650	1.08	-	5.0
C (d = 6mm to 14mm)	B500CWR	500	650	1.12	-	7.0
C (d = 16mm to 40mm)				1.15	-	8.0

Ductility class	*Steel grade*	*Specified characteristic value of upper yield strength (R_{eH})[1] [MPa]*		*Ductility properties*		
				Specified characteristic value of (R_m/R_{eH})[1)2)]	*Specified characteristic value at elongation [%]*	
		Min	*Max*	*Min*	*A, Min*	*A_{gt}[3] , Min*
1) R_{eH} = Upper yield strength 2) R_m = Tensile strength 3) A_{gt} = Percentage total elongation at maximum force						

K.2.4 Testing

K.2.4.1 Bars shall be tested for fatigue properties in an aged condition.

Guidance note:

Bent reinforcement may 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.

---e-n-d---o-f---g-u-i-d-a-n-c-e---n-o-t-e---

K.2.4.2 During the bend test the specimens shall be bent at least 180 degrees around a mandrel of diameter as presented in Table K-2.

Table K-2 Mandrel diameters for bend test

Nominal bar diameter, d	6	8	10	12	14	16	20	25	32	40
Mandrel diameter (max)	10	12	16	20	25	32	50	80	100	125

K.2.4.3 A rebending test with artificial aging between the first and second bending shall be performed. The mandrel diameters are presented in Table K-3.

Table K-3 Mandrel diameters for rebend test

Nominal bar diameter, d	6	8	10	12	14	16	20	25	32	40
Mandrel diameter (max)	20	25	32	40	50	63	100	200	250	320

K.2.4.4 A series of fatigue testing shall be performed, particulars are detailed in Table K-4.

Table K-4 Fatigue testing during initial type testing

Number of specimens	*Lower bound of stress range [MPa]*	*Upper bound of stress range [MPa]*	*Stress range [MPa]*	*Cycles without fracture (average)*
5	22	442	420	60000
5	15	305	290	220000
5	12	242	230	550000
5	10	200	190	3000000

K.2.4.5 The following bars tested for fatigue in accordance with Table K-4 shall be taken as representative of bars in the range in Table K-5.

Table K-5 Suggested bar diameters for fatigue testing

Bar tested [mm]	*Representative of diameters [mm]*
8	6 to 10
16	12 to 20
32	25 to 40

K.2.4.6 It is recommended that the production control testing include periodic verification of fatigue properties. Bars should in that case be tested at the lowest stress range and maximum number of cycles defined in Table K-4.

CHANGES – HISTORIC

There are currently no historical changes for this document.

About DNV GL
Driven by our purpose of safeguarding life, property and the environment, DNV GL enables organizations to advance the safety and sustainability of their business. We provide classification, technical assurance, software and independent expert advisory services to the maritime, oil & gas and energy industries. We also provide certification services to customers across a wide range of industries. Operating in more than 100 countries, our experts are dedicated to helping our customers make the world safer, smarter and greener.

SAFER, SMARTER, GREENER