# INTERNATIONAL STANDARD

# ISO 19903

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## Petroleum and natural gas industries — Fixed concrete offshore structures

*Industries du pétrole et du gaz naturel — Structures en mer fixes en béton*

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# Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 2.

The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights.

ISO 19903 was prepared by Technical Committee ISO/TC 67, *Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries*, Subcommittee SC 7, *Offshore structures*.

ISO 19903 is one of a series of standards for offshore structures. The full series consists of the following International Standards.

- ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*
- ISO 19901 (all parts), *Petroleum and natural gas industries — Specific requirements for offshore structures*
- ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures* [1)]
- ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*
- ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars*
- ISO 19904-2, *Petroleum and natural gas industries — Floating offshore structures — Part 2: Tension leg platforms* [2)]
- ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups* [2)]
- ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary* [2)]
- ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures* [2)]

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1) To be published.

2) Under preparation.

# Introduction

The series of International Standards applicable to offshore structures, ISO 19900 to ISO 19906, constitutes a common basis covering those aspects that address design requirements and assessments of all offshore structures used by the petroleum and natural gas industries worldwide. Through their application the intention is to achieve reliability levels appropriate for manned and unmanned offshore structures, whatever the type of structure and nature or combination of the materials used.

It is important to recognize that structural integrity is an overall concept comprising models for describing actions, structural analyses, design rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one aspect of design in isolation can disturb the balance of reliability inherent in the overall concept or structural system. The implications involved in modifications, therefore, need to be considered in relation to the overall reliability of all offshore structural systems.

The series of International Standards applicable to the various types of offshore structure is intended to provide wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. Sound engineering judgement is therefore necessary in the use of these International Standards.

International Standard ISO 19903 was developed based on experience gained from the design, execution and use of a number of fixed concrete platforms, in particular from more than 30 years of experience with such structures in the North Sea. The background documents when developing this International Standard are from the following types of documents:

— national regulations and other requirements from the authorities;

— regional standards;

— national standards;

— operator’s company specifications;

— scientific papers and reports;

— reports from inspection of structures in use.

This International Standard draws on the experience gained with fixed concrete offshore structures. This experience shows that fixed concrete offshore structures perform well and are durable in the marine environment. These structures are all unique, one-of-a-kind structures, purpose-made for a particular location and a particular set of operating requirements. This is reflected in ISO 19903 by the fact that the standard gives guidance rather than detailed prescriptive rules. This International Standard reflects in particular the experience and the conditions in the North Sea and the east coast of Canada, and the design rules and practices used there, but is intended for worldwide application.

# Petroleum and natural gas industries — Fixed concrete offshore structures

## 1 Scope

This International Standard specifies requirements and provides recommendations applicable to fixed concrete offshore structures for the petroleum and natural gas industries, and specifically addresses

a) the design, construction, transportation and installation of new structures, including requirements for in-service inspection and possible removal of structures,

b) the assessment of structures in service, and

c) the assessment of structures for reuse at other locations.

This International Standard is intended to cover the engineering processes needed for the major engineering disciplines to establish a facility for offshore operation. It can also be used for the design of floating concrete structures as specified in ISO 19904-1 [11] (and the future ISO 19904-2 [12] when published) and for arctic structures (as specified in the future ISO 19906 [7] when published).

In order to provide a standard that will be useful to the industry, a comprehensive treatment of some topics is provided where there is currently no relevant reference. For such well-known topics as the design formulas for concrete structural members, this International Standard is intended to be used in conjunction with a suitable reference standard for basic concrete design (see 8.2.1). The designer can use suitable national or regional design standards that provide the required level of safety. Only other ISO documents will be referenced directly in the text.

## 2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 1920-3, *Testing of concrete — Part 3: Making and curing test specimens*

ISO 1920-4, *Testing of concrete — Part 4: Strength of hardened concrete*

ISO 2394, *General principles on reliability for structures*

ISO 4463-1, *Measurement methods for building — Setting-out and measurement — Part 1: Planning and organization, measuring procedures, acceptance criteria*

ISO 6934 (all parts), *Steel for the prestressing of concrete*

ISO 6935 (all parts), *Steel for the reinforcement of concrete*

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 19901-5, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 5: Weight control during engineering and construction*

ISO 19901-6, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 6: Marine operations*[3)]

ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures* [3)]

## 3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900 and the following apply.

NOTE Terms and definitions relevant for the use of this International Standard are also found in ISO 19901-1, ISO 19901-2, ISO 19901-4 and ISO 19901-6 and in ISO 19902.

**3.1**
**abnormal design situation**
design situation in which conditions exceed conventionally specified design conditions and which is used to mitigate against very remote events

NOTE Abnormal design situations are used to provide robustness against events with a probability of typically $10^{-4}$ per annum or lower by avoiding, for example, gross overloading.

[ISO 19901-2]

**3.2**
**abnormal level earthquake**
**ALE**
intense earthquake of abnormal severity under the action of which the structure should not suffer complete loss of integrity

NOTE The ALE event is comparable to the abnormal event in the design of fixed structures which are described in ISO 19902 and ISO 19903. When exposed to the ALE, a manned structure is supposed to maintain structural and/or floatation integrity for a sufficient period of time to enable evacuation to take place.

[ISO 19901-2]

**3.3**
**accidental design situation**
design situation involving exceptional conditions of the structure or its exposure

EXAMPLE Impact, fire, explosion, local failure or loss of intended differential pressure (e.g. buoyancy).

**3.4**
**action**
external load applied to the structure (direct action) or an imposed deformation or acceleration (indirect action)

NOTE 1 An imposed deformation can be caused by fabrication tolerances, settlement, temperature change or moisture variation.

3) To be published.

NOTE 2 An earthquake typically generates imposed accelerations.

[ISO 19900]

**3.5**
**action effect**
effect of action on structural components

EXAMPLE Internal force, moment, stress or strain.

[ISO 19900]

**3.6**
**addition**
finely divided material used in concrete in order to improve certain properties or to achieve special properties

NOTE This International Standard deals with two types of inorganic additions:

— nearly inert additions (type I);

— pozzolanic or latent hydraulic additions (type II).

**3.7**
**admixture**
material added during the mixing process of concrete in small quantities related to the mass of cement to modify the properties of fresh or hardened concrete

**3.8**
**after-damage design situation**
design situation for which the condition of the structure reflects damage due to an accidental design situation and for which the environmental conditions are specially defined

**3.9**
**aggregate**
granular mineral material suitable for use in concrete

NOTE Aggregate can be natural, artificial or recycled from material previously used in construction.

**3.10**
**air cushion**
air pumped into underbase compartments of the structure

NOTE Normally applied in order to reduce the draft and increase the freeboard of the structure and/or to alter the structural loading.

**3.11**
**atmospheric zone**
part of the load-bearing structure that is above the splash zone

**3.12**
**caisson**
major portion of fixed concrete offshore structure, providing buoyancy during floating phases and the possibility of oil storage within the structure

NOTE The caisson is generally divided into watertight compartments, which can be subdivided into intercommunicating cells for structural reasons. The caisson can also be filled, or partly filled, with ballast water and solid ballast.

**3.13**
**characteristic value of a material property**
value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series, a nominal value being used as the characteristic value in some circumstances

NOTE The characteristic material property generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. Characteristic strength is normally defined as the value of the strength below which 5 % of the population of all possible strength determinations of the material under consideration are expected to fall or, alternatively, 95 % if an upper value is more severe.

**3.14**
**critical shear zone**
zone in which the shear stress is at a maximum in relation to the shear strength

**3.15**
**concrete**
material formed by mixing cement, coarse and fine aggregate and water, with or without the incorporation of admixtures and additions, which develops its properties by hydration of the cement

**3.16**
**condition monitoring**
evaluation of the condition and behaviour of the load-bearing structure(s) in service using data from design, inspection and instrumentation

**3.17**
**construction afloat**
fabrication, construction and related activities taking place on a structure that is afloat, normally at an inshore location and restrained by a temporary mooring system

**3.18**
**deck mating**
marine operation in which the platform topsides is floated into position and connected to the substructure

NOTE This operation is normally conducted by ballasting and deballasting of the substructure.

**3.19**
**deep water construction site**
site for construction of the structure while afloat

NOTE The use of a deep water site might not always be required, depending on the construction method. It might or might not be the same location as that where mating of topsides to the substructure takes place.

**3.20**
**design rules**
rules in accordance with the chosen reference standard for concrete design

NOTE See 8.2.

**3.21**
**dynamic amplification factor**
**DAF**
ratio of a dynamic action effect to the corresponding static action effect

NOTE An appropriately selected dynamic amplification factor can be applied to static actions to simulate the effects of dynamic actions.

**3.22**
**extreme level earthquake**
**ELE**
earthquake with a severity which the structure should sustain without major damage

NOTE The ELE event is comparable to the extreme environmental event in the design of fixed structures which are described in ISO 19902 and ISO 19903. When exposed to an ELE, a structure is supposed to retain its full capacity for all subsequent conditions.

[ISO 19901-2]

**3.23**
**execution**
all activities carried out for the physical completion of the work including procurement, inspection and documentation thereof

NOTE The term covers work on site; it might also signify the fabrication of components off-site and their subsequent erection on site.

**3.24**
**exposure level**
classification system used to define the requirements for a structure based on consideration of life safety and of environmental and economic consequences of failure

NOTE The method for determining exposure levels is described in ISO 19902. An exposure level 1 platform is the most critical and exposure level 3 the least. A normally manned platform which cannot be reliably evacuated before a design event will be an exposure level 1 platform.

[ISO 19900]

**3.25**
**finite element analysis**
**FEA**
analysis method whereby a structure or a part thereof is subdivided into small elements of known or assumed behaviour, then analysed by numerical matrix methods to determine action effects, static or dynamic

**3.26**
**fixed concrete offshore structure**
**FCS**
concrete structure designed to rest on the sea floor

NOTE Sufficient structural stability can be achieved through its own weight, or in combination with suction in skirt compartments, or founding of the structure on piles into the seabed. It includes the mechanical outfitting of the structure.

**3.27**
**fixed structure**
structure that is bottom founded and transfers all actions on it to the seabed

[ISO 19900]

**3.28**
**float-out**
transfer of a major assembly from a dry construction site to a self-floating condition

NOTE Typically, it is the transfer of the lower part of the concrete structure from a flooded drydock.

**3.29**
**global analysis**
determination of a consistent set of either internal forces and moments or of stresses in a structure that are in equilibrium with a defined set of actions on the entire structure and which depend on geometrical, structural and material properties

NOTE For a global analysis of a transient situation (e.g. seismic), the internal response is part of the equilibrium.

**3.30**
**inspection**
conformity evaluation by observation and judgement accompanied, as appropriate, by measurement, testing or gauging to verify that the execution is in accordance with the project work specification

**3.31**
**installation**
marine operation in which the platform is positioned and set down on the sea floor at the offshore site

**3.32**
**instrumentation**
outfitting of a fixed concrete offshore structure with instruments for data measurement and recording

**3.33**
**interface manual**
document defining all interfaces between the various parties and disciplines involved in the design and construction, ensuring that responsibilities, reporting and information routines, as appropriate, are established and maintained

**3.34**
**lightweight aggregate**
aggregate of mineral origin having an oven-dry particle density $\leqslant$ 2 000 kg/m$^3$ or a loose oven-dry bulk density $\leqslant$ 1 200 kg/m$^3$

**3.35**
**local analysis**
determination of a consistent set of internal forces and moments, or stresses, in a cross-section of a structural component, or in a subset of structural components forming part of the structural system, that are in equilibrium with the boundary conditions

**3.36**
**marine operation**
planned and controlled vertical or horizontal movement of a structure or component thereof over, in or on water

**3.37**
**method statement**
document stating the methods and procedures to be used to perform the work

**3.38**
**normal-weight aggregate**
aggregate with an oven-dry particle density between 2 000 kg/m$^3$ and 3 000 kg/m$^3$

**3.39**
**offshore site**
offshore location where the structure is to be installed for its operational life

**3.40**
**operations manual**
document giving the requirements and restrictions related to a safe operation of the concrete structure and all its systems

**3.41**
**owner**
representative of the companies which own a development

NOTE The owner will normally be the operator on behalf of co-licensees.

**3.42**
**primary structure**
all main structural components (concrete or steelwork) that provide the structure's main strength and stiffness

**3.43**
**procedure**
document that describes a specified way to carry out an activity or a process, the detailed sequence and inter-relationships required for the completion of a particular task

**3.44**
**project specification**
document giving the overall technical requirements provided by the owner

**3.45**
**project work specification**
all information and technical requirements necessary for the execution of the works, includes documents and drawings, etc. as well as references to relevant regulations, specifications, etc.

**3.46**
**quality plan**
document specifying which procedures and associated resources shall be applied by whom and when, covering the entire project or defined parts of the project and all relevant products, processes or contracts

**3.47**
**secondary structure**
structural components that do not contribute significantly to the overall strength and stiffness of the structure but which support individual items of equipment, transferring the actions thereon onto the primary structure

**3.48**
**shaft**
compartment extending from the caisson of the fixed concrete offshore structure to the topsides

NOTE A shaft is generally used to house and support the wells (drill shaft), mechanical systems (utility shaft) and risers and J-tubes (riser shaft). The part of a shaft extending above a caisson is also often referred to as a leg.

**3.49**
**skirts**
structural components constructed in concrete and/or steel that extend from the foundation downwards and penetrate into the seabed

NOTE Skirts are used to increase the capacity of the foundation to resist vertical and horizontal actions and improve erosion resistance. Skirts can also be needed to form compartments facilitating the under-base grouting.

**3.50**
**solid ballast**
non-structural material added to a structure

NOTE Solid ballast is normally applied in order to increase the self weight of the structure or to lower the centre of gravity for floating stability purposes.

**3.51**
**splash zone**
area of a structure that is frequently wetted due to waves and tidal variations

[ISO 19900]

**3.52**
**structure**
organized combination of connected parts designed to withstand actions and provide adequate rigidity

[ISO 19900]

**3.53**
**submerged zone**
part of the structure that is normally submerged and exposed to the constant influence of sea water

**3.54**
**subsidence**
that part of the settlement of the structure that results from extraction of reservoir hydrocarbons and factors other than the weight of the structure

**3.55**
**summary report**
document including the most important assumptions on which the design, construction and installation work is based with regard to the load-bearing structure

**3.56**
**topsides**
structures and equipment placed on a supporting structure (fixed or floating) to provide some or all of a platform's functions

NOTE A separate fabricated deck or module support frame is part of the topsides.

[ISO 19900]

**3.57**
**tow to field**
marine operation in which the complete platform or structure is moved from the dry dock or inshore construction site to the offshore site

**3.58**
**works**
construction work described in the project work specification

**3.59**
**works certificate**
**mill certificate**
document issued by the manufacturer or a testing institute certifying the materials delivered, and giving

- test method, specifications and criteria (e.g. test standard used),
- all relevant test data,
- certification that the tests have been carried out on samples taken from the delivered products, and
- all necessary information for identification of product, producer and purchaser.

NOTE A works certificate is normally required for construction materials that are not subject to an accepted certification scheme.

## 4 Symbols and abbreviated terms

### 4.1 Symbols

$A$ accidental action

$A_c$ actual surface area to be protected

$C_a$ total current capacity of the anodes

| | |
|---|---|
| $D$ | action due to imposed deformation |
| $E$ | environmental action |
| $E^{o}_{a}$ | design closed-circuit anode potential |
| $E^{o}_{c}$ | design protective potential |
| $G$ | permanent action |
| $I_{a}$ | anode current output |
| $I_{a,initial}$ | initial current output |
| $I_{a,final}$ | final current output |
| $I_{c}$ | current demand |
| $I_{c,average}$ | average current demand |
| $I_{c,initial}$ | initial current demand |
| $I_{c,final}$ | final current demand |
| $L$ | lap length |
| $M_{x}$, $M_{y}$, $M_{xy}$ $N_{x}$, $N_{y}$, $N_{xy}$ | six force components giving stresses in the plane of the member |
| $R$ | radius |
| $R_{a}$ | anode resistance |
| $Q$ | variable action |
| $a$ | mass content of the active addition (type II) |
| $c$ | cement mass content |
| $c_{a}$ | current capacity of an anode |
| $f_{c}$ | coating breakdown factor for any coated surfaces ($f_{c}$ = 1 for bare steel) |
| $f_{cd}$ | design compressive strength of concrete |
| $f_{ck}$ | characteristic compressive strength of concrete |
| $f_{cn}$ | nominal compressive strength of concrete |
| $f_{yk}$ | characteristic strength of steel |
| $i_{c}$ | design current density |
| $k$ | factor which takes into account the activity of a type II addition |
| $m$ | effective water/cement ratio |
| $m_{T}$ | total net anode mass |

| | |
|---|---|
| $n$ | number of anodes |
| $t$ | thickness |
| $u$ | utilization factor of the anode |
| $w$ | water mass content in concrete |
| $t_f$ | design life of the cathodic protection system |
| $\varepsilon$ | anode material's electrochemical efficiency |
| $\nu$ | Poisson ratio |
| $\gamma_F$ | partial factor for action taking account of model and geometrical uncertainties |
| $\gamma_M$ | partial factor for material resistance properties taking account of material, model and geometric uncertainties |
| $\gamma_G$ | partial factor for permanent actions, also accounting for dimensional variations |
| $\gamma_Q$ | partial factor for variable actions |
| $\gamma_E$ | partial factor for environmental actions |
| $\gamma_D$ | partial factor for actions resulting from imposed deformations |
| $\gamma_A$ | partial factor for accidental actions |

## 4.2 Abbreviated terms

| | |
|---|---|
| AAR | alkali aggregate reaction |
| ALE | abnormal level earthquake |
| ALS | accidental limit state |
| CCTV | closed-circuit television |
| CFD | computational fluid dynamics |
| ELE | extreme level earthquake |
| FCS | fixed concrete offshore structure |
| FLS | fatigue limit state |
| GRP | glass-fibre reinforced plastic |
| HAT | highest astronomical tide |
| HAZOP | hazard and operability analysis |
| HISC | hydrogen-induced stress cracking |
| HVAC | heating, ventilation and air conditioning |
| IC | inspection class |

LAT lowest astronomical tide

MIC microbiologically induced corrosion

ROV remotely operated vehicle

SCC self-compacting concrete

SLS serviceability limit state

SRB sulfate-reducing bacteria

ULS ultimate limit state

# 5 General requirements

## 5.1 General

Fixed concrete offshore structures shall be designed in accordance with ISO 19900, this International Standard and the requirements given in the project specification.

The structure shall be designed, constructed, transported and installed in such a way that

— the installed structure meets the intended reliability level, and

— all functional and structural requirements are met.

General principles for the verification of the reliability of the structure shall be in accordance with ISO 2394.

This International Standard assumes that the owners will operate an organization that supervises and monitors the project, and ensures that an appropriate level of independent verification of design and construction is performed.

## 5.2 National requirements

National regulations and standards applicable in the place where a structure will be used can be different from those given in this International Standard. In such cases it shall be ensured that the requirements of safety, reliability and durability implicit from the requirements of this International Standard are met. This applies to all phases of planning, design, construction, transportation, installation, service in-place and possible removal.

## 5.3 Overall planning requirements

### 5.3.1 General

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

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

All functional requirements affecting the layout and design of the structure shall be established in a clear format such that these can form the basis for the engineering process and the structural design.

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

### 5.3.2 Quality systems

The quality system shall comply with the requirements of ISO 19900 and specific requirements quoted for the various engineering disciplines in this International Standard.

All work performed in accordance with this document shall be subject to quality control in accordance with an implemented quality plan. The quality plan shall be in accordance with an internationally accepted quality system, for example, the ISO 9000 series. There can be one quality plan covering all activities or one overall plan with separate plans for the various phases and activities to be performed.

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 ensures that responsibilities, reporting and information routines are established as appropriate.

### 5.3.3 Qualifications of personnel

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

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

### 5.3.4 Documentation

Documentation shall be prepared for all activities that are to be performed in the engineering, design, construction, transportation, installation and possible removal of fixed concrete offshore structures. This includes investigations establishing data on which the design is based, engineering reports, design reports and calculations, drawings, specifications, etc., sufficient to give complete information about the structure. Documentation shall also be prepared showing records of all inspection and control of materials used and execution work performed that has an impact on the quality of the final product.

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

The assumptions on which the design, construction and installation work is based with regard to the load-bearing structures 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.

An operations manual shall be prepared giving all necessary information for the safe operation of the fixed concrete offshore structure including all systems.

## 5.4 Functional requirements

### 5.4.1 General

The engineering of a fixed concrete offshore structure shall be performed in such a way that all functional and operational requirements relating to its safety and its operation as an offshore platform are met.

The functional requirements affect the layout of the structure as well as the load cases that have to be considered in the design of the structure. The functional requirements are related both to the site-specific conditions and to the structure as a production or storage facility for the production of hydrocarbons, or any other activities in the operation of a field.

### 5.4.2 Site-related functional requirements

#### 5.4.2.1 Position on site

The structure shall be positioned and oriented on site such that its orientation takes account of the reservoir, construction requirements, other platforms in the vicinity, accessibility by ships and helicopters, and safety in case of fire or leakages of hydrocarbons. Position tolerance is to be defined by the owner.

#### 5.4.2.2 Environmental considerations

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

The deck elevation shall be determined such that it provides an adequate air gap, based on site-specific data, allowing the passage of extreme wave crests higher than the design wave crest, in accordance with ISO 19901-1. Due account shall be taken of wave crest modifications caused by the structure, caisson effect, local or regional features of the sea floor and wave run-up along the shafts.

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

### 5.4.3 Platform operational requirements

The functional requirements to be considered that are related to the production system include

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

b) storage volume, compartmentalization, 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, and

f) provisions for supply boats and other vessels servicing the platform.

All hazards (fire, explosions, loss of intended pressure differentials, flooding, leakages, rupture of pipe systems, falling objects, ship impacts, etc.) that can be anticipated during operations shall be established and evaluated. The structure shall be designed to give adequate safety to personnel, safety against damage to the structure or pollution of the environment.

## 5.5 Structural requirements

### 5.5.1 General

Structures and structural members shall perform satisfactorily during all design situations, with respect to structural strength, 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 the mechanical and other installations that are needed for it to operate. Adequate performance shall be demonstrated in design documentation.

#### 5.5.2 Structural concept requirements

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

a) has adequate robustness with low sensitivity to local damage,

b) can be constructed in a controlled manner,

c) provides simple stress paths that limit stress concentrations,

d) is adequately protected from corrosion and other degradation,

e) is suitable for condition monitoring, maintenance and repair, if required, and

f) fulfils requirements for decommissioning and removal if required.

#### 5.5.3 Materials requirements

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

It shall be ensured that the specified quality of the materials, all structural components and the physical structure itself is maintained during all stages of construction.

#### 5.5.4 Execution requirements

Requirements for execution, testing and inspection of the various parts of the structure shall be specified on the basis of the significance of the various parts with regard to the overall safety of the completed and installed structure as well as to the structure in temporary phases.

#### 5.5.5 Temporary phases requirements

The structure shall be designed for all phases with the same intended reliability (per annum) as for the final condition unless otherwise agreed. This applies also to temporary moorings or anchorage systems applied to those construction phases when the structure is afloat.

For all floating phases during marine operations or construction, sufficient positive stability and reserve buoyancy shall be ensured. Both intact and damaged stability shall be evaluated on the basis of an accurate geometric model. Adequate freeboard shall be provided. One-compartment damage stability should normally be provided. For short transient phases, the one-compartment damage stability may be waived, provided this can be justified by a risk analysis.

Weight control required for temporary phases should be performed by means of a well-defined, documented, robust and proven weight control method. Procedures shall be in accordance with ISO 19901-5. The system should provide up-to-date weight reports containing the necessary data for all operations.

### 5.6 Design requirements

#### 5.6.1 General

The structural design of a fixed concrete offshore structure and its foundation design shall be in accordance with this International Standard. The design shall be performed according to the principles of limit state design as defined in ISO 19900.

The design shall provide adequate strength and tightness in all design situations such that the assumptions made are complied with.

The design of structural steel components shall be in accordance with ISO 19902.

### 5.6.2 Design actions

The representative values of actions shall be selected according to 6.5.

The partial factors for actions shall be chosen with respect to the limit states and the combination of actions concerned. Values are given in 6.4.

### 5.6.3 Design resistance

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

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

### 5.6.4 Characteristic values for material strength

The characteristic strength of materials shall be determined according to the relevant design standards and recognized standards for material testing.

For concrete, the 28 days characteristic compressive strength $f_{ck}$ is defined as a 5 % fractile value (5th percentile) found from statistical analysis of testing 150 mm × 300 mm cylindrical specimens.

NOTE In some standards (e.g. NS 3473.E [2]), a nominal compressive strength $f_{cn}$ is used, which is less than $f_{ck}$; this considers transition of test strength into *in situ* strength and ageing effects due to high sustained stresses. If this nominal value is used for the calculation of design strength, reduced partial factors for materials reflecting this can be applied.

For reinforcement steel, the specified minimum yield stress shall be taken as the characteristic strength $f_{yk}$; for prestressing, the 0,1 % proof stress may be applied.

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

For the fatigue limit state FLS, the characteristic material strength shall be determined statistically as a 5 % fractile for reinforcement, prestressing assemblies, couplers, welded connections, etc. unless other values are specified in the reference standard for design. For concrete, normally a design reference strength shall be used. For soil, the characteristic strength shall be used. For other materials, acceptance criteria shall be specified which offer a safety level equivalent to that of the requirements in this International Standard.

Where high resistance of a member is unfavourable (e.g. in weak link considerations), an upper value of the characteristic resistance shall be used in order to give a low probability of failure of the adjoining structure. The upper value shall be chosen with the same level of probability of exceedance as the probability of lower values being underscored. In such cases, the partial factor for material shall be 1,0 for calculating the resistance that is used when actions are applied on adjoining members.

### 5.6.5 Partial factors for materials

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

For steel members in the mechanical outfitting, embedment, skirts, etc., the partial resistance factor shall be in accordance with ISO 19902.

Foundation design shall be performed in accordance with Clause 9. The partial factor for soil material shall normally not be lower than 1,25.

#### 5.6.6 Design by testing

Where actions acting on a structure or the resistance of materials or structural members cannot be determined with reasonable accuracy, model tests should be considered.

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

A test structure, a test structural detail or a test model shall be sufficiently similar to the structure to be considered. The results of the tests shall provide a basis for a reliable interpretation, in accordance with a recognized standard; scale effects shall be taken into account where relevant.

## 6 Actions and action effects

### 6.1 General

#### 6.1.1 Classification of actions

In accordance with ISO 19900, actions are classified as

— permanent actions ($G$),

— variable actions ($Q$),

— environmental actions ($E$), and

— accidental actions ($A$).

In this International Standard, an additional category is distinguished, being

— actions resulting from imposed deformations ($D$).

The actions shall include the corresponding external reactions. The representative actions shall be chosen according to the design situation under investigation. The following design situations are distinguished:

— normal operations;

— temporary situations;

— accidental situations;

— abnormal situations;

— damaged situations.

#### 6.1.2 Determination of action effects

Action effects shall be determined by means of recognized methods that take into account the variation of the action in time and space, the configuration and stiffness of the structure, relevant soil conditions and the limit state that is under consideration.

Simplified methods for computing actions may be applied if it can be verified that they produce conservative results.

Dynamic or non-linear effects shall be considered where appropriate.

Hydro- and aerodynamic actions and the associated action effects shall be determined by methods which take the kinematics of the liquid or air into account. For hydrodynamic actions the interaction between liquid, structure and soil shall be considered. For calculation of overall action effects from wind, simplified models normally suffice.

Seismic actions shall be considered in accordance with ISO 19901-2 and 7.5.7.

When determining the soil reactions used in the calculation of action effects in the structure, the soil-structure interaction shall be accounted for. 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 sea floor, degrees of elasticity and plasticity in the soil and, if relevant, in the structure.

## 6.2 Environmental actions

### 6.2.1 General

Wind, wave, tide and current are important sources of environmental actions ($E$) on many structures located offshore. In addition, depending on location, seismic actions (6.2.6) or ice actions (6.2.7) or both can be significant environmental actions.

### 6.2.2 Wind, wave and current actions

The determination of actions due to wind, wave and current requires an appropriate description of the physical environment in the form of sea state severity and direction, associated wind velocity 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 the calculation of actions is described in ISO 19901-1.

Actions from wind, wave and current occur as a result of various mechanisms. The most important sources of action are:

— viscous or drag effects, which are generally of most importance for relatively slender bodies;

— inviscid effects due to inertia of the water particles and wave diffraction, which are generally of most importance in terms of global effects for relatively large-volume bodies.

For fixed concrete offshore structures, static analyses can be adequate, but the possibility that dynamic analysis is required for local behaviour of components or for the global behaviour of the whole platform shall be investigated. In the specific case of wave action, the possibility that non-linear effects can lead to responses at frequencies either above or below the frequency range in the wave spectrum shall be investigated. This applies to the as-installed situation at the permanent location as well as to temporary floating situations. Potential dynamic effects due to local or global actions from wind and current shall also be investigated.

The influence of the structure on the instantaneous water surface elevation shall be investigated. The possibility of direct impact of greenwater on topsides or shafts shall also be investigated. Total water surface elevation depends on storm surge and tide, on the crest height of incident waves, and on the interaction of the incident waves with the structure.

Environmental actions due to wind, wave and current relate particularly to ultimate limit state requirements. In addition, these actions can contribute to the fatigue, serviceability, and accidental limit states. Environmental actions due to wind, wave and current shall also be considered for temporary configurations of the structure during construction, tow and installation. The complete design life cycle of the structure, from initial construction to removal, shall be considered and appropriate combinations of wind, wave and current shall be determined to define design situations for all phases.

### 6.2.3 Extreme wave action

#### 6.2.3.1 General

Wave actions from extreme conditions shall be determined by means of an appropriate analysis procedure, supplemented, if required, by a model test programme. Global actions on the structure shall be determined. In addition, local actions on various appurtenances, attachments and components shall also be determined.

The appropriate analysis procedure for computing wave actions generally depends on the ratio of wavelength to a characteristic dimension of the structure, such as the diameter of a cylinder or caisson. For ratios less than approximately five, 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 the Morison equation may be used. 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 actions on various external attachments to a structure, both procedures can be required.

Model testing should be considered for supplementing analytical results, particularly in cases where it is anticipated that non-linear effects can be significant, or where previous experience is not directly applicable because of the configuration of the structure.

#### 6.2.3.2 Slender-body theory

The Morison equation can be appropriate for application to determine wave actions on slender members. The local wave kinematics are considered to be unaffected by the presence of the structural component under investigation, but local kinematics can be significantly influenced by adjacent structures.

The required inertia and drag coefficients for application in the Morison equation shall be based on recognized procedures such as those given in ISO 19902.

The Morison equation shall be applied

— with a regular (single-period) extreme wave;

— with irregular sea states in the time domain;

— with spectral representations in the frequency domain using appropriate linearization of the drag term.

The Morison equation may also be applied to calculate local actions from the kinematics around a global structure derived from diffraction theory.

The Morison equation does not account for various non-linear or higher order interactions between wave and structure such as slamming or ringing.

#### 6.2.3.3 Diffraction analysis

##### 6.2.3.3.1 General

Global actions on large-volume bodies shall generally be determined by applying a validated diffraction analysis procedure. In addition, local kinematics, required in the design of various appurtenances, shall be evaluated, including incident kinematics, diffraction and (if necessary) radiation effects.

The fundamental assumption of diffraction analysis 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 structure can then be predicted based on linearized three-dimensional potential theory.

#### 6.2.3.3.2 Methods

Analytical procedures shall generally be implemented 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 method may be considered in specialized cases. Programs should be validated by comparisons with model tests and/or verified with published data.

#### 6.2.3.3.3 Special considerations for panel methods

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 of relatively high gradients, denser grids shall be employed. By way of example, in the vicinity of the free surface grid densities will generally need to be increased. Grid densities shall also be related to the wave period in order to provide an adequate description of fluctuations over the wavelength. In general, convergence tests with grids of variable density shall be carried out to confirm the adequacy of any proposed panel model.

Diffraction models shall be combined with Morison models in the assessment of various relatively slender attachments to large-volume structures. Diffraction methods provide the required kinematics field, the Morison equation may be applied to compute resulting actions on slender attachments.

The proximity of additional relatively large-volume structures shall be included in assessing actions. Disturbances to the kinematics field around two or more structures can interact and this interaction shall be accounted for in the analysis.

Structures with significantly varying cross-section near the waterline, within the likely wave-affected zone, call for additional consideration. Structures that are not wall-sided across the waterline are not consistent with the underlying assumptions of linear diffraction theory; both local and global actions as well as action effects can be significantly non-linear relative to the magnitude of the sea state.

The calculation of actions caused by waves on surface-piercing structures that will be overtopped by the progressing wave need special attention and validation of the calculation method is necessary.

Careful consideration shall be given to possible pressure fluctuations on the base of a structure during the passage of a wave field. If the foundation conditions are such that pressure fluctuations are expected to occur on the base, then such pressure fluctuations shall be included in the analysis.

Diffraction analysis programs may be used to determine coefficients required in the evaluation of various non-linear effects, typically involving sum and/or difference frequencies.

#### 6.2.3.4 Additional requirements for dynamic analysis

In cases where the structure can respond dynamically, the additional effects associated with the motions of the structure shall be determined. Typically, these additional effects shall be captured in additional inertia and damping terms in the dynamic analysis. Structures can, for example, respond dynamically in the as-installed situation due to wave or seismic actions, or in temporary floating situations due to wave or wind actions.

Ringing can control the extreme dynamic response of particular types of fixed concrete offshore structures. A ringing response resembles that generated by an impulse excitation of a linear oscillator; it features a rapid build-up and slow decay of the response at the resonant period of the structure. Ringing is excited by non-linear (second, third and higher order) processes in the actions due to waves that are only a small part of the total applied environmental actions on a structure.

The effects of motions of the structure on internal fluids such as ballast water in tanks shall also be evaluated. Sloshing in tanks generally affects the pressures, particularly near the free surface of the fluid.

#### 6.2.3.5 Model testing

##### 6.2.3.5.1 The role of model testing

The necessity of model tests to determine extreme wave actions shall be determined on a case-by-case basis. Generally, model tests shall be considered when the following is required.

— Verification of analytical procedures: model tests should be performed to confirm the results of analytical procedures, particularly for cases with structures of unusual shape, for structures in shallow water with steep extreme waves, or for any other case where known limitations of analytical procedures are present.

— Complementing analytical procedures: model tests should be performed where various effects such as ringing, wave run-up, potential occurrence of deck slamming are suspected, or in cases where the higher order terms that are neglected in analytical procedures can be important. These effects cannot normally be assessed by the basic analytical procedure.

##### 6.2.3.5.2 Scaling of model tests

Froude scaling is normally appropriate for typical gravity-driven processes like waves acting on large-volume structures. However, in any decision to apply Froude scaling, the possible influence of viscosity and Reynolds number effects should be considered.

##### 6.2.3.5.3 Validation of model tests

Actions determined by model test shall be validated by comparison with analytical solutions or with the results of prior appropriate test programmes.

##### 6.2.3.5.4 Estimation of actions

When model tests are performed, appropriate test data shall be recorded to facilitate computation of wave actions. Data that can be appropriate include

— the time history of the local instantaneous air/water surface elevation at various locations,

— local particle kinematics,

— global actions such as base shear, vertical load or overturning moment, as well as local actions in the form of the pressure distribution on individual components, and

— structural response such as displacements and accelerations, particularly if dynamic response occurs.

Model test data shall be converted to full scale by appropriate factors consistent with the physical scaling procedures applied in the test programme.

##### 6.2.3.5.5 Limitations of model testing

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 actions. The primary sources of inherent limitations include the following.

— Surface tension effects: these are not generally allowed for in model test programme definition and can 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 air entrainment: various actions that depend on this phenomenon such as slamming actions will not in general be accurately scaled in typical Froude-scale-based model tests.

The influence of particular effects on actions determined in model tests shall be assessed and steps shall be taken in the testing programme to reduce or minimize them. Such effects can be

- wave reflections from the ends of model test basins,
- scattering of waves from large-volume structures,
- reflection of spurious scattered waves from model basin sidewalls interfering with target design wave conditions,
- breakdown of wave trains representing the target design wave due to various instabilities leading to an inaccurate realization of design wave conditions, and
- difficulties in the inclusion of wind or currents in association with wave fields.

#### 6.2.4 Current action

##### 6.2.4.1 General

Currents, including directionality over the water column, shall be combined with the design wave conditions.

The disturbance of the incident current field due to the presence of the fixed structure shall be accounted for.

##### 6.2.4.2 Methods

Current actions 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 actions.

Analytical procedures based on computational fluid dynamics (CFD) may be used in the evaluation of current actions or other effects associated with current. These procedures are based on an exact solution of the equations of motion of viscous fluids (the Navier Stokes equations). Only well-validated implementations of the CFD procedure shall be used in the computation of current effects. The method can provide a more economic and reliable procedure for predicting drag forces than physical modelling techniques.

##### 6.2.4.3 Local effects

Disturbances of the incident current field lead to modifications in the local current velocity in the vicinity of the structure. Actions on local attachments to the structure shall be computed based on the modified current field. The possibility of vortex-induced vibrations on various attachments shall be investigated.

##### 6.2.4.4 Scour around the base

The presence of water motions in the vicinity of the base of a structure can lead to scour or sediment transport around the base. The potential for sediment 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 or sediment transport.

NOTE There is a substantial body of mostly empirical data (including data related to coastal and port engineering fields) that can be consulted for additional insight into sediment transport processes and the prevention of scour.

### 6.2.5 Wind action

Wind actions on a fixed concrete offshore structure consist of two parts:

— wind actions on the topsides;

— wind actions on the concrete offshore structure above sea level.

Global wind actions shall be determined based on the appropriate design wind velocity in combination with recognized calculation procedures. In a typical case, global wind action may be estimated by simplified procedures such as a block method. In this type of procedure wind actions may be based on calculations that include empirical coefficients for simple shapes for which data are available, an appropriate exposed area and a pressure that is a function of the square of the wind speed normal to the exposed area.

The wind action on the exposed part of the concrete offshore structure is normally small compared to the wind action on the topsides and to wave actions. A simplified method of applying the effect of wind to the concrete structure is using the wind actions on the topsides only.

NOTE For a more complete description of wind actions, see ISO 19901-1 and ISO 19902.

Global dynamic effects of wind action shall be investigated if relevant. By way of example, a structure that is afloat and in a temporary condition during the construction, transportation or installation phases can be susceptible to wind dynamics. An appropriate description of the wind field, such as a wind spectrum, shall be included to determine global dynamic effects of wind action.

### 6.2.6 Seismic actions

Procedures for the determination of seismic actions are provided in ISO 19901-2 at two levels, ELE and ALE respectively. For the ELE event the structure shall meet the normal ultimate limit state requirements. Seismic actions at ALE level may be considered as an abnormal event in the design. Unless otherwise specified in the project specification, the return periods should be as specified in 6.5, the annual probability of exceedance as specified in Table 2, and the action factors as specified in Table 1 of this International Standard.

### 6.2.7 Ice actions

The computation of ice actions is highly specialized and location-dependent and is not covered by this International Standard; see ISO 19906 [7], 4) for pertinent information.

## 6.3 Other actions

### 6.3.1 Permanent actions

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

— self-weight of the structure, including topsides,

— weight of permanent ballast,

— weight of permanently installed parts of mechanical outfitting, including risers, etc.,

— external hydrostatic pressure up to the mean water level, and

— prestressing.

NOTE Prestressing can alternatively be considered as actions from imposed deformations.

4) Under preparation. Until published, see the specialist literature.

### 6.3.2 Variable actions

Variable actions ($Q$) originate from normal operations of the structure in the different phases and vary in magnitude, position and direction during the time period considered. They include actions from

— personnel,

— modules, parts of mechanical outfitting and structural parts planned to be added or removed during the operational 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,

— actions caused by installation and drilling operations, etc., and

— ordinary boat impact, fendering and mooring.

NOTE Variable actions from temperatures may also be considered as actions from imposed deformations.

The assumptions that are made concerning variable actions shall be reflected in the summary report, see 5.3.4, and shall be complied with in the operations. Possible deviations shall be evaluated and, if appropriate, shall be considered in the assessment of accidental actions.

### 6.3.3 Actions from imposed deformations

Certain actions, which can be classified as either permanent or variable, may be treated as resulting from imposed deformations ($D$). Action effects caused by imposed deformations shall be treated in the same way as action effects from normal actions or by demonstration of strain compatibility and equilibrium between applied actions, deformations, and internal forces.

Potential imposed deformations are derived from sources that include

— thermal effects,

— prestressing effects (including effects of prestressing sequences etc.),

— creep and shrinkage effects,

— differential settlement of foundation components, and

— locked-in deformations due to construction stages.

### 6.3.4 Accidental actions

#### 6.3.4.1 General

Accidental actions ($A$) can occur from abnormal environmental events, malfunction, maloperation or accident. The accidental actions to be considered in the design shall be based on an evaluation of the operational conditions for the structure, due account being taken of factors such as personnel qualifications, operational procedures, facilities and equipment, safety systems and control procedures. Concerning evaluations of risk related to fire and explosions, see ISO 13702[6].

Primary sources of accidental actions include

- rare occurrences of abnormal environmental events,
- fires,
- explosions,
- flooding,
- dropped objects,
- collisions, and
- unintended changes in pressure differences.

#### 6.3.4.2 Rare occurrences of abnormal environmental events

These include abnormal environmental events such as the 10 000 year return period wave condition and the ALE seismic event, as well as other abnormal environmental events when relevant.

#### 6.3.4.3 Fires

The principal fire and explosion events are associated with hydrocarbon leakage from flanges, valves, equipment seals, nozzles, etc.

The following types of fire scenarios shall, among others, be considered, where relevant:

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

The fire action intensity may be described in terms of thermal flux as a function of time and space or, simply, as 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.

#### 6.3.4.4 Explosions

The following types of explosions shall be considered:

- ignited gas clouds;
- explosions in enclosed spaces, including machinery spaces and other equipment rooms, as well as FCS shafts and storage tanks.

The overpressure action 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 action 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 action effects. Simple, conservative single degree of freedom models may be applied. If necessary, non-linear time domain analyses based on numerical methods such as 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.

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

#### 6.3.4.5 Flooding

Flooding of compartments in temporary and operational phases shall be considered in the design. In phases where the structure is afloat, the effect on tilt and waterline shall be considered. If mechanical systems are used to minimize the structural effects, these systems shall be designed to operate under the relevant conditions.

#### 6.3.4.6 Dropped objects

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

- cargo dropped from lifting gear;
- falling lifting gear;
- unintentionally swinging objects;
- loss of drilling equipment, pipes, etc.

The impact energy from the lifting gear shall 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 actions from falling objects may be based on the safe working action for the lifting equipment. The action shall be assumed to be due to objects falling from lifting gear from the 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 trajectories and velocities 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.

The impact effect of long objects such as pipes and drill stem equipment shall be subject to special consideration.

#### 6.3.4.7 Collisions

The effect of a vessel impact shall be evaluated if the probability of collision is not negligible. In such an evaluation the nature of all vessel operations in the platform vicinity shall be taken into account. These can include

- vessels in service to and from the installation, including supply vessels,
- tankers loading at the field,

— floating installations, such as flotels, and

— ships and fishing vessels passing the installation;

Where appropriate, impacts from sea ice or icebergs and from aircraft servicing the field shall be treated in the same manner as impacts from vessels.

The most probable impact locations and impact geometry shall be established, based on the dimensions and geometry of the structure and vessel. This shall account for tidal changes, operational sea states, and motions of the vessel and structure in free vibration modes. Potential vessel impact on the structure’s waterline members, risers and external wells shall be considered. Effective operational restrictions on vessel approach sectors can limit the exposure to impacts in some areas of the structure. Unless more detailed investigations are done for the relevant vessel and structure, the impact zone for supply vessels may be considered to be between 10 m below LAT and 13 m above HAT. Barge bumpers, boat landings and other external fendering may be used as protection.

Depending on the risk of collision and the consequences for the structural integrity of the structure, an analysis of vessel impact conditions can be required. Irrespective of whether an analysis is required, robustness in relation to vessel collisions should be incorporated into the design by indirect means such as

— avoiding weak elements in the structure,

— selecting materials with sufficient toughness, and

— ensuring that critical components are not placed in vulnerable locations.

Impact actions are characterized by kinetic energy, impact geometry and the relationship between action and indentation. In a rigorous impact analysis, if required, accidental design situations shall be established representing bow, stern and beam-on impacts on all exposed components. The collision events shall represent both a fairly frequent condition, during which the structure should only suffer insignificant damage, and a rare event where the emphasis is on avoiding a complete loss of integrity of the structure.

Two energy levels shall be considered:

a) low energy level, representing the frequent condition, based on the type of vessel which would routinely approach alongside the platform (e.g. a supply boat) with a velocity representing normal manoeuvring of the vessel approaching, leaving or standing alongside the platform.

b) high energy level, representing a rare condition, based on the type of vessel that would operate in the platform vicinity, drifting out of control in the worst sea state in which it is allowed to operate close to the platform.

Level a) represents a serviceability limit state to which the owner can set his own requirements based on practical and economical considerations. Level b) represents an ultimate limit state in which the structure is damaged but progressive collapse shall not occur. In both cases the analysis shall account for the vessel’s mass, its added mass, orientation and velocity. The possibility of leaks due to damages in the impact zone and flooding shall be assessed.

The collision energy can be determined on the basis of relevant masses, velocities and directions of vessels that can collide with the structure. All traffic in the relevant area shall be mapped and possible future changes in vessel operational pattern shall be accounted for. Design values for collisions are determined based on an overall evaluation of possible events.

The mass of supply ships selected should normally be not less than 5 000 tonnes. A hydrodynamic (added) mass of 40 % for sideways and 10 % for bow and stern impact can be assumed. For low energy impacts, a vessel velocity of 0,5 m/s is commonly used, representing a minor “bump” during normal manoeuvring of the vessel while loading or unloading or while standing alongside the platform. For high energy conditions, a vessel velocity of 2 m/s is commonly used, representing a vessel drifting out of control in a sea state with significant wave height of approximately 4 m.

#### 6.3.4.8 Unintended changes in pressure difference

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 shall also be considered.

#### 6.3.4.9 Floating structures in damaged condition

Floating structures that experience buoyancy loss or flooding have an abnormal floating position. The corresponding abnormal variable and environmental actions shall be considered.

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

#### 6.3.4.10 Combination of accidental actions

Where accidental actions occur simultaneously, the annual probability level ($10^{-4}$) applies to the combination of these actions. Unless the accidental actions are caused by the same phenomenon (like hydrocarbon gas fires and explosions), the occurrence of different accidental actions may be assumed to be statistically independent.

NOTE While, in principle, the combination of two different accidental actions with exceedance probability of $10^{-2}$, or one at $10^{-3}$ and the other at a $10^{-1}$ level, corresponds to a $10^{-4}$ event, individual accidental actions at a probability level of $10^{-4}$ will normally be most critical.

### 6.4 Partial factors for actions

Partial factors for actions to be applied with representative actions according to Table 2 are given in Table 1. The factors should be adjusted as required for consistency with the reference standard used to provide an equivalent level of safety.

NOTE The recommended factors are consistent with the use of NS 3473.E [2] as the reference standard.

The ultimate limit state shall be checked for two sets of combinations of actions, ULS (A) and ULS (B) (see Table 1).

**Table 1 — Partial factors $\gamma_F$ for actions for different limit states**

| Limit state | Classification of action | | | | |
|---|---|---|---|---|---|
| | $\gamma_G$ | $\gamma_Q$ | $\gamma_E$ | $\gamma_D$ | $\gamma_A$ [a] |
| ULS (A) | 1,3 | 1,3 | 0,7 [b] | 1,0 | 0 |
| ULS (B) | 1,0 | 1,0 | 1,3 [b] | 1,0 | 0 |
| SLS | 1,0 | 1,0 | 1,0 | 1,0 | 0 |
| FLS | 1,0 | 1,0 | 1,0 | 1,0 | 0 |
| ALS | 1,0 | 1,0 | 1,0 | 1,0 | 1,0 |

[a] A value of 0 for a partial factor for actions means that the action is not applicable to the design situation.

[b] These values may have to be adjusted for areas with long-term distribution functions that differ from those for North Sea conditions.

The actions shall be combined in the most unfavourable way, provided that the combination is physically possible and permitted according to the action specifications. Combinations of actions that are physically possible but not intended or permitted to occur in operations shall be included by assessing their probability of occurrence and shall be accounted for either as an accidental design situation in the accidental limit state (ALS) or shall be treated as part of the ordinary design situations included in the ULS. Such conditions may be omitted in cases where the annual probability of occurrence can be determined to be less than $10^{-4}$.

For permanent actions, a partial factor for action of 1,0 in action combination ULS (A) shall be used where this gives a more unfavourable action effect.

For external hydrostatic pressure, and for internal pressures resulting from a free surface, an action factor of 1,2 may normally be used, provided that the action effect can be determined with normal accuracy. Where second order effects are important, a partial factor for action of 1,3 shall be used. Where an action is the result of the difference between independent and counteracting hydrostatic pressures, the pressure difference shall be multiplied by the partial factor for action. 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.

Prestressing actions may be considered as actions resulting from imposed deformations. Due account shall be taken of the time-dependent effects in calculation of effective internal forces. The more conservative value of 0,9 or 1,1 shall be used as a partial factor for action in the design.

A partial factor for action of 1,0 shall be applied to the weight of soil included in the geotechnical calculations.

For calculation of the soil capacity during cyclic actions, the design action effect shall be determined for the following two cases:

a) using a partial factor for action equal to 1,0 for the cyclic actions and 1,3 for the largest environmental action;

b) using a partial factor for action larger than 1,0 on the cyclic actions throughout the load history, including the greatest environmental action. Appropriate values of the partial factor for action shall be determined on the basis of an evaluation of the uncertainties associated with the cyclic action history.

The load history shall represent the wave conditions affecting the pore pressure build-up in a design storm, with respect to the distribution of load cycles, and their magnitude and number, prior to the time considered in the analysis. When the method in case b) is employed, a partial factor for action of 1,15 has been considered appropriate in a number of cases.

## 6.5 Combinations of actions

Table 2 gives a more detailed description of how actions shall be combined. When environmental and accidental actions are acting together, the given probabilities apply to the combination of these actions.

**Table 2 — Representative actions and combinations of actions**

| | Representative actions and combinations of actions | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **Temporary phases** | | | | | **Normal operations** | | | | |
| | Serviceability limit state (SLS) | Fatigue limit state (FLS) | Ultimate limit state (ULS) | Accidental limit state (ALS) | | Serviceability limit state (SLS) | Fatigue limit state (FLS) | Ultimate limit state (ULS) | Accidental limit state ALS | |
| | | | | Abnormal situation | Damaged situation | | | | Abnormal situation | Damaged situation |
| **Permanent actions** | **Representative value** | | | | | | | | | |
| **Variable functional actions** | **Specified value** | | | | | | | | | |
| **Environmental actions** | Dependent on operational requirements | Representative action history | Dependent on measures taken [a] | | | Dependent on operational requirements | Representative action history | Annual probability of exceedance $= 10^{-2}$ | Annual [b] probability of exceedance $= 10^{-4}$ | Annual probability of exceedance $= 10^{-2}$ |
| **Deformation actions** | **Representative extreme value** | | | | | | | | | |
| **Accidental actions** | Not applicable | | Dependent on operational requirements | | Not applicable | | | | Annual probability of exceedance $= 10^{-4}$ | Not applicable |

[a] See 6.5.2 and Table 3.

[b] When an abnormal environmental load is applied in ALS as accidental load, the annual probability of exceedance for an abnormal level earthquake ALE shall be agreed with relevant authorities, typically $10^{-4}$ has been used for fixed concrete offshore structures in low to moderate seismic areas.

For temporary phases, where a progressive collapse in the installation does not entail the risk of loss of human life, injury, or damage to people or the environment, or of significant financial losses, a reduced return period for environmental actions may be considered. The return period to be considered should be related to the duration of the operation. As general guidance, the criteria given in Table 3 may be applied:

**Table 3 — Environmental criteria**

| Duration of operation | Environmental conditions |
|---|---|
| Up to 3 days | Specific weather window |
| 3 days to 1 week | 1 year return period, seasonal |
| 1 week to 1 month | 10 year return period, seasonal |
| 1 month to 1 year | 100 year return period, seasonal |
| More than 1 year | 100 year return period, all year |

## 6.6 Exposure levels

Structures can be categorized by various levels of exposure to determine criteria that are appropriate for the intended service of the structure. The levels are determined by consideration of life-safety and of environmental and economic consequences.

Life-safety considers the manning situation in respect of personnel on the platform when the design environmental event occurs.

Consequence considers the potential risk to life of personnel brought in to react to any incident, the potential risk of environmental damage and the potential risk of economic losses.

Three categories for each of life-safety and consequence can, in principle, be combined into nine different exposure levels. This results in three exposure levels, according to Table 4.

**Table 4 — Determination of exposure level**

| Life-safety category | **Exposure level** (L1 to L3) | | |
|---|---|---|---|
| | **Consequence category** | | |
| | **High consequence** | **Medium consequence** | **Low consequence** |
| Manned–nonevacuated | L1 | L1 | L1 |
| Manned–evacuated | L1 | L2 | L2 |
| Unmanned | L1 | L2 | L3 |

The exposure level applicable to a structure shall be determined by the owner prior to the design of a new structure, and be agreed with the regulator where applicable.

NOTE The exposure levels are intended to have the same meaning as specified in greater detail in ISO 19902.

Structures in exposure level L1 shall be designed in accordance with the requirements of this International Standard for permanent, variable, environmental, deformation and accidental actions. Inspection of execution shall be according to inspection class 3, extended inspection, see 8.6.2.

For structures in exposure level L2 the same requirements apply as for L1 structures except that inspection of execution may be performed according to inspection class 2, normal inspection.

For structures in exposure level L3 the same requirements apply as for L2 structures except that accidental actions with a probability of less than $10^{-3}$ may be disregarded. Additionally, a reduction of the factor for environmental actions of not more than 10 % may be considered, if permitted by the project specification.

# 7 Structural analysis

## 7.1 General

Appropriate structural analyses shall be performed to determine the action effects within a structure. These analyses shall define the responses of the structure during each stage of its life in accordance with Clause 6.

Structural analysis refers to calculations based on numerical techniques including longhand calculations and/or computer-based methods. The development of action effects by model testing or by instrumentation of other similar structures is not precluded by this International Standard.

Complex or unusual structural types can require forms of analysis not described within this International Standard. If required, these shall be performed in accordance with the principle that a sufficient number of suitable analyses are carried out to accurately assess all significant action effects within the structure.

## 7.2 General principles

### 7.2.1 Planning

In order to ascertain that analysis of a fixed concrete offshore structure is successful, the following are required:

- all necessary analyses shall be performed on the basis of an accurate and consistent structural model that defines the structural system to the appropriate level of detail and enables the required corresponding actions on the structure to be assessed;
- these analyses shall be performed using appropriate methods, shall have relevant boundary conditions and shall be of suitable type;
- suitably verified results in the form of action effects shall be available in due time for use in design or reassessment.

Interfaces with structural designers, topsides designers, hydrodynamicists, geotechnical engineers and other relevant parties shall be set up. The schedule of supply of data on actions (including reactive actions) shall be determined and monitored. Interfaces shall ensure that these data are in the correct format, cover all necessary locations and are provided for all required limit states and for all appropriate stages in the lifetime of the structure.

### 7.2.2 Extent of analyses

Sufficient structural analyses shall be performed to provide action effects suitable for checking all parts and all structural components of the primary structure for the required design situations and limit states. At least one analysis should normally represent global behaviour of the structure during each relevant stage of its life.

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

The simulation of a part of the structure or of a structural component shall, as a minimum, comprise a three-dimensional representation of the stiffness of all primary structure. In general, the stiffness of secondary structures may be omitted from structural analyses, although significant action effects due to secondary structure shall be incorporated. Secondary structure can provide such restraint to the primary structure that additional section forces are developed. Such effects shall be assessed and included where necessary.

Secondary components of the structure shall be assessed, by analysis if necessary, to determine their integrity and durability, and to quantify their contribution to action effects in the primary structure. Such analyses may be performed separately from analysis of the primary structure, but shall include deformations of the supporting primary structure, where relevant.

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

Where appropriate, the analysis shall include a representation of the foundation of the structure, simulated by stiffness elements or by reactive actions.

### 7.2.3 Analysis requirements

All structural analyses required for the design of the structure shall be carried out in accordance with the planned analysis schedule using the most recent data on geometry, materials, boundary conditions, actions and other relevant information.

The structure shall be analysed for appropriate actions during each stage of its life. Where simultaneous actions are possible, these actions shall be applied in combination in such a way as to maximize action effects at each location to be checked. The actions that contribute to these combinations shall include appropriate partial factors for action, as specified in Clause 6, for each limit state being checked.

Appropriate analysis types shall be selected to provide accurate action effects covering all requirements of the design or assessment process of the concrete listed in Clause 8; 7.5 contains requirements for typical analyses that shall be performed on a fixed concrete offshore structure and its structural components, including the selection of appropriate analysis types and execution requirements for each design situation. Execution of an analysis for a particular stage in the life of the structure shall be performed in accordance with the specific requirements of the relevant clause.

### 7.2.4 Calculation methods

Various calculation methods may be used for the determination of action effects in response to a given set of actions. These include, but are not limited to, hand calculations and computer methods, such as spreadsheets or finite element analyses.

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

Analysis of the global structure or local components is normally performed by the finite element method. Computer software used to perform a finite element analysis shall comply with a recognized international quality standard, such as that given in ISO 90003 [1], or shall be validated for its intended use prior to the start of the analysis. Element types, action applications, meshing limits and analysis types to be used in the structural analysis shall all be included in the validation.

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. Validation and “benchmark” testing of the software shall be used to identify element limitations and the computer modelling shall be arranged to provide reliable results.

Hand calculations are generally limited to simple structural members (beams, regular panels, secondary structures, etc.) under simplified actions (i.e. uniform pressure, point or distributed actions). 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.

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

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

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

### 7.2.5 Verification of analysis results

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. This verification is in addition to the validation of computer software described in 7.2.4.

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

— that the structural model adequately represents the geometry of the intended structure or structural part;

— that the specified material properties have been used;

— that sufficient and correct actions 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.

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

— individual and summed reactions, to ensure that these balance the applied actions;

— deformations of the structure, to verify that these are sensible and that they demonstrate compatibility between structural components;

— natural periods and mode shapes, if appropriate, to verify that these are sensible;

— load paths, bending moment diagrams, stress levels, etc., to check that these satisfy equilibrium requirements.

### 7.2.6 Documentation

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

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

— the purpose and scope of the analysis and the limits of its applicability;

— references to methods used and the justification of any assumptions made;

- the assumed geometry, showing and justifying any deviations from the current structural geometry;
- material properties used in the analysis;
- boundary conditions applied to the structure or structural component;
- the summed magnitude and direction of all actions;
- pertinent results from the analysis and cross-checks to verify the accuracy of the simulation, in accordance with 7.2.5;
- a clear presentation of the results of the analysis that are required for further analysis, structural design or assessment.

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

- displacements and vibrations, which shall be within acceptable limits for operation of the platform;
- section forces, from which the capacity of concrete sections and necessary reinforcement requirements can be determined;
- section strains, used to determine crack widths and watertightness.

## 7.3 Physical representation

### 7.3.1 Geometrical definition

Dimensions used in structural analysis calculations shall represent the structure as accurately as necessary to produce reliable values of action effects. Changes in 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 as described in 7.2.1, the changes shall be incorporated by reanalysis.

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 partial factors for material are used. Geometrical and material tolerances considered in this International Standard are defined in Clause 8. These are consistent with the partial factors for material defined in 5.6.5.

Where as-built dimensions differ from nominal sizes by more than the permissible tolerances, the effect of this dimensional mismatch shall be incorporated in the analysis. The effect of tolerances shall also be incorporated in the analysis where action effects and hence the structural design are particularly susceptible to their magnitude (imperfection bending in walls, implosion of shafts, etc.).

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 partial factors being applied to component material properties.

The effects of wear and corrosion shall be accounted for in the analysis where relevant and where adequate measures are not provided to limit such effects.

It will normally be sufficient to consider centre-line distances as the support spacing for beams, panels, etc. In certain circumstances, however, face-to-face distances can 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.

### 7.3.2 Material properties

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.

For most limit states it is normally acceptable to simulate the concrete by equivalent linear elastic properties. Unless a different value can be justified, the modulus of elasticity (E-modulus) of plain concrete may be used as the modulus of reinforced concrete in such an analysis. The value used shall be in accordance with the reference standard to concrete design in use. For actions that result in very high strain rates, the increase in concrete modulus of elasticity should be considered in the analysis of the corresponding action effects. The applicability of linear elastic analysis is discussed in 7.4.1.

Age effects of the concrete may be included, if permitted by the design rules in use. Effects of the duration of actions and resultant creep of the concrete shall also be considered, where relevant. Where actions can occur over a significant period in the life of the structure, the least favourable instance shall be considered in determining age effects.

Accurate evaluation of concrete stiffness can be particularly important for natural frequencies and for dynamic analysis, and for simulations that incorporate steel components, such as the topsides or conductor framing. Consideration shall be given to extreme values of concrete stiffness in such analyses.

Non-linear analysis techniques are often applied to local structural components. It is then 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.

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 where relevant.

Unless another value is shown to be more appropriate, a Poisson's ratio of $\nu = 0{,}2$ shall be assumed for uncracked concrete. For cracked concrete, a value of $\nu = 0$ may be used. A coefficient of thermal expansion of $1{,}0 \times 10^{-5}$ per degree centigrade shall 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 for the materials in use.

### 7.3.3 Soil-structure interaction

The representation of the foundation of a fixed concrete structure will differ with the type of analysis being undertaken. For static analysis, reactive soil pressures on soil contact surfaces are normally sufficient, but for dynamic analysis or where soil-structure interaction is significant, an elastic or inelastic representation of the foundation will normally be required to provide suitable stiffness. Seismic analysis is typically very dependent on soil properties. Further details of foundation modelling requirements for specific analysis types are given in 7.5.

Reactive actions on the structure from its foundation, or effective foundation stiffness, shall be based on general principles of soil mechanics in accordance with Clause 9. Sufficient reactive actions 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 foundation pressures across the base of the structure.

The calculations used shall reflect the uncertainties inherent in foundation engineering. Upper and lower bounds and varied patterns of foundation reactions shall be incorporated and an appropriate range of reactive actions shall be assessed. In particular, the sensitivity of structural response to different assumptions on the distribution of reactive actions between the base and any skirts shall be determined.

Consideration shall also be given to the unevenness of the sea floor or soil layers in the seabed, which can potentially cause high local reactive actions. Foundation unevenness may be considered as actions resulting from imposed deformations in subsequent design checks, in accordance with Clause 6. Other than this, foundation pressures shall be considered as reactive actions, their magnitude being sufficient to counteract all other factored actions.

The analyses shall include intermediate conditions during installation, such as initial contact and skirt penetration as well as the fully grouted permanent condition, where relevant. Disturbance of the seabed due to the installation procedure should be considered in calculating subsequent foundation pressures.

Where it significantly affects the design of certain components, soil interaction with conductors shall be incorporated in the analysis, particularly with regard to local analysis of conductor support structure.

Upper limits of soil resistance should be considered during analysis of the removal of the structure.

### 7.3.4 Other support conditions

Other than direct support from foundation soils, a structural component can be supported by

— external water pressure, while floating,

— other components of the structure,

— an air cushion, and

— any combination of the above, together with foundation soils.

The action of water pressure to support a structure while floating shall be evaluated by suitable hydrostatic or hydrodynamic analysis and shall be applied to appropriate external surfaces of the structure. Water pressure considered in this way is a reactive action. In order to maintain the water line in the correct position the action and the reaction shall both be scaled with the same action factor(s) to maintain equilibrium.

NOTE To scale the reaction by the buoyancy forces to correspond to the applied action including action factors in order to maintain equilibrium can be seen as scaling the unit weight of water in order to respect the waterline and load the actually wetted areas. This is not physical but an accepted way of simulating floating conditions while accounting for the uncertainties to be covered by the partial factors.

Representative boundary conditions shall be applied to the analysis of a structural part extracted from the global structure. These boundary conditions shall include possible settlement or movement of supports, based on a previous analysis of the surrounding structure.

In the absence of such data, suitable idealized restraints shall be applied to the boundary of the structural part to represent the behaviour of the surrounding structure. Where there is uncertainty about the effective stiffness at the boundaries of the component, a range of possible values shall be considered.

Internal force, stiffness or displacement may be applied as boundary conditions to support a structural member. 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 action effects chosen for design.

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.

### 7.3.5 Actions

Actions shall be determined by recognized methods, taking into account their variation in time and space, in accordance with Clause 6. Such actions shall be included in the structural analysis in a realistic manner, representing the magnitude, direction and time variance of such actions.

Permanent and variable actions 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 anticipated values. The former govern some aspects of the design of gravity-based structures.

Hydrostatic pressures shall be based on the specified range of fluid surface elevations and densities. Hydrostatic pressures on floating structures during the transportation, installation and removal stages shall include the effects of trim and heel as well as pitch and roll of the structure, due to influences such as intentional trim, wind heel, wave action or differences in stable attitude of the structure when damaged.

Prestressing effects shall be applied to the model as external actions at anchorages and bends, or as compatible internal strain effects. In both cases, due allowance shall be made for all likely losses in prestressing force. Where approximated by external actions, relaxation in tendon forces due to the effect of other actions on the state of strain in the concrete shall be considered.

Initial prestressing forces at lock-off will normally have only local effect but should be considered in analysis if relevant.

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 structural components. The temperatures shall be determined with due regard to thermal boundary conditions and material conductivity. Thermal insulation effects, e.g. due to insulating concrete or drill cuttings, shall be considered if present.

Wave, current and wind actions shall include the influence of such actions on the motion of the structure while floating. In cases where dynamic response of the structure can be of importance, such response shall be considered in determining action effects. Quasi-static or dynamic analyses shall be used, in accordance with 7.4.

Uncertainties in the centre of gravity of the topsides and in built-in forces and deformations from the transfer of the topsides weight 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.

### 7.3.6 Mass simulation

A suitable representation of the mass of the structure is required for the purposes of motion prediction, for actions due to accelerations of masses while floating and for dynamic analyses. The mass simulation shall include relevant quantities from at least the following:

- the mass of 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, flooding, etc.;
- the mass of solid ballast within the structure;
- the mass of snow and ice accumulation on the structure, where relevant;
- the mass of drill cuttings or other deposits on the structure;
- the mass of marine growth on the structure;
- added water mass;
- added soil mass.

The magnitudes of masses within the structure shall be distributed as accurately as necessary to determine all significant modes of vibration (including torsional modes) and all actions due to accelerations of masses for the structural analysis being performed. Particular attention shall be paid to the height of topsides equipment or modules above the structural steelwork.

For quasi-static analyses it is normally only necessary to consider the maximum mass associated with a given design situation for the structure. However, for dynamic analyses this does not necessarily produce the worst response, in particular with respect to torsional modes, and a range of values of mass and centre of gravity shall be considered. For a fatigue analysis, the variation in the history of actions 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.

Calculation of the added mass of water and 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 for small submerged members, reducing to 40 % of the mass of displaced water for larger structural members. Added mass effects may be ignored along the axial length of prismatic members, such as the shafts.

### 7.3.7 Damping

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 deformation levels of structural members and soil.

NOTE Typical values for damping will be in the range from 1 % to 3 % of critical damping; for seismic analysis, see 7.5.7.3.

## 7.4 Types of analyses

### 7.4.1 Static linear elastic analysis

It is generally acceptable for the behaviour of a structure or structural part to be based on static linear elastic analysis, unless there is a likelihood of significant dynamic or non-linear response to a given type of actions. In such cases, dynamic or non-linear analysis approaches shall be performed, as defined in 7.4.2 and 7.4.3.

Static analysis is always permissible if all actions on the component being considered are substantially invariant with time. If actions are periodic, transient or impulsive in nature, the magnitude of dynamic response shall be evaluated in accordance with 7.4.2 and static analysis shall only be permitted if dynamic effects are small.

Reinforced concrete is typically non-linear in its behaviour, but it is generally acceptable to determine global load paths and section forces for ultimate, serviceability and fatigue limit states based on an appropriate linear elastic analysis, subject to the restrictions presented below. Non-linear analysis can be required for accidental limit states, abnormal level earthquakes and local analysis.

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 localized yielding or cracking. In particular, response to actions caused by deformations 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.

Reduction of the stiffness of components should be considered if it can be shown that, for example, due to excessive cracking, more accurate load paths can be determined by such modelling. Such reduced stiffness shall be supported by appropriate calculations or by non-linear analysis.

A linear analysis preserves equilibrium between externally applied actions and internal 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 level of actions. A solution is hence always obtained, irrespective of the action levels. Linear analysis can be carried out for many independent load cases, after which the independent load cases may be superimposed into combined cases or load combinations representing complete design situations.

NOTE Experience has shown that the use of a structural analysis representing all actions as unit load cases that afterwards can be scaled in magnitude and added to represent complete load combinations, i.e. loading scenarios, is very effective. It is, however, important to ensure equilibrium between actions and reactions in such a way that there are no unbalanced reactions at the model support boundaries, for example, by creating equilibrium groups that are afterwards scaled and combined to represent the loading scenarios.

### 7.4.2 Dynamic analysis

#### 7.4.2.1 General

Fixed structures with global natural periods greater than 2,5 s can be susceptible to dynamic response due to wave action during in-service conditions, at least for fatigue assessment. Structures in shallow water or subject to extreme wave conditions can exhibit significant dynamic response at lower natural periods due to the higher frequency content in shallow water or unusually steep waves.

Other situations to which the structure can be subjected, such as waves during sea tow, wind turbulence, vibration, impact or explosion, can also impose dynamic actions of significant magnitude close to fundamental natural periods of the structure or its components. Structures that respond to a given set of actions by significant motion or vibration at one or more natural periods shall be assessed by dynamic analysis techniques.

Earthquakes are a particularly severe form of oscillatory excitation that shall always require detailed dynamic analysis if the zone of seismic activity produces significant ground motions.

Where dynamic effects can be significant, global dynamic response can often be evaluated using a simplified model representation of the structure; in some cases dynamic effects can be assessed by the calculation of natural periods and the evaluation of dynamic amplification factors. In evaluating dynamic amplification factors for wave action, consideration shall be given to higher frequency components of wave and wind action that occur due to drag loading, sharp crested shallow water waves, ringing, etc.

Where substantial dynamic response of the structure is predicted, having magnitudes 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 mode of motion or vibration of the structure is significantly excited by the applied actions, as is the case for seismic response. Dynamic analysis requirements are presented in 7.4.2.2.

Where dynamic effects are relatively insignificant, a quasi-static analysis of the structure or its components may be performed, and dynamic effects may be included in accordance with 7.4.2.3.

#### 7.4.2.2 Dynamic analysis requirements

Where dynamic response is likely to be relatively important, a 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.

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 excitation on the foundation. Specific requirements for seismic analysis are presented in 7.5.7.

Actions applied to the structure or its components shall include all frequency content likely to cause dynamic response. The relative phasing between different actions shall be rigorously applied.

Harmonic or spectral analysis methods are suitable for most forms of periodic or random cyclic excitation. Where significant dynamic response is associated with non-linear excitation or non-linear behaviour of the structure, of a structural component or of the structure's foundation, then transient dynamic analysis shall be required.

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.

For impulsive actions, such as those due to ship impacts, slamming or explosions, dynamic amplification effects may be quantified by the response of single- or multi-degree of freedom systems representing the stiffness and mass of the structural components being analysed. Transient dynamic analysis will normally be required.

#### 7.4.2.3 Quasi-static analysis

Quasi-static analysis refers to any analysis in which the influence of structural accelerations is small enough that dynamic actions can be represented approximately by a factor on static loads or by equivalent quasi-static actions. Both approaches are only appropriate if static and dynamic action effects give an essentially similar response pattern within the structure, but differ in magnitude.

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 columns or shafts of a structure, appropriate local values of bending moment should be used. Base shear, overturning moment and soil pressure are representative responses for the structure’s base.

For the latter approach, additional actions shall be applied to the structure to represent the incremental effects due to acceleration of masses. All actions applied in a quasi-static analysis may be considered constant over time except in the case of non-linear response, where knowledge of the excitation history can be important, and the excitation should be applied to the simulation in appropriate steps.

Factored dynamic results shall be combined with factored static effects due to permanent and variable actions, etc. in accordance with the limit states being checked. Partial factors for action for dynamic actions should be consistent with the excitation that causes the dynamic response, normally environmental. The most detrimental magnitude and direction of dynamic excitation shall be considered in design combinations.

### 7.4.3 Non-linear analysis

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 the actions effects (imposed deformations, dynamic response, etc.);

— where the structural component depends on significant non-linear material behaviour to resist a given set of actions, such as in response to accidents or abnormal level seismic events;

— where slender members are in compression and deflections can cause significant action effects (imperfection bending or buckling).

A non-linear analysis is able to simulate effects of geometrical and material non-linearities in the structure or in a structural component. These effects increase with an increase in actions and require application of the actions in steps with a solution of the equations at each time step; at each level of actions iterations to determine equilibrium condition shall be carried out.

Non-linear solutions cannot be superimposed. This implies that a non-linear analysis shall be carried out for every design situation for which a solution is required.

Non-linear analysis of the global structure or of structural components may be based on a relatively simple structural model. Where linear elastic elements are included in the model, it shall be demonstrated that these remain linear throughout the applied actions. Appropriate stress-strain or load-deflection characteristics shall be assigned to other parts that behave non-linearly. Deflection effects shall be incorporated where relevant.

Non-linear analysis of components to determine their ultimate strength may be performed, provided that the model can appropriately cover all failure modes (e.g. bending, axial force, shear, compression failure affected by reduced effective concrete strength, etc.) and that the concrete tensile stresses are covered by reinforcement. If one analysis is not sufficient to verify all failure mechanisms, separate additional verifications should be carried out. Material properties used in non-linear analysis should be reduced by appropriate partial factors for material, in accordance with 5.6.5. Where components of the structure rely upon non-linear or ductile behaviour to resist extreme actions, such components shall be detailed to permit such behaviour, in accordance with 8.2.

Complex non-linear analysis of discontinuity regions (see 8.2.11) using finite element methods should not be used without prior calibration of the method against relevant experimental results.

#### 7.4.4 Probabilistic analysis

It is generally acceptable to base in-service structural analysis of a concrete offshore structure subjected to wave action on the principles of deterministic analysis, predicting responses to specific events. However, where stochastic or probabilistic methods are more appropriate for a particular limit state these shall be substituted as needed.

Probabilistic methods typically require linearization of action effects. This can restrict their use where non-linear response of the structure or structural component is significant. If non-deterministic analysis methods are used, time domain response to transient excitation can be necessary.

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

#### 7.4.5 Reliability analysis

Reliability assessment of structures is permitted under the provisions of this document to assess the risk of failure of a structure and to ensure that this is below acceptable levels. However, such an analysis is beyond the scope of this International Standard and shall be performed in accordance with industry practice according to the state-of-the-art at the time of performing the analysis and by agreement between all parties involved.

### 7.5 Analyses requirements

#### 7.5.1 General

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

Table 5 gives general guidance as to the type of analysis that should be adopted for each design situation for the structure. Further details are provided in 7.5.2 to 7.5.9.

#### 7.5.2 Analysis of construction stages

Sufficient analyses shall be performed for construction stages to ensure the integrity of parts of the structure 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.

Consideration shall be given to the sequence of construction in determining action effects and to the age of the concrete in determining resistance. Specific consideration shall be given to the stability of components during construction. Adequate support for temporary actions, such as crane footings, shall be provided in the analysis.

**Table 5 — Guidance on appropriate types of analysis for different design situations**

| Design situation | Appropriate type of analysis |
|---|---|
| Construction | Linear static analysis is generally appropriate. |
| Transportation | Linear static analysis is generally appropriate. Dynamic wave effects can normally be simulated in a quasi-static analysis. |
| Installation and deck mating | Linear static analysis is generally appropriate. |
| In-service strength and serviceability | Linear static or quasi-static analysis is generally appropriate for global load path analysis. |
| Fatigue | Linear analysis is generally appropriate. Dynamic effects can be significant for relatively short period waves when calculating structural response history due to wave action for assessment of cumulative damage. |
| Seismic | Dynamic analysis is normally required, where seismic ground motion is significant. It can be necessary to consider non-linear effects for abnormal level earthquakes. |
| Accidental | Non-linear analysis is normally required for significant impacts. Dynamic response can be significant. |
| Removal/reuse | As per transportation and installation. |

### 7.5.3 Transportation analysis

Analysis of a fixed concrete offshore 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 modelling of the structure during such operations shall be consistent with the stage being represented, incorporating the correct amount of ballast and modelling only those components of the topsides that are actually installed.

Analysis during sea tow should normally be based on linear static analysis, representing the motion of the concrete offshore structure by an inclination of the permanent actions in accordance with the maximum angles of pitch and roll, and by mass inertial actions associated with peak heave, sway, surge, pitch and roll accelerations, both as predicted by motion analysis. For such analysis to be valid, it shall be demonstrated that motions in the natural periods of major components of the structure, such as the shafts, will not be significantly excited by global motions of the structure. If dynamic effects of major parts are deemed important, they shall be incorporated in accordance with 7.4.2.

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.

### 7.5.4 Installation and deck mating analysis

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

Deck mating, ballasting down and setting down on the sea floor shall normally be analysed by a linear static analysis. As these phases normally represent the largest external hydrostatic pressures, implosion or buckling should be analysed. The effect of unevenness of the seabed soil layers shall be considered in assessing seabed reactions in an ungrouted situation.

### 7.5.5 In-service strength and serviceability analyses

At least one global analysis of the structure shall be performed in its in-service configuration suitable for subsequent strength and serviceability assessment. The structural model shall allow simulations of built-in stresses or deformations from the different stages of construction, if relevant.

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

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

In-service analysis may normally be performed using linear static methods, the behaviour of the reinforced concrete being represented in accordance with 7.3.2. However, whenever significant dynamic effects are expected, quasi-static or full dynamic analysis shall be performed, in accordance with 7.4.2.

### 7.5.6 Fatigue analysis

Fatigue analysis shall be based on a cumulative damage assessment performed over the design service life of the structure, and shall consider the effects of the range of sea states and directions to which the structure will be subjected, in accordance with Clause 5 of this document. Where relevant, fatigue damage accrued during construction and/or transportation from the construction site to the permanent location shall be included in the accumulation of fatigue damage.

A linear model of the overall structure is generally acceptable for the evaluation of global action effects. Dynamic effects are 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 quasi-static or by dynamic response analysis, in accordance with 7.4.2. Deterministic or stochastic types of analysis are both permissible, subject to the following requirements.

Where deterministic analyses are deemed adequate, the selected individual waves to which the structure is subjected shall be based on a representative spread of wave heights, periods and directions. 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.

As noted in 7.4.4, for probabilistic analysis sufficient wave cases shall be analysed to adequately represent the stress-transfer functions of the structure. Where relevant, non-linear response of the structure shall be incorporated into the analysis using appropriate methods.

### 7.5.7 Seismic analysis

#### 7.5.7.1 General

Two levels of seismic excitation on a structure shall be considered, in accordance with Clause 6:

— extreme level earthquake (ELE), which shall be assessed as a ULS condition;

— abnormal level earthquake (ALE), for which ductile behaviour of the structure assuming extensive plasticity is permissible provided the structure survives.

Where ductile response of specific components of the structure under the ALE event is predicted or considered in the analysis, such components shall be designed for ductile behaviour, in accordance with Clause 8. Expected best estimates of stress/strain parameters associated with ductile behaviour may be adopted in the analyses. Due consideration shall be given to the effects of greater than representative strength with respect to the transfer of forces into adjoining members, and for the design of those failure modes that are not ductile, such as shear failure. For those cases where the structure can be designed to the ALE event applying normal elastic analysis and ALS criteria, no special detailing for ductility is required.

NOTE 1 Experience with fixed concrete offshore structures installed at locations with low to moderate seismic activity has shown that even for the ALE event analysed based on an elastic structural model, the action effects from seismic excitation are less than those for other actions. In such cases a more refined seismic response analysis considering plastic behaviour will not be necessary.

Seismic events may be represented by input response spectra or by time histories of significant ground motion, in accordance with ISO 19901-2. If the global response of the structure is essentially linear, a dynamic spectral analysis shall normally be appropriate. If non-linear response of the structure is significant, transient dynamic analysis shall be performed.

NOTE 2 In ISO 19901-2 a seismic reserve capacity factor $C_r$ is employed. There is little or no experience in establishing realistic $C_r$ values for fixed offshore concrete structures. An appropriate value will therefore have to be established based on the actual structure. Normally, for concrete structures with continuity and good ductility, a factor of more than 1,4 has been assumed acceptable.

Seismic response of a structure is highly dependent on the natural periods of the structure over a range of modes. This relies on accurate assessments of the structure's mass and stiffness, and a best estimate of soil stiffness. Such parameters shall be carefully assessed and, if necessary, the sensitivity of the response of the structure to changes in these parameters shall be evaluated.

#### 7.5.7.2 Structure and foundation simulation

Interaction of the structure with its foundation is particularly significant for seismic analysis. The foundation shall be modelled with sufficient accuracy for global structural analysis to ensure an accurate assessment of natural periods of vibration and a suitable distribution of soil actions into the structure.

Two principal types of seismic analyses can be performed for fixed concrete offshore structures:

— direct soil-structure analysis;

— structure analysis using impedance functions.

For direct soil-structure analysis, the base of the structure may be modelled as a rigid structure connected to a flexible model of the foundation. In an analysis of the structure, the base of the structure may be considered as a rigid circular disk for the computation of impedance functions.

Consideration shall be given to the range of likely values of soil stiffness in the analysis. In particular, the possible degradation of soil properties during high-level seismic events, such as the abnormal level earthquake, shall be considered. Appropriate non-linear or reduced soil stiffness properties shall be used.

Soil properties, particularly shear wave velocity, dynamic shear modulus and internal damping are dependent on the shear strains used. These values should be adjusted for the expected strains appropriate to the seismic excitation and the variation in vertical effective stress and voids ratio due to the presence of the structure.

#### 7.5.7.3 Mass and damping

The simulation shall include a representation of the mass of the structure, in accordance with 7.3.6. Enclosed fluids can be included as lumped masses where the heights of the water columns are small and where sloshing is not significant.

Unless a detailed evaluation is made, internal damping of not more than 5 % shall be used to simulate damping from joint structural and hydrodynamic origins for seismic analysis. Any increased value shall be subject to justification based on the expected response. Values of soil damping shall be determined based on the soil type present.

#### 7.5.7.4 Analysis procedures

For the extreme level earthquake (ELE), linear dynamic global structural analysis may be performed using the response spectrum approach. Spectra used shall be in accordance with ISO 19901-2, but the analysis shall incorporate the effect of the structure's response on soil motions, if appropriate. Where degradation of soil properties and non-linear soil-structural interaction or base sliding are important, a non-linear dynamic time history analysis procedure should be adopted to address these effects; the structure itself may be modelled as linear elastic. Where seismic isolation or passive energy dissipation devices are employed to mitigate the seismic risk, a non-linear time history procedure will be required.

Sufficient modes shall be included in the analysis to provide an accurate estimate of total global response. At least two modes shall be considered in each of the two principal horizontal directions, as well as a torsional mode about a vertical axis. This requirement may be considered satisfied if it is demonstrated that with the modes considered in the analysis, at least 90 % of the participating mass of the structure is included in the calculation of the response for each principal horizontal direction.

One design spectrum may be used in each principal horizontal direction, combined with 2/3 of this spectrum in the vertical direction unless a lesser value can be justified based on site-specific data. These spectra may be combined modally using the complete quadratic combination method and directionally using a square root of the sum of squares approach. Alternative methods are permitted with suitable justification that all seismic action effects are included.

Secondary spectra may be developed for the analysis of structural parts such as the topsides or conductor frames to evaluate the response of parts, appurtenances and equipment not modelled for the global analysis. Alternatively, the design of local components may be based on equivalent quasi-static analysis of such components, based on maximum vertical and horizontal accelerations obtained from the global seismic analysis.

Action effects from seismic analysis shall be combined with similar results from permanent and variable actions to produce action effects for structural design. Appropriate directions of seismically induced actions shall be considered to maximize these action effects.

For the abnormal level earthquake (ALE), non-linear seismic analysis may be performed using a time history or transient approach. Unless time histories are available by scaling or by other means, they may be developed numerically from the design spectra. Multiple time histories are required to represent the random nature of seismic ground motions.

The computer model for the ALE analysis shall include discrete models of all primary components of the structure using either linear elastic or material non-linear simulations. Deflection effects shall be evaluated and permanent and variable actions shall be included in the analysis to ensure that second order effects are modelled with sufficient accuracy.

The action effects on structural members that are simulated as linear elastic in either the ELE or the ALE analyses shall be evaluated and used to confirm that these components satisfy ULS criteria. Components that demonstrate ductile response shall be so designed, and shall be assessed against acceptance criteria relevant for the actual limit state with respect to all relevant response parameters.

### 7.5.8 Analysis of accidental or abnormal design situations

Analysis of the structure for accidental events, such as ship, helicopter 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.

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 can be required since the structure will generally deform substantially under the accidental actions. Appropriate boundary conditions shall be provided far enough away from the damaged region for inaccuracies to be minimized.

Global analysis of the structure under accidental actions can be required to ensure that a progressive collapse is not instigated. The analysis should include the weakening effect of damage to the structure in the impacted area. If ductile response of the structure is likely for the impact actions determined, global non-linear analysis can be required to simulate the redistribution of action effects as section resistances are exceeded. The global analysis may be based on a simple model of the structure sufficient to simulate progressive collapse. Deflection effects shall be included where relevant.

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.

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.

#### 7.5.9 Platform removal/reuse

Analysis of the structure for removal shall accurately model the structure during this phase. The analysis shall have sufficient accuracy to simulate relevant pressure differential effects occurring during this stage. The analysis shall include the effect of suction that shall be overcome prior to separation from the seabed, if appropriate. Suitable sensitivity to the suction effect 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.

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.

The condition of the concrete and reinforcement should account for degradation of the materials during the life of the structure. 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.

# 8 Concrete works

## 8.1 General

### 8.1.1 Documentation

It shall be documented that design and execution of the concrete works are performed according to this International Standard. This applies to preparatory works, i.e. the preparation of the project work specifications forming the basis for design and execution, tests and inspections performed during the execution, documentation of the final product, materials used, finished design calculations and execution drawings. The most important of these points with respect to the later operation of the platform shall be brought into the summary report.

All requirements specified in Clause 5 shall be documented, either by project-specific documents or by the manufacturer’s specifications, product standards, etc.

### 8.1.2 Technical documentation

The technical documentation of fixed concrete offshore structures shall comprise:

a) design calculations for individual members and for the complete structure;

b) project work specification.

All technical documentation shall be dated and signed.

The project work specification shall comprise the following:

— drawings, giving all necessary information such as geometry of the structure, amount and position of reinforcing and prestressing steel, etc.; for precast concrete elements, also tolerances, lifting devices, weights, inserts, etc.;

— description of all products to be used, with any requirements for the application of the materials. This information shall be given on the drawings, in the work description or in a separate document. Material specifications, product standards, etc. shall be included;

— the work description, which is a technical document that describes the work, requirements to personnel, methods and equipment, classes of inspection to be applied, any special tolerances, requirements to surface finishing, etc.

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

The work description shall further 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 structure;

— 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 (see also 8.5).

### 8.1.3 Execution documentation

The execution documentation shall comprise the following:

— quality control procedures;

— method statements;

— sources of materials, material test certificates and/or suppliers' attestation of conformity, 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 work specification;

— records of possible dimensional checks at handover;

— a diary or log where the events of significance in the execution process are reported;

— documentation of the inspection performed.

## 8.2 Design

### 8.2.1 Reference standard for design

The design shall be performed in accordance with a recognized reference standard, covering all aspects relevant for the structural design of fixed concrete offshore structures. This subclause identifies areas of design that can be relevant, dependent on circumstances. If these areas are applicable they shall be appropriately covered by the selected reference standard, where necessary supplemented with additional requirements. For complex structures, where higher grades of concrete are used, and where the loading conditions are severe, most or all of the items listed in 8.2.2 to 8.2.17 shall be covered.

The reference standard to be used shall be agreed at an early stage in a project, as the choice of standard can strongly influence the platform geometry and dimensions, while standards not intended for offshore use can be unnecessarily conservative on certain aspects relevant to offshore conditions.

The reference standard to be used shall comply with the basic principles of ISO 19900.

NOTE NS 3473.E [2] has been widely used and is recognized as meeting the requirements of this subclause (8.2.1). However, any standard can be used as a reference standard provided that it is supplemented by additional requirements, where necessary, to ensure that all relevant aspects for the design are properly covered. The following subclauses of 8.2 can be used as a check list when selecting the reference standard to be used for a specific project.

### 8.2.2 Concrete, type and grade

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 and lightweight concretes, the effect of reduced ductility shall be considered. This applies in particular to the stress/strain diagram in compression, to the design parameter used for the tensile strength in the calculation of bond strength and to the transverse shear resistance.

### 8.2.3 Design principles for shell members

Shell types of members are typical in concrete 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 six force components giving stresses in the plane of the member ($N_x$, $N_y$, $N_{xy}$, $M_x$, $M_y$, $M_{xy}$) and all limit states.

### 8.2.4 Design principles for transverse shear

The reference standard shall give the principles required for the design for transverse shear, where the general situation of combinations of simultaneously acting in-plane forces (e.g. tension and compression) and directionality of transverse forces (e.g. principal transverse shear direction) shall be covered. The interaction, which is dependent on the directionality of in-plane 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.

### 8.2.5 Design principles for fatigue

The reference standard shall give the principles required for the design against fatigue for all possible 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 can include certain fatigue-related requirements; these are normally not adequate for offshore applications. The fatigue properties for offshore applications are significantly different, also for materials that pass such general material requirements for fatigue. SN-curves representing the 5 % fractile should be prepared for the design of rebars, and in particular for items that have stress concentrations such as couplers, end anchors and T-heads.

NOTE Materials that are fatigue tested are normally tested at $10^6$ or $2 \times 10^6$ cycles for a given stress range. Offshore structures will typically experience $10^8$ load cycles or more at strongly varying stress ranges, consequently the fatigue testing of materials will not be adequate for all situations.

### 8.2.6 Design principles for durability

The reference standard shall give the design principles and design criteria applicable to ensure a durable design in a marine environment. Important in this context are the following:

a) the selection and combination of the appropriate materials, which shall be in accordance with 8.3;

b) adequate concrete cover of reinforcement, a minimum of 50 mm in the splash zone and a minimum of 40 mm elsewhere; for prestressing tendons, a minimum of 90 mm (these are minimum values; permitted tolerance (negative deviation) on the position of reinforcement should be added to give the nominal cover ($c_{nom} = c_{min} + |\Delta c_{minus}|$). A typical value is $\Delta c_{minus} = -10$ mm);

c) limitation of crack widths under SLS conditions.

### 8.2.7 Design principles for watertightness

The reference standard shall give the design principles for watertightness control. Watertightness shall be considered under SLS conditions. This shall apply to the ingress of water in structures while afloat and during the installed condition in situations where there is internal underpressure. It shall also apply to the possibility of leakage, in particular of stored hydrocarbons from structures having internal overpressure. Leakage shall also be considered in the design of members for which maintaining a pressure gradient is vital, such as occurs in suction foundations and when using air cushions.

Adequate watertightness or leakage control shall be required in ULS and ALS checks for those conditions where leakage can cause collapse or loss of the structure due to flooding, or where a pressure condition that is required to maintain equilibrium can be lost.

### 8.2.8 Design principles for prestressed concrete

The reference standard shall give the principles required for the design of prestressed concrete, including principles for partial prestressing when appropriate.

The effect of the presence of empty ducts during certain phases of the execution 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. Anchorages shall be placed in positions and protected in such ways that they are not susceptible for wear or damages.

### 8.2.9 Design principles for second order effects

The reference standard shall give the principles required for the design of all relevant types of members for second order effects, including buckling in the hoop direction of shell types of members.

### 8.2.10 Principles for handling water pressure in pores and cracks

The reference standard shall give the principles required to assess the effects of pressure from water or stored fluids penetrating into cracks and pores of the concrete, affecting both the action effects and the resistance. The methods to be used will be dependent on how water pressure is applied in the initial calculation of action effects. As water pressure will penetrate into shear cracks, shear force capacity enhancement (or shear force reduction) for loads near supports ($x < 2{,}5\ d$) cannot be applied for shear caused by water pressure.

### 8.2.11 Design principles for discontinuity regions

The reference standard shall give the principles for the local design of discontinuity regions where strut and tie models can be used to demonstrate the mechanisms for proper force transfer.

### 8.2.12 Principles for design against imposed deformations

The reference standard may additionally give the principles required to permit design against 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 potential brittle failure modes.

Imposed deformations can often be seen as imposed additional strain that affects crack widths in SLS, but with minor effect on section capacity for bending and axial force in ULS. While the effects with respect to transverse shear are more complex, modifications to shear forces due to cracking can be made only where it can be demonstrated that the assumed reduction will take place, using upper values for cracking strength and stiffness, before failing in shear.

### 8.2.13 Increase in strength of concrete with time

The reference standard may give guidance for assessing the effect of a gain in strength beyond 28 days and also for assessing the effect of sustained actions, or repeated actions at high stress levels, on the reduction in strength of concrete, where the gain in strength is intended for use in the design.

### 8.2.14 Design for fire resistance

The reference standard shall give design principles required for demonstrating adequate fire resistance of members subjected to fire, including relevant material and strength parameters at elevated temperatures.

### 8.2.15 Design for earthquakes

In zones with low to moderate seismic activity, the action effects obtained from an analysis in which the 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.

In cases where the seismic action causes large amplitude cyclic deformations, which can only be sustained by 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 and to experience excessive deformations shall be carefully detailed to ensure appropriate ductility and confinement.

### 8.2.16 Design of embedment

The reference standard shall give design principles for the anchorage of load-bearing embedment into concrete. Additional reinforcement adequate to transfer all tensile forces into the concrete and anchoring the pull-out forces on the opposite face should normally be provided.

### 8.2.17 Partial factors for material, $\gamma_M$

The partial factors for material, $\gamma_M$, shall be such that a safety level consistent with that inherent in ISO 19900 and this International Standard is obtained, and is consistent with the partial factors that are used.

NOTE The material factors to be used will depend on the way the design parameters are defined and specified in the reference standard used, as well as on the corresponding partial factors for action. If the design is performed in accordance with NS 3473.E [2], material factors according to that standard can be used, with the partial factors for action of Table 1. If Eurocode 2 (EN 1992)[3] is used as a reference standard, supplemented with additional requirements to satisfy the requirements of 8.2.2 to 8.2.16, the material factors for reinforcement and concrete recommended in that standard can be applied with the action factors of Table 1, provided that the factors $\alpha_{cc}$ and $\alpha_{ct}$ are not taken to be greater than 0,85. If the guidance of EN 1992:2004, Annex A, is used, the material factors for concrete and reinforcement cannot be taken to be less than 1,40 and 1,10, respectively. For other reference standards, the material factors can require a separate evaluation.

## 8.3 Materials

### 8.3.1 General

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

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

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

NOTE Fibre reinforcement can be applied in order to improve certain aspects of structural behaviour (e.g. cracking, fire resistance, etc.) but not included in the general calculation of section resistance.

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

Materials complying with recognized product standards are acceptable, provided the requirements of this document are met.

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

Material can be rejected at any stage of the execution process, notwithstanding any previous acceptance or certification, if it is established that the conditions upon which the approval or certification was based were not fulfilled.

### 8.3.2 Material requirements — Concrete constituents

#### 8.3.2.1 Cement

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

The cement shall be delivered with a mill certificate giving, as a minimum, the chemical and mineralogical composition, and the test values for all those properties for which there are requirements. Cement shall be tested according to approved methods. Table 6 lists different types of tests that can be required.

The tricalcium aluminates ($C_3A$) content 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 approximately 5 %.

The mill certificate shall, in addition to demonstrating compliance with specified requirements, also state the type/grade of cement with reference to the approved standard/project work specification, the batch identification and the tonnage.

The requirement for a mill certificate may be waived provided the cement is produced and tested under a national or international certification scheme, and all required properties are documented based on statistical data from the producer.

**Table 6 — Cement testing**

| Physical properties | Chemical properties |
|---|---|
| Strength in mortar | Loss on ignition |
| Initial/settling time | Insoluble residue |
| Soundness | Sulphate content |
| Fineness | Chloride content |
| | Pozzolanity |

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

— Portland cements;

— Portland composite cements (with silica, fly ash or slag and minimum 80 % clinker).

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

— Portland composite cements (with other pozzolanas or clinker content below 80 % clinker);

— Blast-furnace cements (less than 64 % clinker);

— Pozzolanic cements (less than 64 % clinker);

— Composite cement (less than 64 % clinker).

NOTE The above types of cement have characteristics specified in regional and national standards, while no ISO standards for cement exist. Cements can be specified in grades based on the 28 day strength in mortar (e.g. 32,5 MPa; 42,5 MPa; 52,5 MPa).

Cements shall normally be classified as normal-hardening, rapid-hardening or slowly hardening cements.

#### 8.3.2.2 Mixing water

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

Water resulting in a concrete strength of less than 90 % of that obtained by using distilled water tested at 7 days shall not be used. Neither shall water be used that reduces the setting time to less than 45 min or changes the setting time by more than 30 min relative to distilled water.

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

Water sources shall be investigated and approved for their suitability and dependability for supply.

#### 8.3.2.3 Normal-weight aggregate

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, expand or shrink.

They shall be resistant to decomposition when wet. They shall not react with the products of hydration of the cement and shall not affect the concrete adversely. Marine aggregates shall not be used unless they are properly and thoroughly washed to remove chlorides, see 8.3.3.1.

Aggregate sources shall be investigated and approved for their suitability and for supply dependability. Aggregate shall be delivered with a test report containing, at least, the following:

— 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;

— potential reactivity with alkalis in cement;

— petrographical composition and properties that can affect the durability of the concrete.

Normal-weight aggregate shall 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. Aggregate shall be free from harmful substances in quantities that can affect the properties and the durability of the concrete adversely. Examples of harmful substances are claylike and silty particles, organic materials, sulphates and other salts.

Aggregate shall be evaluated for risk of alkali aggregate reaction (AAR) in concrete according to recognized test methods. Suspect aggregate shall not be used unless specifically tested and approved. The approval of an aggregate that can combine with the hydration products of the cement to cause AAR shall state which cement the approval applies to.

Aggregates of different grading shall be stockpiled and transported separately.

Testing of aggregate shall be carried out at regular intervals both at the quarry and at the construction site during concrete production. The frequency of testing shall be determined after taking the quality and uniformity of supply and the concrete production volume into account, in accordance with requirements of national standards in the place of use.

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

Maximum aggregate size shall be specified based on considerations of concrete properties, spacing of reinforcement and cover to the reinforcement; the maximum aggregate size should not be less than 16 mm.

#### 8.3.2.4 Lightweight aggregate

Lightweight aggregate 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.

Lightweight aggregate shall have uniform strength properties, stiffness, density, degree of burning, grading, etc. The dry density shall not vary more than ±7,5 %.

#### 8.3.2.5 Additions

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

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

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

#### 8.3.2.6 Admixtures

Only admixtures with established suitability shall be used. Admixtures to be used in concrete shall be tested under site conditions with the cement and additions to be used to verify that these products will yield the required effects without impairing the other properties required. The risks involved in overdosage shall be assessed. A test report shall be prepared to document such verification. The test report shall form a part of the concrete mix design documentation.

The extent of testing of admixtures shall normally be in accordance with the requirements given in national standards or recognized international standards.

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 plastic cracking.

#### 8.3.2.7 Repair materials

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 elasticity and temperature-dependent properties.

Requirements for non-cementitious 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.

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

### 8.3.3 Material requirements — Concrete

#### 8.3.3.1 Concrete

The concrete composition and the constituent materials shall be selected to satisfy the requirements of this International Standard and the project work 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.

The required properties of fresh and hardened concrete shall be specified. These required properties shall be verified by the use of recognized testing methods, international standards or recognized national standards. Compressive strength shall always be specified; in addition, tensile strength, modulus of elasticity ($E$-modulus) and fracture energy can be specified. Properties which can cause cracking of structural concrete shall be accounted for, i.e. creep, shrinkage, heat of hydration, thermal expansion and similar effects. The durability of structural concrete is related to permeability, absorption, diffusion and resistance against physical and chemical attacks in the given environment; a low water/cement ratio is generally required in order to obtain adequate durability. The concrete shall have an effective water/cement ratio not greater than 0,45. In the splash zone this ratio shall not be higher than 0,40; this also applies to areas that can be exposed to severe frost action.

The aggressiveness of hot oil with respect to the concrete and the build-up of $H_2S$ should be considered, see 10.2.4.

Where 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 ratio, $m$, expressed by:

$$m = w/(c + k\,a)$$

where

$w$ is the mass of water;

$c$ is the mass of cement;

$k$ is an efficiency factor for active additions (type II) such as silica, fly ash and slag, where the efficiency of the material as replacement of cement with respect to durability in the given environment is considered;

$a$ is the mass of the active addition (type II).

In this calculation, the total quantity of such additions ground into the cement and/or added in the mixer shall not exceed the limits permitted by the cement standard for Portland composite cements or for blast-furnace cement if slag is the only addition added to the clinker. *In lieu* of other values, the following values of $k$ may be used:

$k = 1$ to 2 for silica

$k = 0{,}5$ to 0,7 for slag

$k = 0{,}2$ to 0,4 for fly ash

Concrete subjected to freezing and thawing shall have adequate frost resistance. This frost resistance shall be demonstrated by appropriate test methods. For Portland and Portland composite cements with fly ash or silica and with more than 80 % clinker, this requirement may be considered satisfied when entrained air is used if the air content, at the form, of the fresh concrete made with natural aggregate is at least 4 % for a maximum particle size of 40 mm, or at least 5 % for a maximum particle size of 20 mm. With clinker contents less than 80 % the frost resistance shall be demonstrated by appropriate testing methods to evaluate scaling and freezing and thawing resistance. In concrete with slag content of more than 35 % the concrete shall be carbonated prior to testing of frost resistance. The entrained air should give an evenly distributed air void system. In areas with severe frost the specific surface should be larger than 25 $mm^2/mm^3$, the spacing factor should not exceed 0,25 mm.

The total chloride ion content of the fresh concrete shall not exceed 0,10 % of the weight of cement in ordinary reinforced concrete and in concrete containing prestressing steel.

In the splash zone, the cement content shall not be less than 400 $kg/m^3$. For reinforced or prestressed concrete outside the splash zone, the minimum cement content shall be 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 $kg/m^3$.

For concrete exposed to sea water and stored oil, the characteristic cylinder strength at 28 days should not be less than 40 MPa.

Where lightweight aggregate 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$^3$.

Where 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.

#### 8.3.3.2 Grout and mortar

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

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

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

All materials shall be batched by mass, except the mixing water, which may also be batched by volume. The batching shall be within an accuracy of 2 % for cement and admixtures and 1 % for water. The water/cement ratio shall not be higher than 0,44. Batching and mixing shall be such that specified requirements for fluidity and bleeding in the plastic condition, volume change when hardening, and strength and stiffness when hardened are complied with.

### 8.3.4 Material requirements — Reinforcement

#### 8.3.4.1 Reinforcing steel

Reinforcing steel shall comply with ISO 6935-1 and ISO 6935-2 or relevant national standards on reinforcing steel. Consistency shall be ensured between material properties assumed in the design and requirements of the standard used. In general, hot-rolled, ribbed bars of weldable quality and with high ductility shall be used. Where the use of seismic detailing is required, the reinforcement provided shall meet the ductility requirements of the reference standard used in the design.

Fatigue properties and S-N curves shall be consistent with the assumptions of design.

Reinforcing 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.

Galvanized reinforcement may be used where requirements are made to ensure that there will be no reactions with the cement that have a detrimental effect on the bond to the galvanized reinforcement.

Stainless steel may be used provided the requirements for mechanical properties of ordinary reinforcement steel are met.

#### 8.3.4.2 Mechanical splices and end anchorages for reinforcement

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

Couplers that give a permanent slip on unloaded specimen of more than 0,1 mm after 10 cycles of loading to 75 % of nominal yield strength should not be used in structures where tightness and fatigue is a concern.

Mechanical splices and end anchorages shall be delivered with a works certificate.

Friction-welded end anchorages on rebars (T-heads) shall be qualification tested in advance with the actual type of rebar and be routinely tested during production. The test programme 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.

#### 8.3.4.3 Approval of weld procedures

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

### 8.3.5 Material requirements — Prestressing steel

#### 8.3.5.1 Prestressing steel

Prestressing steel as a product shall comply with the relevant parts of ISO 6934 and/or relevant national standards on prestressing steel.

Prestressing steel shall be delivered with a works certificate.

#### 8.3.5.2 Components for the prestressing system

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

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

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

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

### 8.3.6 Material requirements — embedded materials

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

## 8.4 Execution

### 8.4.1 Falsework and formwork

#### 8.4.1.1 Basic requirements

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, and

— 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.

#### 8.4.1.2 Materials for formwork

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 these exist. Properties of the specific materials, such as shrinkage, shall be taken into account if they can affect the end product.

The materials employed shall be consistent with any special requirements for the surface finish or later surface treatment.

#### 8.4.1.3 Release agents

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.

#### 8.4.1.4 Falsework

The method statement shall describe the method of erection and dismantling of temporary structures. The method statement shall specify the requirements for handling, adjusting, intentional precambering, loading, unkeying, striking and dismantling.

Deformations of formwork during and after concreting shall be limited to prevent deleterious cracking in the young concrete. This can be achieved by limiting the deformations and by organizing the casting operations in a manner so as to avoid harmful deformations.

#### 8.4.1.5 Formwork

Formwork shall keep the concrete in its required shape until it has hardened.

Formwork and the joints between boards or panels shall be sufficiently tight against loss of water and fines.

Formwork that absorbs moisture or facilitates evaporation shall be suitably wetted to minimize water loss from the concrete, unless the formwork was designed specifically for that purpose.

The internal surface of the formwork shall be clean. When slipforming is used, the form panels shall be thoroughly cleaned and a grease-like mould-release agent shall be applied prior to assembling of the form.

#### 8.4.1.6 Slipform systems

Where using the slipforming method, the design and erection of the form and the preparation of the works shall take into account the difficulties of controlling the geometry and the stiffness of the entire working platform. The entire slipform structure shall be designed with the appropriate stiffness and strength. Due account shall be taken of friction against hardening concrete, weight of materials stored on the form, systems for adjusting geometry such as diameter and wall thickness, as well as climatic conditions to be expected during the slipforming period.

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 are to 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.

The materials applied in the form may be either steel or wood, and shall comply with the requirements for formwork materials. The form shall have a height and batter consistent with the concrete to be used. The slipforming rate and other conditions affecting the hardening process of the concrete shall be such as to reduce or eliminate the tendency for lifting cracks.

The slipform shall have a hanging platform below the main form, giving access for application of curing as well as inspection and, if necessary, light repair of the hardening concrete when appearing from under the slipform.

The concrete cover to the reinforcement shall be kept within the tolerances using sufficiently long and stiff steel guides between the reinforcement and the form, adequately distributed around the form.

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.

#### 8.4.1.7 Jumpforming systems

Jumpforming systems, when used, shall have adequate strength and stiffness for all loads exerted during the erection and casting period. There shall be a robust system for support of the forms in the previously cast concrete. Inserts for support shall be approved for the actual application.

The jumpform, when installed, shall allow the necessary preparation and cleaning of construction joints. The jumpform system shall accommodate the necessary walkways and access platforms to ensure that the concreting works can be performed in an appropriate manner.

#### 8.4.1.8 Inserts in formwork, recesses and block-outs

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.

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 the item, the concrete or the reinforcement.

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 be cast with normal concrete. Their surfaces shall be keyed or slanted and prepared as construction joints.

#### 8.4.1.9 Removal of formwork and falsework

Falsework and formwork shall not be removed until the concrete has gained sufficient strength

— to resist damage to surfaces that can arise during the striking,

— to take the actions imposed on the concrete member at that stage,

— to avoid deflections beyond the specified tolerances due to elastic and inelastic (creep) behaviour of the concrete.

Striking shall be made in a manner that will not subject the structure to overload or damage.

Propping or re-propping may be used to reduce the effects of the initial loading, subsequent loading and/or to avoid excessive deflections. Propping can be required in order to achieve the intended structural behaviour of members cast in two or more casting operations.

If formwork is part of the curing system, the time of its removal shall take into account the requirements of 8.4.5.5.

### 8.4.2 Reinforcement

#### 8.4.2.1 Reinforcement — Bending, cutting, transport and storage

The surface of all reinforcing steel shall be free from loose rust, mill-scale and deleterious substances that can adversely affect the steel or concrete or the bond between them. It shall be stored free of the ground, and protected from mechanical damage, rupture of welds and reduction of cross-section through corrosion.

The cutting and bending of reinforcing steel shall be performed conforming to national standards or other relevant documents. Particular care shall be taken in cases where the execution, reinforcement supply and design is not performed according to a consistent suite of standards. The following requirements shall apply:

— bending shall be done at a uniform rate;

— bending of reinforcement with temperature below 0 °C shall only be performed as permitted by national standards, and in accordance with procedures prepared for the particular construction site;

— unless specifically permitted by the project work specification, bending using heat treatment is not permitted.

For bars, wires, welded reinforcement and fabric bent after welding the diameter of the mandrel used should be as specified by design and in accordance with the standard relating to the type of reinforcement. Under no condition shall reinforcement be bent over a mandrel with a diameter which is not at least 1,5 times greater than a test mandrel used for bending tests to document what that steel and bar diameter can take without cracking or damage.

In-place bending of steel in the formwork can be allowed if it can be demonstrated that the prescribed bending radius is obtained, and the work can be performed without misplacing the reinforcement.

The straightening of bent bars is prohibited unless the bars are originally bent over a mandrel with a diameter of at least 1,5 times greater than a test mandrel used to document what that particular steel and bar diameter can take, and be straightened without damage; a procedure for such work shall be prepared and approved.

Reinforcement delivered on coil shall be handled using the appropriate equipment; straightening shall be performed according to approved procedures and all required mechanical properties shall be maintained.

Prefabricated reinforcement assemblies, cages and elements shall be adequately stiff and strong to be kept in shape during transport, storage, placing and concreting; they shall be accurate so that they meet all the requirements regarding placing tolerances for reinforcement.

#### 8.4.2.2 Welding

Welding is only permitted on reinforcing steel that is classified as weldable in the relevant product standard, or according to the relevant parts of ISO 6935 or national standards, and only if specified in the project work specification.

Welding shall be used and performed in accordance with specifications by design, and shall conform to special requirements in national standards as relevant.

Welding should not be executed at or near bends in a bar, unless specifically approved by design.

Welding of galvanized or epoxy-coated reinforcement is only permitted when a procedure for repair is specified and approved.

All welding shall be performed to a qualified welding procedure, and only by qualified welders.

#### 8.4.2.3 Joints

Joints on bars shall be made by laps or couplers. Only couplers whose effectiveness is tested and approved may be used. Butt welds can be permitted to a limited extent but only when subjected to prequalification testing with non-destructive examination and visual quality inspection of all welds during execution.

The length and position of lapped joints and the position of couplers shall be in accordance with design drawings and the project work specification. Staggering of such joints shall be considered in design, the overlapping bars should be placed in contact and preferably tied together.

#### 8.4.2.4 Assembly and placing of reinforcement

The reinforcement shall be placed according to the design drawings and fixed within the tolerances for fixing of reinforcement in this International Standard, and secured so that its final position is within the tolerances given in this document.

Assembly of reinforcement should be done by tie wire. Spot or tack welding is not allowed for the assembling of reinforcement unless permitted by national standards and the project work specification, and due account has been taken of the risk of fatigue failure of the main rebar at the weld.

The specified cover of the reinforcement shall be maintained by the use of suitable chairs and spacers. Spacers in contact with the concrete surface in a corrosive atmosphere shall be made from concrete of at least the same quality as the structure.

In areas of congested reinforcement, measures shall be taken to ascertain that the concrete can flow and fill all voids without segregation and can be adequately compacted.

### 8.4.3 Pre- and post-tensioning

#### 8.4.3.1 Transport and storage

All components of the entire prestressing assembly or system, consisting e.g. of prestressing steel, ducts, sheaths, anchorage devices, couplers, as well as prefabricated tendons and tendons fabricated on site, shall be protected from harmful influences during transport and storage and also whilst placed in the structure prior to permanent protection. Materials that have been damaged or corroded shall be replaced.

#### 8.4.3.2 Fabrication of prestressing assembly

The prestressing assembly, e.g. all components of the tendons, shall be assembled in accordance with the relevant standards, where these exist, or suppliers' specifications or approval documents, and as shown in the construction drawings.

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.

Documentation of prestressing steel of different deliveries shall be made in the as-built records.

Cutting shall be done by an appropriate method in a way that is not harmful.

Prestressing steel shall not be subjected to welding. Steel in the vicinity of prestressing steel shall not be subjected to oxygen cutting or welding, except when sufficient precautions have been taken to avoid damage to prestressing steel and ducts.

#### 8.4.3.3 Placing of the prestressing assembly

The prestressing assembly shall be placed in compliance with the project's/supplier's 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 a kink of any kind.

#### 8.4.3.4 Post-tensioned tendons

The straight entry into anchorages and couplers as well as the coaxiality of tendon and anchorage shall be as specified by the supplier's specifications or system approval documents.

Vents and drains on the sheaths shall be provided at both ends, and at all points where air or water can accumulate. In the case of sheaths of considerable length, inlets, vents and drains can be necessary at intermediate positions. As an alternative to drains, other documented methods of removing water may be considered.

Inlets, vents and drains shall be properly marked to identify the cable.

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.

Sheaths shall be checked after pouring of concrete to ensure sufficient passage for the tendons.

Sheaths shall be cleared of any water immediately prior to tendon threading.

#### 8.4.3.5 Tensioning of tendons

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 programme 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.

Stressing devices shall be as permitted for the prestressing system.

The valid calibration records for the force-measuring devices shall be available on site before the tensioning starts.

Application and/or transfer of prestressing forces to a structure may only be done 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.

#### 8.4.3.6 Pre-tensioning

If, during stressing, the calculated elongation cannot be achieved within a range of ±3 % for a group of tendons or ±5 % for a single tendon within the group for the specified tensioning force, action shall be taken in accordance with the method statement, with regard either to the tensioning programme or to the design.

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.

Where the fresh concrete cannot be cast in due time after tensioning, temporary protective measures shall be taken which will not affect the bond or have detrimental effects on the steel and/or the concrete.

#### 8.4.3.7 Post-tensioning

Tensioning shall not take place when concrete temperatures are below +5 °C within the structure, unless special arrangements can assure the corrosion protection of non-grouted tendons. Tensioning is prohibited if there is a risk of the grout freezing. Tensioning is prohibited at air temperatures below −10 °C.

Where, during the stressing operation, the calculated elongation cannot be achieved within a range of ±5 % for a group of tendons or ±10 % for a single tendon within the group for the specified tensioning force, action shall be taken in accordance with the method statement, with regard either to the tensioning programme or to the design.

In the case of deviations from the planned performance during tensioning, tendon-ends shall not be cut off and grouting is not permitted. Works that can impair re-tensioning shall not be carried out. No tendons shall be cut if the obtained elongations deviate from the theoretical by more than 5 %, without design approval. Further work shall be postponed until the tendon has been approved or further action decided.

NOTE In case of deviations between theoretical and obtained results, tests to confirm friction factors and $E$-modulus of the tendon assembly can be necessary.

The prestressing tendons should be protected from corrosion in the period from threading to prestressing. This period should normally not be allowed to exceed four weeks without special precautions being taken.

### 8.4.4 Protective measures, grouting, greasing, concreting

#### 8.4.4.1 General

Tendons placed in sheaths in the concrete, couplers and anchorage devices shall be protected against detrimental corrosion. This protection shall be ensured by filling all voids with a suitable grouting/injection material such as cement grout, grease or wax. Anchorage areas and end caps shall be filled and protected as well as the tendons; inlets and outlets shall be suitably sealed.

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 two weeks. If a delay is likely to permit corrosion, protective measures should be considered in accordance with national regulations or recommendations by the supplier.

A method statement shall be provided for the preparation and execution of the grouting/injection. All important data/observations from the grouting shall be reported in a log, e.g. volume consumed compared to theoretical volume, temperature of the structure and mix proportions and problems/stops.

Grouting devices shall be as permitted for the prestressing system.

#### 8.4.4.2 Unbonded tendons

Anchorage areas of unbonded tendons or single strands, their sheaths and end caps shall be filled by non-corrosive grease or wax. End caps shall be encased in concrete tied to the main structure by reinforcement.

Sheathed unbonded tendons shall be adequately sealed against penetration of moisture at their ends.

#### 8.4.4.3 Grouting operations

Grouting with cement-based grouts shall only be done at temperatures in the structure in the range of +5 °C to +35 °C. The temperature of the grout shall neither be less than +10 °C nor above +25 °C. If a frost-resistant grout is used, grouting at lower temperatures than +5 °C can be permitted.

Where the temperature in the structure is above +35 °C, grouting can be permitted provided special precautions valid in the place of grouting can ensure a successful grouting.

Grouting shall be carried out at a continuous and steady rate from the lowest inlet until the emerging grout through anchor heads and outlets has the appropriate quality, not affected by evacuated water or preservation oil from the duct. In vertical ducts, the grouting pressure shall be given particular attention. Normally the grout pressure inside the duct should not be allowed to exceed 2 MPa, unless permitted by the design. Non-retarded grout and grout with an expanding admixture shall be used within 30 min after mixing.

In vertical or inclined ducts or ducts of particularly large diameter, post-injection can be necessary in order to remove bleed water or voids. Post-injection shall be performed before the grout has stiffened. If voids are detected at inlets or outlets after the grout has stiffened, post-grouting shall be carried out, if required, by vacuum grouting.

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.

Requirements for vacuum grouting or re-injection shall be made in case of discovery of a blockage in the duct. Ducts shall, under no circumstances, be left empty and ungrouted without specific approval by design.

After completion of grouting, unintended loss of grout from the ducts shall be prevented by sealing them under a minimum pressure of 0,5 MPa for a minimum of 1 min.

If grouting of a duct is interrupted, corrective actions, such as washing out all fresh grout, shall be taken. No ducts shall be left with incomplete filling of grout.

#### 8.4.4.4 Greasing operations

Greasing shall be carried out at a continuous and steady rate. After completion of greasing, unintended loss of grease from the ducts shall be prevented by sealing them under pressure.

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.

### 8.4.5 Concreting

#### 8.4.5.1 Procedures before commencing work

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, compacting, formwork striking and curing.

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 can be regarded as valid pretesting. The quality control procedures shall be available on site. The procedures shall include the possible corrective actions to be taken in the event of non-conformity with the project work specification or agreed procedures.

#### 8.4.5.2 Procedures in connection with each delivery

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 testing of conformity with given requirements shall be taken at the point of placing. In the case of ready-mixed concrete produced according to a recognized certification scheme certifying conformity with all given requirements, only identity testing at the point of delivery shall be required.

NOTE Identity testing is testing to verify that the concrete comes from a conforming population.

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.

Concrete may be cooled or heated either during mixing, during transport to the site or at the site if documented acceptable by pretesting. The temperature of the fresh concrete shall be within the specified or declared limits.

Concrete may be retempered by post-dosing of admixtures if documented acceptable by pretesting.

#### 8.4.5.3 Pre-concreting operations

Initial testing of concreting by trial casting, where required, shall be documented before the start of execution. If pumping of concrete is used, adequate back-up or emergency plans shall be prepared to ensure that blockage of piping will not lead to an unacceptable end result.

All preparation works shall be completed, inspected and documented as required for the inspection class before the casting is initiated.

Construction joints shall be prepared and roughened in accordance with project work specifications. In monolithic structures an adequately roughened surface can be obtained by the application of a surface retarder on the fresh concrete and later cleaning by water jetting. Construction joints shall be clean, free of laitance and thoroughly saturated with water, but surface dry.

Construction joints in contact with the section to be cast shall have a temperature that does not result in freezing of the adjoining concrete.

The form shall be free of detritus, ice, snow and standing water.

If the ambient temperature is forecasted to be below 0 °C at the time of casting or in the curing period, precautions shall be planned to protect the concrete against damage due to freezing.

Where the ambient temperature is forecasted to be above 30 °C at the time of casting or in the curing period, precautions shall be planned to protect the concrete against damaging effects of high temperatures.

#### 8.4.5.4 Placing and compaction

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.

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 the top surface shall be avoided by re-vibration.

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.

Concrete shall be placed in such a manner as to avoid segregation. Free fall of concrete from a height of more than 2 m shall not occur unless the mix is demonstrated to allow this without segregation. Concrete shall be compacted by means of high-frequency vibrators or by alternative methods that can be demonstrated to give adequate compaction. 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.

Alternative methods to the use of vibrators in order to obtain an adequately compacted concrete are permitted, provided they are able to be documented for the relevant type of conditions by trial casting; this can include the use of self-compacting concrete (SCC), provided the concrete composition complies with 8.3.

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.

#### 8.4.5.5 Curing and protection of hardening concrete

Concrete in its early life shall be cured and protected

- to minimize plastic shrinkage and losses in strength and durability,
- to ensure adequate surface strength,
- to ensure adequate surface zone durability,
- to prevent freezing,
- to prevent harmful vibration, impact or damage.

The curing methods to be employed shall prevent significant evaporation from the concrete surface. This shall preferably be achieved by keeping the surface permanently wet during the required curing period. Sea water shall not be used for curing. Fresh concrete shall not be permitted to be submerged in sea water until an adequate strength of the surface concrete is obtained.

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, measures to reduce loss of water shall be applied prior to finishing.

The duration of applied curing shall be a function of the development of the concrete properties in the surface zone and depends on the climate conditions prevailing in the region where the concrete member is located. Curing shall be applied until the strength of the concrete has reached a degree of hydration, characterized by strength proportions, given in Table 7.

**Table 7 — Minimum values of proportions of the strength of concrete at the end of curing**

| Climatic conditions when curing | | | Strength proportion | | |
|---|---|---|---|---|---|
| | | | Submerged zone | Splash zone | Other zones |
| H | Humid | RH $>$ 80 % | 0,5 | 0,6 | 0,5 |
| M | Moderate | 65 % $<$ RH $\leqslant$ 80 % | 0,6 | 0,7 | 0,6 |
| D | Dry | 45 % $<$ RH $\leqslant$ 65 % | 0,6 | 0,7 | 0,6 |
| VD | Very dry | RH $\leqslant$ 45 % | 0,7 | 0,8 | 0,7 |

The strength proportion is defined as the ratio between the mean concrete strength at the end of the curing period and the mean strength at the age of 28 days, made, cured and tested in accordance with ISO 1920-3 and ISO 1920-4.

The duration of curing may be 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.

The equivalence of curing may also be proven by applying suitable methods to measure the surface permeability or the strength of the concrete cover at the end of curing.

The curing methods given in Table 8 shall be used individually or in sequence; method C should normally be used in combination with method A or B.

**Table 8 — Methods of curing**

| Method | | Measure |
|---|---|---|
| A | Without adding water | Keeping the formwork in place.<br>Covering the concrete surface with vapour-proof sheets that are secured at the edges and joints to prevent through-draughts. |
| B | Keeping the concrete moist by addition of water | Placing of wet coverings on the concrete surface and protection of these coverings against drying by vapour-proof sheets.<br>Keeping the concrete surface visibly wet by spraying with clean water.<br>Ponding the concrete surface with clean water. |
| C | Use of curing compounds | Curing compounds shall have an established suitability. Suitability may be based on testing, comparing the effectiveness of the curing compound to the effectiveness of water. Curing compounds may be used in accordance with international or national standards. |

Curing compounds are not permitted on construction joints and 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 on bonding.

Early-age thermal cracking resulting from thermal gradients or restraints from adjoining members and previously cast concrete shall be minimized. In general, a differential temperature across a section should not be allowed to exceed 10 °C per 100 mm.

The concrete surface temperature shall not fall below 0 °C until the concrete surface has reached a compressive strength of at least 5 MPa and until the strength is also adequate for all actions in frozen and thawed conditions 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 parts that can become saturated shall be protected from the ingress of external water for at least seven days.

The peak temperature of the concrete within a member shall not exceed 70 °C unless data are provided documenting that higher temperatures will have no significant adverse effects such as reduced strength or delayed ettringitte formation.

The set concrete shall be protected from vibrations and impacts that could damage the concrete or its bond to reinforcement.

The surface shall be protected from damage by heavy rain, flowing water or other mechanical influences.

#### 8.4.5.6 Post-concreting operations

After removal of formwork, all surfaces shall be inspected for conformity to the requirements.

The surface shall be protected from damage and disfigurement during construction.

#### 8.4.5.7 Special execution methods

Special methods shall be stated in the project work specification or agreed.

Special execution methods shall not be permitted if they can have an adverse effect on the structure or its durability. Special execution methods can be required in cases where concrete with lightweight or heavyweight aggregate is 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 can be required as part of the documentation and approval of the methods to be used.

Concrete for slipforming shall have an appropriate setting time. Slipforming shall be performed with adequate equipment and methods for transportation of concrete to the form and distribution within the form. The methods employed shall ensure that the specified cover of the reinforcement, the concrete quality and the surface finish are achieved.

#### 8.4.5.8 Finish and repair

Formed finishes shall be obtained by the use of properly designed formwork of timber, plywood, plastic, concrete or steel. Small blemishes caused by air or entrapped water can be expected, but the surface shall be free from voids, honeycombing or other blemishes.

Unformed finishes shall be screeded to produce a uniform and smoothed surface. After the concrete has stiffened sufficiently, this finish shall be floated by hand or by a machine producing a satisfactorily uniform surface.

Prior to construction there shall be plans and procedures prepared and agreed for remedying any foreseen defective work. It shall be clearly established what types of repair will require involvement by design.

Trial construction can be required to confirm that the required quality and finish can be achieved by the proposed concrete mix and method of construction, and to serve as a reference standard by which the quality of the works can be assessed.

### 8.4.6 Execution with precast concrete elements

#### 8.4.6.1 General

The following subclauses give requirements for construction operations involving precast concrete elements, whether produced in a factory or a temporary facility at or outside the site, and are applicable to all operations from the time the elements are available on the site, until the completion of the work and final acceptance.

The manufacture and design of precast concrete elements, when used in concrete offshore structures, are covered by this document, and shall meet all requirements to materials, strength and durability as if they were cast *in situ*.

Where precast concrete elements are used, these shall be designed for all temporary conditions as well as for 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, such as integral *in situ* casting, toppings and reinforcement,

— load and arrangement conditions due to transient situations during execution of the *in situ* works, and

— differential time-dependent behaviour for precast and *in situ* concrete.

Precast concrete elements shall be clearly marked and identified with their intended positions in the final structure. As-built information and records of conformity testing and control shall be available.

A complete erection work programme 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 programme is approved.

#### 8.4.6.2 Handling and storage

Instructions shall be available giving the procedures for the handling, storage and protection of the precast concrete elements.

A lifting scheme defining the suspension points and forces, the arrangement of the lifting system and any special auxiliary provisions shall be available. The total mass and centre of gravity for the concrete elements shall be given.

Storage instructions for the precast concrete element shall define the storage position and the permissible support points, the maximum height of the stack, the protective measures and, where necessary, any requirements to maintain stability.

#### 8.4.6.3 Placing and adjustment

Requirements for the placing and adjustment of the precast concrete elements shall be given in the erection programme, which shall also define the arrangement of the supports, the necessary props and possible temporary stability provisions. Access and work positions for lifting and guiding of the concrete elements shall be defined. The erection of the precast concrete elements shall be performed in accordance with the assembly drawings and the erection programme.

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.

During installation, the correct position of the precast concrete elements, the dimensional accuracy of the supports, the conditions of the precast concrete element and the joints, and the overall arrangement of the structure shall be checked and any necessary adjustments shall be made.

#### 8.4.6.4 Jointing and completion works

The completion works are performed on the basis of the requirements given in the erection programme and taking climatic conditions into account.

The execution of the structural joints shall be made in accordance with the project work 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.

Connectors of any type shall be undamaged, correctly placed and properly executed to ensure an effective structural behaviour.

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.

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.

### 8.4.7 Embedded components

#### 8.4.7.1 General

Components that are to be cast-in and form an integral part of the permanent structure, serving either structural or functional purposes for either permanent or temporary use, shall be produced and installed in accordance with specifications and drawings. Embedded items shall be stiff enough to preserve their shape unaffected by the concreting operation. They shall

— be fixed robustly enough to ensure that they will keep their prescribed position during placing and concreting within the tolerances,

— not introduce unacceptable actions on the structure,

— not react harmfully with the concrete, the reinforcement or prestressing steel,

— not produce unacceptable surface blemishes,

— not impair functional ability and durability of the structural member,

— not prevent adequate placing and compaction of the fresh concrete.

Components that are to be cast-in shall be inspected and approved while access for an appropriate inspection is available.

Embedded components that receive heat treatment after concreting shall be inspected for damages caused by the heat treatment and thermal expansion such as warping of the embedment; concrete spalling or cracking shall be rectified.

NOTE In cases where extensive welding is performed on embedments that are intended for transfer of significant shear forces, special provisions can be necessary to ensure that the concrete in areas important for the transfer of the shear forces has not had a significant reduction in concrete strength due to the heat input.

## 8.5 Geometrical tolerances

### 8.5.1 General

The following subclauses define the types of geometrical deviations relevant to offshore structures. Indicative values for normal tolerances are given for certain parameters. For other dimensions, only the type of requirement is indicated; numerical values are left to be specified in the project work specifications.

In general, tolerances on dimensions are specified in order to ensure that

- geometry is such as to allow parts to fit together as intended,
- geometrical parameters used in design are satisfactorily accurate,
- construction work is performed with a satisfactorily accurate workmanship,
- weights are sufficiently accurate for weight control and floating stability considerations (see ISO 19901-5).

All these factors shall be considered when tolerances are specified. Tolerances assumed in design and tolerances specified for construction shall be in agreement.

The requirements relate to the completed structure. Where components are incorporated in a structure, any intermediate checking of such components shall be subordinate to the final checking of the completed structure.

Changes in dimensions following temperature effects, concrete shrinkage, creep, post-tensioning and application of actions, including those resulting from different construction sequences, are not part of the construction tolerances. When deemed important, these changes shall be considered separately.

### 8.5.2 Reference system

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.

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 of ISO 4463-1. 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.

### 8.5.3 Tolerances of structural members

Requirements shall be given for the following types 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 skirts;

— 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 heights.

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 and vertical.

### 8.5.4 Cross-sectional tolerances

Requirements shall be given for the following type of tolerances.

a) Thickness:

- individual measured points, $\Delta t$ –10/+30 mm;
- overall average for area, $\Delta t$ –10/+20 mm.

b) Reinforcement position:

- tolerance on concrete cover, –10/+10 mm;
- tolerance on distance between rebar layers in the same face, –5/+10 mm;
- tolerance on distance between rebar layers at opposite faces, –20/+40 mm;
- tolerances on spacing of rebars in same layer;
- tolerances on laplengths, $(L)$ $L_{\text{min}} > 0{,}95L$.

c) Prestressing:

- tolerance on position of prestressing anchors;
- position of ducts/straightness at anchors;
- position of ducts in intermediate positions, $0{,}05t < 20$ mm;
- tolerances on radius for curved parts of tendons, $\Delta R < 0{,}05R$.

### 8.5.5 Embedments and penetrations

Tolerances shall be for items individually or for groups, as appropriate. Requirements shall be given for the following type of tolerances as relevant.

a) Embedment plates:

- deviation in plane parallel to concrete surface;
- deviation in plane normal to concrete surface;
- rotation in plane of plate.

b) Attachments to embedments: deviation from intended position (in global or local system).

c) Penetrations:

- deviation of sleeves from intended position of centre;
- deviation of sleeves from intended direction;
- deviation of manholes from intended position and dimension;
- deviations of block-outs from intended position and dimensions.

## 8.6 Quality control — Inspection, testing and corrected actions

### 8.6.1 General

Supervision and inspection shall ensure that the works are completed in accordance with this International Standard and the requirements of the project work specification.

### 8.6.2 Inspection classes

In order to differentiate the requirements for inspection according to the type and use of the structure, this International Standard defines three inspection classes:

IC 1 simplified inspection;

IC 2 normal inspection;

IC 3 extended inspection.

The inspection class to be used shall be stated in the project work specification.

Inspection class may refer to the complete structure, to certain members of the structure or to certain operations of execution.

Unless otherwise stated in the project work specification, IC 3, “extended inspection”, applies. IC 1, “simplified inspection”, shall not be used for concrete works of structural importance.

### 8.6.3 Inspection of materials and products

The inspection of the properties of the materials and products to be used in the works shall be in accordance with Table 9.

**Table 9 — Inspection of materials and products**

| Subject | Inspection class | | |
|---|---|---|---|
| | IC 1 (Simplified) | IC 2 (Normal) | IC 3 (Extended) |
| Materials for formwork | Not required | In accordance with project work specification | |
| Reinforcing steel | In accordance with the relevant parts of ISO 6935 and relevant national standards | | |
| Prestressing steel | N/A | In accordance with the relevant parts of ISO 6934 | |
| Fresh concrete: ready-mixed or site-mixed | In accordance with this International Standard | | |
| Other items [a] | In accordance with project work specification | | |
| Precast elements | In accordance with this International Standard | | |
| Inspection report | Not required | Required | |

[a] Could be items such as embedded steel components, etc.

#### 8.6.4 Inspection of execution

##### 8.6.4.1 General

Inspection of execution according to this document shall be performed in accordance with Table 10 unless otherwise stated in the project work specification.

**Table 10 — Inspection of execution**

| Subject | Inspection class | | |
|---|---|---|---|
| | IC 1 (Simplified) | IC 2 (Normal) | IC 3 (Extended) |
| **Scaffolding, formwork and falsework** | Random checking | Major scaffolding and formwork shall be inspected before concreting. | All scaffolding and formwork shall be inspected before concreting. |
| **Ordinary reinforcement** | Random checking | Major reinforcement shall be inspected before concreting. | All reinforcement shall be inspected before concreting. |
| **Prestressing reinforcement** | N/A | All prestressing components shall be inspected before concreting, threading, stressing.<br>Materials are to be identified by appropriate documentation. | |
| **Embedded items** | According to project work specification | | |
| **Erection of precast elements** | N/A | Prior to and at completion of erection | 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 | |
| **As-built geometry** | N/A | According to project work specification | |
| **Documentation of inspection** | N/A | Required | |

##### 8.6.4.2 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 wires 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, and
- openings and blockouts.

The structure shall be checked after formwork removal to ensure that temporary inserts have been removed.

#### 8.6.4.3 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,
- the cover is in accordance with the specifications,
- reinforcement is not contaminated by oil, grease, paint or other deleterious substances,
- reinforcement is properly tied and secured against displacement during concreting, and
- space between bars is sufficient to place and compact the concrete.

After concreting, the starter bars at construction joints shall be checked to ensure that they are correctly located.

#### 8.6.4.4 Inspection of prestressing works

Before casting operations start, inspections shall verify that

- the position of the tendons, sheaths, vents, drains, anchorages and couplers is in accordance with the design drawings, including the concrete cover and the spacing of tendons,
- the tendons and sheaths are securely fixed, also against buoyancy, and that the stability of their supports is ensured,
- the sheaths, vents, drains, anchorages, couplers and their sealing are tight and undamaged,
- the tendons, anchorages and/or couplers are not corroded, and
- the sheaths, anchorages and couplers are clean.

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 ensure that

- the results of preparation/qualification tests for grout are as required,
- the results of any trial grouting on representative ducts are as required;
- ducts are open for grout through their full length and free of harmful materials, e.g. water and ice;
- vents are prepared and identified;
- materials are batched and sufficient to allow for overflow.

During grouting, the inspection shall include checking of

- the 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,
- the order of blowing and washing operations,
- the precautions to keep ducts clear,
- the order of grouting operations,
- the actions in the event of incidents and harmful climatic conditions, and
- the location and details of any re-injection.

#### 8.6.4.5 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 11.

The inspection class for concreting operations shall be IC 3, extended inspection, unless otherwise specified in the project work specification.

Different structural parts in a structure may be allocated to different inspection classes, depending on the complexity and the importance in the completed structure.

**Table 11 — Requirements for planning, inspection and documentation**

| Subject | Inspection class | | |
|---|---|---|---|
| | **IC 1 (Simplified )** | **IC 2 (Normal )** | **IC 3 (Extended )** |
| **Planning of inspection programme** | N/A | Inspection plan, procedures and work instructions<br>Actions in the event of a non-conformity | |
| **Inspection** | Random | Frequent but random inspection | Continuous inspection of each casting |
| **Documentation** | Records from all inspections<br>All non-conformities and corrective action reports | All planning documents<br>Records from all inspections<br>All non-conformities and corrective action reports | |

#### 8.6.4.6 Inspection of precast concrete elements

Where precast concrete elements are used, inspection shall include

- visual inspection of the concrete element at arrival on site,
- delivery documentation,
- conditions of the concrete element prior to installation,
- conditions at the place of installation, e.g. supports, and
- conditions of the concrete element, any protruding rebars, connection details, position of the concrete element etc., prior to jointing and execution of other completion works.

#### 8.6.5 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:

- the implications of the non-conformity for the execution and the work procedures being applied;
- the implications of the non-conformity for the structure with regard to its safety and functional ability;
- the measures necessary to make the member acceptable;
- the necessity of rejection and replacement of non-conforming members.

Documentation of the procedure and materials to be used shall be approved before repair or corrections are made.

## 9 Foundation design

### 9.1 Introduction

Foundation design applies to the geotechnical aspects of the interface between structural elements and the seabed soils or fill placed on the sea floor, which provides support to the structure and its associated appurtenances. The mechanisms for transfer of forces from the structure to the supporting soil shall be verified and the need for grouting of the voids between the structure and the sea floor assessed.

NOTE Foundations for the structures considered herein are commonly referred to as “gravity foundations” and typically comprise raft, strip or pad footings with or without an underlying grid of skirts to key the foundation into the seabed and allow grouting of the void.

### 9.2 General

The following principal elements and considerations involved in the design of foundations for fixed concrete offshore structures are addressed in this clause:

- soil investigation (9.3);
- selection of representative soil properties (9.4);
- partial factors (9.5);
- design principles (9.6);
- stability analyses (9.7);
- soil-structure interaction (9.8);
- installation and removal (9.9);
- scour (9.10).

This clause should be read in conjunction with ISO 19901-4.

NOTE This International Standard supplements and amplifies many of the topics covered in ISO 19901-4, which addresses, among other things, model and prototype testing and instrumentation and monitoring.

## 9.3 Soil investigation

### 9.3.1 Purpose of the investigation

The purpose of a soil investigation is to ascertain the soil conditions and to obtain the geotechnical data necessary for the determination of representative material properties.

A soil investigation shall be performed utilizing the skills of professional personnel in the fields of geology, geophysics, seismology and geotechnics. It should include evaluation of all relevant existing data.

Potential geohazards such as submarine slope instability, liquefaction, faulting, shallow gas and subsidence shall be considered and investigated as necessary.

### 9.3.2 Site investigation

A site investigation shall cover the area and the soil layers of significance to the structure. Particular attention shall be given to determining the disposition and properties of shallow soils, as relatively thin layers of weak soils near the sea floor can have a significant influence on foundation stability.

### 9.3.3 Laboratory investigation

A laboratory programme shall provide appropriate test data to enable relevant analyses to be performed. The laboratory programme should comprise, *inter alia*,

— classification, consolidation and permeability tests,

— static and cyclic soil testing for the determination of strength, deformation and pore pressure generation parameters in the relevant stress and strain ranges, and

— stiffness and damping parameters of the foundation for calculating the dynamic behaviour of the structure.

## 9.4 Representative soil properties

Representative strength and deformation parameters shall be determined for all soil layers and for the relevant stress ranges. In these evaluations, caution should be exercised in the utilization of strength that depends on dilatancy of the foundation soils.

The representative parameters shall be appropriate to the issues to which they are applied. Limitations of the validity of the parameters, or conditions for their application, shall be clearly stated.

The effect of cyclic stresses induced by environmental actions on the structure (e.g. due to hydrodynamic, seismic and ice actions), which are transferred to the foundation soils, shall be determined when such effects are relevant to the particular issues under consideration.

## 9.5 Partial factors for actions and materials

### 9.5.1 General

The safety level in geotechnical engineering depends on the extent and reliability of the basic data, their interpretation, the analysis method and the monitoring and maintenance procedures. The choice of partial action and material factors is only one of several factors influencing foundation safety.

### 9.5.2 Partial factors for actions

Requirements regarding partial factors for actions and the combination of actions into design situations are given in 6.4 and 6.5. In geotechnical analyses, combinations of actions shall be selected such as to give the most unfavourable result for each of the stability mechanisms and deformation analyses performed.

### 9.5.3 Partial factors for materials

The material factor for soil may be expressed as the ratio of the undrained shear strength to the shear stress mobilized for equilibrium, or as the ratio of the tangent of the representative friction angle to the tangent of the friction angle mobilized for equilibrium.

The material factor should not be lower than 1,25. It may be modified, however, with regard to the consequences of failure, the failure mechanism and the way in which the representative strength of the soil material was determined and expressed. Consideration should also be given to what is recognized practice for the calculation procedures applied and the stability mechanisms analysed.

The material factor should be increased in the case of new types of structure and/or soil conditions with which no previous experience has been gained.

In the serviceability, fatigue and accidental limit states, the material factor shall be 1,0.

## 9.6 Geotechnical design principles

### 9.6.1 General

The design shall consider equilibrium in the limit states between actions and resistances using the method of partial factors.

The sensitivity of the calculated results should be analysed, making due allowance for reasonable variation of the assumptions made in the models that are used for the calculations, as well as for variations in material properties and static and cyclic actions.

Probabilistic calculations may be applied when it is considered appropriate.

Transfer of forces between the structure and the adjacent soil shall be analysed with respect to all limit states and all phases in the design service life of the structure, including the removal of the structure when such removal is required.

In the analysis of the interaction between soil and structure, consideration shall be given to whether the stiffness of the structure should be taken into account. The analysis should be based on representative deformation properties of the soil.

All significant effects from environmental actions shall be considered, including the stability of adjacent sea floor slopes. Structures located in the vicinity of each other should be considered with respect to possible reciprocal or unilateral effects, where relevant.

The drainage condition of the soils should be the most representative with respect to soil permeability, length of drainage path, load rate, etc. In case of uncertainty, the most unfavorable condition should be used.

### 9.6.2 Dynamic analysis for action effects

In some circumstances, it can be appropriate that analyses are based on a quasi-static simplification of structural response to environmental actions and soil resistance. Dynamic analyses can be necessary, however, for certain types of structure where the frequency content of the action is such that foundation design cannot be treated in a quasi-static manner under environmental or earthquake actions. The results can be sensitive to variations of stiffness and damping properties of the soil and the structure. A probable range of variation of soil properties should therefore form the basis of such analyses.

Soil parameters and calculation methods shall be selected in accordance with the histories of actions included in the analysis of the structure and the stress and strain levels resulting from the calculations.

Earthquake response shall be calculated on the basis of dynamic properties of all significant soil layers. The effect of interaction between soil and structure shall be taken into account.

### 9.6.3 Serviceability limit states

Settlements, differential settlements and horizontal displacements of the structure, and their development over a period of time, should be calculated. The calculated settlements and displacements shall not exceed specified limits to be established in advance for each structure and its equipment.

In predicting the evolution of settlements with time, the drainage conditions and uncertainties related thereto shall be considered, including possible assumptions regarding watertight structural components and draining soil layers.

The effect of cyclic actions on the deformation properties of the soil shall be taken into account.

The possibility and the consequences of subsidence of the seabed due to consolidation of the subsoil and depletion of the producing reservoir, where applicable, should be considered.

The possibility of corrosion, erosion or other deterioration of foundation elements and soil during the design service life shall be considered.

The consequences of seepage or compression of shallow gas should be considered where relevant.

### 9.6.4 Fatigue limit state

Fatigue analyses shall be based on unfactored representative soil properties, with due regard for the effects of cyclic degradation of soil stiffness and strength.

### 9.6.5 Ultimate limit states

The design resistance of the soil under and around the foundation shall be demonstrated to be sufficient to resist the action effects caused by the design actions. The development of the design resistance over a period of time shall be assessed.

Displacements of the foundation due to design actions should be calculated using representative deformation parameters. These displacements shall be considered when checking the ultimate limit state of the structure or of vital appurtenances, such as conductors, casings and risers.

The effects of cyclic actions on the generation and accumulation of pore pressures and deformations in the soil and the consequential potential reduction in shear strength shall be analysed. The combined effects of cyclic actions during several storms and subsequent consolidation over a period of time should be considered.

### 9.6.6 Accidental limit state

The foundation, as well as any skirts or other structural components for transfer of forces to the soil, shall be designed so as to prevent either local yield in structural parts or the soil spreading, leading to collapse.

In earthquake analyses for the ALE limit state, a range of probable values of all soil properties shall be used, reflecting the uncertainty of the soil conditions.

## 9.7 Bearing and sliding stability

Bearing and sliding stability analyses shall take the equilibrium of kinematically admissible modes of displacement and deformation of the soil into account.

All potential rupture surfaces in the soil mass shall be investigated, with special consideration given to the influence of weak layers or zones.

In a drained condition of the soil, the horizontal and vertical actions for limit equilibrium methods shall be assumed to be acting on the effective foundation area only.

In an undrained condition, the actions may be assumed to be distributed over all, or a greater part of, the total foundation area. In these cases, there should be documentation to show that this stress distribution is possible and will not lead to new forms of failure with a lower safety level.

The shape and location of the critical failure surfaces and critical shear zones shall be determined. The shape and location of these surfaces or zones depend on the action effects for the design situation, the soil stratification and the applied calculation model. The soil stress condition along the critical shear zones should be calculated.

Wave pressures acting on the seabed around the structure shall be included in the analyses.

The potential for destabilization of the foundation due to liquefaction of seabed soils in earthquakes or during drilling activities (e.g. for conductor installation) shall be assessed.

## 9.8 Soil reactions on structures

The forces or stresses from soil reactions acting on all foundation elements which either bear upon, or penetrate into, the seabed shall be calculated. The foundation system and the structures shall be designed for these reactions.

Representative actions and soil properties shall be used in determining the distribution of soil reactions. The reactions should then be considered as representative actions that are multiplied by the partial factors for action for those combinations that give the most unfavourable results with respect to the various limit states. Reasonable alternative distributions of soil reactions, which follow from the uncertainties in the calculation models used and the properties of the soil and the foundation system, shall be assessed.

Different distributions of soil reactions can govern the design of different parts of the foundation and structure.

Account shall be taken of the potential extent and distribution of partial contact between the sea floor and the base of the structure due to sea floor irregularity and/or the structural form.

The potential for uplift pressures arising under the base as a result of partial contact between the base and the sea floor shall be considered.

The potential for drag-down (negative skin friction) acting on foundation skirts and/or conductors shall be assessed. Drag-down on foundation skirts can arise from differential settlement between the foundation and the surrounding consolidating soil. Drag-down on conductors can arise from soil settlement under the imposed weight of the structure relative to the conductor.

Installation effects, changes in soil properties over a period of time, changes in properties of grout below the foundation, as well as local effects of pipes or other structures carried through the foundation or placed in the ground, shall be taken into account.

The analysis of the ultimate limit state of the structure shall include soil reactions distributed according to the assumptions made in the calculation of bearing capacity.

## 9.9 Installation and removal

### 9.9.1 Seabed preparation

The seabed shall be prepared to receive the platform, and this may include the following, as appropriate:

— removal of obstacles, debris, boulders etc.;

— dredging and removal of unsuitable materials;

— levelling the sea floor, i.e. by placement of a rock/gravel/sand bed of suitable materials.

### 9.9.2 Installation

Where the design requires that dowels, skirts or ribs shall penetrate into the seabed during installation, it shall be demonstrated that the planned penetration can be achieved, and that the maximum moment due to uneven penetration resistance is less than the available ballasting moment.

The potential for "piping" of enclosed water under the tip of the skirts during penetration shall be considered when planning the rate of penetration and the skirt evacuation system.

The soil resistance against skirts or other penetrating components shall be analysed with respect to its maximum and minimum value. These values shall be used in the evaluation of the installation process.

The consequences of heave of the seabed resulting from soil displaced during skirt installation should be considered.

The structure shall be demonstrated to be stable during touch-down as well as before and during any foundation grouting.

The stability during installation shall be calculated on the basis of planned progress of the operations and the expected setting time of grouts, if used.

Where an underpressure is required during installation, the geotechnical and hydraulic stability of the foundation soils should be analysed.

### 9.9.3 Removal

Where removal of the structure is anticipated, an analysis shall be made of the likely upper bound forces on the underbase and on the skirts to ensure that removal can be achieved with the means available.

In the calculation of the extraction forces, the effects of soil adhering to the foundation shall be considered.

Where an overpressure is used under the base, the geotechnical and hydraulic stability of the foundation soils should be analysed.

## 9.10 Scour

The possibility of sea floor sediment transport shall be considered, as well as the effects of the structure in modifying the sediment transport regime.

Where there is a risk of scour occurring around the foundation, precautions shall be taken based on one of the following principles:

— the foundation is designed to tolerate the erosion of material from the sea floor;

— adequate scour protection is placed around the structure during installation;

— the foundation is regularly observed, and scour resistant materials are immediately placed if unacceptable scour occurs (the adoption of this procedure pre-supposes that a critical extent of scour cannot develop during a single storm).

# 10 Mechanical systems

## 10.1 Introduction

Mechanical systems contained within a fixed concrete offshore structure can generally be classified as temporary or permanent. Temporary systems are those required during construction afloat, tows and installation at the offshore site. Permanent systems are those required during the operational life and removal of the structure. Some portions of permanent systems can be used in temporary systems. Examples of temporary systems are grouting, skirt air/water evacuation, air cushion, temporary and construction ballast water. Permanent systems are defined in the project requirements and can include such systems as crude oil storage and export, sea water supply and return, service water, fire water, vents, drains, ballast water, risers, J-tubes, conductors, shale chutes, foundation and structure condition monitoring, cathodic protection monitoring, dropped object protection and removal of the structure.

The provision of access, a suitable working environment and the required level of safety for personnel can result in the need for other systems and facilities such as heating, ventilation and air conditioning (HVAC), stairs, ladders, elevator, active and passive fire protection, fire and gas detection, safety shower and eyewash, communications and lighting.

The support of both permanent and temporary systems generally requires the provision of decks (frequently of steel) within the structure; these systems are generally located in the utility shaft and other shafts where present.

The support of decks, ancillary structures and piping within the fixed concrete offshore structure generally requires the use of steel plates embedded in the concrete of the structure (embedment plates). The embedment plates can be held in place by a passive (anchor bolts) or an active (cables/bars) system.

Concrete penetrations to allow the passage of piping should be used if necessary and can be accomplished by the use of cast-in sleeves, cast-in spools, block-outs subsequently completed with cast-in spool or sleeve, or holes cast into the concrete. The effect of service temperatures shall be considered in the design of piping and its supports.

Fixed concrete offshore structures can be used to house or support storage facilities for LNG; the storage facility and the mechanical systems to operate it are normally a separate package and are not covered by this document. All interfaces and loading scenarios from the operations and possible accidental events shall be considered in the design of the structure.

Mechanical systems used during construction and during marine operations shall be able to operate under the relevant conditions, including wave action and tilt under accidental conditions.

For structures installed in cold climates it shall be ensured that the mechanical systems shall be protected from freezing that can cause damage or prevent them from being operable as required.

## 10.2 Permanent mechanical systems

### 10.2.1 General

The permanent systems to be included shall be defined in the project work specification. Permanent mechanical systems for the operational life of the structure and for its removal at the end of its operational life shall have a design life as defined in the project work specification.

Permanent mechanical systems are generally designed in accordance with the same specifications as for topsides or to specifications providing a similar level of integrity and safety. The design, sizing and construction of permanent mechanical systems shall take into account that access and the ability to perform maintenance or repairs is limited or can even be infeasible in some cases. Full capacity testing of many systems is also not practical until the structure is installed and in operation at the offshore site.

National standards shall be complied with in all relevant disciplines, such as pressure vessels, piping, electrical, access, elevators, lifting devices, fire protection, noise levels, escape routes and emergency lighting, communications and area classification.

A hazard and operability analysis (HAZOP) shall be performed on all hydrocarbon systems and on all systems affecting the safety and operability of the structure and/or the topsides. The HAZOP for the structure shall be carried out, where relevant, in conjunction with topsides/subsea systems HAZOPs.

### 10.2.2 Crude oil storage system

#### 10.2.2.1 General

Where applicable, the crude oil storage system can be either dry (see 10.2.2.2) or wet (see 10.2.2.3). Only stabilized crude oils shall be stored in a fixed concrete offshore structure. Means to avoid problems when storing waxy crudes shall be considered.

For any crude oil (or other fluid) stream from the topsides to storage compartments, the location and elevation of any control valve(s) should be determined such as to prevent vacuum or damaging cavitations/flashing conditions at the outlet(s) of the valve(s).

#### 10.2.2.2 Dry storage

The dry storage system is a system whereby the crude oil is stored within the structure (e.g. in the caisson), with a vapour space above the crude oil. For the dry-type storage, the vapour space shall be filled with inert gas before any hydrocarbon is introduced and thereafter maintained filled with inert gas as the crude oil level rises or falls.

Sizing of the inert gas supply and venting system shall ensure that crude oil filling and offloading operations do not cause pressures in any portion of the system to fall below atmospheric pressure or the minimum design pressure of any component within the system.

Guidance should be taken from a standard such as API 2000 [4] for the supply of inert gas and the venting of the inert gas/crude oil vapours due to oil movement.

The minimum inert gas supply capacity should be 1,1 times the maximum crude oil offloading rate, while use of API 2000 [4] would result in a minimum vent capacity of 2,15 times the maximum crude oil filling capacity for crude oils with flash point below 38 °C. Supply and vent capacities are expressed as actual volumes at system temperature and pressure.

The inert gas system shall be equipped with pressure, control and relief devices, as required, to safeguard against under- and over-pressurization of the oil storage compartments. Materials in the gaseous mixture shall be selected such as to be resistant in the potentially sour/reducing atmosphere due to hydrogen sulphide.

In the same manner, protection devices shall be provided to prevent overfilling and over-pressurization of the storage compartments by an oil column in the feed piping.

The crude oil export system shall be able to pump out the oil fully from the storage space. Auxiliary pumps, slopes in storage bottoms, increase of inert gas pressure, flushing systems with water, etc. may be considered for this purpose. The possibility of accumulation of an oil/water emulsion (sludge) and water at the bottom of the storage should be taken into consideration.

For all glass-fibre reinforced plastic piping systems, if used in dry storage, consideration shall be paid to risk of static discharge; therefore, the appropriate type of glass-fibre reinforced epoxy pipes shall be specified.

#### 10.2.2.3 Wet storage

The wet storage system is a system whereby the crude oil is stored on top of sea water provided by the permanent ballast water system. The storage compartments are always full of liquid and the oil/water interface varies in elevation.

The wet storage system may be under-pressurized, balanced or over-pressurized with respect to the sea water pressure. All the approaches have their respective merits and are acceptable provided that

— actions due to pressurization of the storage system are recognized in the design of the structure, including the actions arising from accidents, maloperation, faulty control system, erroneous interface level indication, etc., and

— the risk of crude oil escape to sea in the lifetime of the structure meets the project requirements.

The wet storage system shall be provided with a venting system to remove potential gas accumulation at the top of the storage if the storage compartment is not self-venting through the crude oil piping.

Consideration shall be given in the design to the possibility of a sludge being formed at the oil/water interface and the need to remove this substance periodically by means of the crude oil piping system.

For a storage system that is spread over several compartments, greater operational flexibility can be achieved by designing it in such a way that one compartment can be decommissioned while the others are kept in operation.

Where the storage is split into several independent compartments, each compartment shall have its own independent level-measurement system. The system shall be provided with alarms to stop further filling/emptying of crude oil when the interface is close to the top or bottom of the compartment during normal operations.

NOTE The system that has predominantly been selected for North Sea structures is based on thermal conductivity sensors located at multiple elevations within a rod passing vertically through the cell. Ultrasonic systems have also been used.

A back-up level-measurement system, which can be based on pressure or differential pressure measurements, should be considered where maintenance of the primary system is impractical.

The crude oil distribution piping in multi-compartment storage should be designed to ensure that the oil is distributed evenly during operation; for this purpose, pressure drops in each distribution line of the crude oil and ballast water systems should be similar.

Fluid velocities in crude oil and permanent ballast water distribution piping should be limited to avoid imbalance, vibration and erosion effects in these inaccessible lines. Crude oil piping within storage compartments should terminate in such a manner as to minimize the disturbance of the stored liquids.

Experience has shown that standing waves several metres in height can occur at the oil/water interface due to disturbances in the inlet/outlet flows and/or deformations of the structure from environmental actions and the relatively small difference in mass densities. High and low operational interface levels should take this possibility into account. Diffusers providing a maximum crude and ballast water exit/inlet velocity of 0,1 m/s have been used successfully.

The piping design pressures and ballast water tank volume of crude oil and permanent ballast water systems should be checked for the “breathing” effect of the structure (e.g. variations of storage volumes due to deformation of the structure under extreme waves).

Where one storage compartment is composed of several intercommunicating cells, the openings providing the intercommunication shall cover the full range of the interface level, and shall be sized to limit the interface level difference between two cells to an acceptable value when the compartment is filled or emptied at full rate. Partial blockage of such openings from wax build-up or sludge shall be considered in the sizing.

### 10.2.3 Other storage systems

The structure may provide compartments for the storage of auxiliary fluids such as brine, drill water, fresh water, diesel oil, methanol and glycol. Except for diesel oil, these systems are normally of the dry storage type. Vapour space can be air filled for non-flammable fluids. Design considerations are generally the same as for crude oil storage.

### 10.2.4 Permanent ballast water system

A permanent ballast water system is designed to keep the wet storage compartments of the structure filled with liquid using sea water and can need to be designed to cater for the differing requirements of several interfacing systems such as

— the ballast water associated with the wet crude oil system and the ballast water for any other wet storage compartments, and

— the ballast water for wet shafts or other compartments, which are designed to be dewatered during the operational life of the platform.

The ballast water system shall be designed so that the sea water can be sufficiently frequently renewed or treated to avoid the build-up of excessive $H_2S$ concentrations in stagnant sea water.

NOTE 1 $H_2S$ can be formed by the action of anaerobic bacteria feeding on organics, sulphates, sulphites, etc. in sea water, in solid ballast materials and in hydrocarbons. $H_2S$ can also come from the stored crude oil itself. $H_2S$ build-up in the ballast water associated with the crude oil storage can generally be limited to acceptable levels [e.g. mass fraction $< (10 \times 10^{-6})$] by ensuring regular displacement of ballast water from all compartments, followed by renewal with fresh sea water.

$H_2S$ formation in dead spaces can be mitigated by the following:

— renewal of the ballast water by natural convection (in the drill and riser shafts, top and bottom openings to the sea usually provide sufficient sea water circulation), where renewal to cold shafts (or cold compartments) may be provided by openings just above sea level so that waves bring new sea water in at regular intervals, circulation through the shaft being achieved via openings near the base of the shafts;

— forced circulation;

— injection of air or biocide (e.g. hypochlorite-dosed sea water).

The ballast water system associated with wet storage systems shall be designed so that ballast water returning to sea complies with environmental regulations.

NOTE 2 North Sea experience with fixed concrete offshore structures has shown that ballast water associated with crude oil storage with no treatment during normal operation can reach an oil content level less than a mass fraction of $10 \times 10^{-6}$.

The ballast water tank or the buffer cell (the cell used for storage systems kept in balance with sea water pressure) should be provided with facilities to allow any oil accumulated above the ballast water level to be removed.

### 10.2.5 Sea water systems

Sea water systems include sea water circulation (for cooling of shafts) and utility systems associated with topsides (sea water lift, service water, fire water, sea water return from topsides to the sea, etc.).

Permanent systems handling sea water generally should be made of materials with excellent fouling- and corrosion-resistant properties such as titanium, duplex stainless steel, 6 % Mo (molybdenum) austenitic stainless steel and glass-fibre reinforced plastic. Special circumstances such as piping embedded in concrete or piping with a short design life could justify the choice of carbon steel.

Piping for sea water systems crossing the caisson below sea water level is generally medium-to-large diameter, such that rupture within a dry compartment (e.g. utility shaft) would cause rapid flooding. For such piping a remotely operated and highly reliable isolating valve should be located at a minimum distance from the wall of the dry compartment.

NOTE It could be desirable to leave sufficient length of piping between the isolating valve and the wall to allow use of a freeze-plug for maintenance.

For large-diameter piping, inlets and outlets to the sea should be fitted with removable screens of the same material to avoid the entry of fish, ropes, umbilicals, hoses, etc. They should also be capable of being closed by doors or blind flanges operated by divers or ROV (remotely operated vehicle). Suitable structures around these inlets or outlets should be included to provide protection against snagging by ropes, chains, etc., and to provide assistance to divers or ROV

All systems that could, through leakage, result in flooding the utility shaft or another dry compartment, shall be analysed. In consideration of flooding scenarios, emphasis should be placed on the design to minimize the likelihood of such an event by selection of materials, dropped object protection, rapid detection and alerting of such an event, etc.

Sea water discharge lines originating below sea level within the utility shaft can be grouped together into a common line and routed to sea via an inverted U-loop (with siphon breaker as necessary) located above sea water level, to decrease the risk of flooding.

### 10.2.6 Drains, sumps and bilge

Drainage systems shall be provided, as required, to safely handle the draining of equipment and piping for operational and maintenance considerations, and to collect and dispose of water or other liquids released into the open, such as wash-down or fire water release.

A closed drainage system shall be designed to prevent the escape of flammable or harmful liquids, vapours and gases. The vapour space within tanks containing flammable vapours or gases shall be kept free of air, using a closed vent system with inert gas purge. Provision shall be made to safely handle the overflow of such tanks in the event that the pump-out facilities are unable to meet the demands.

### 10.2.7 Vents

Vents from tanks (sump, ballast, compartments, etc.) within the concrete structure which lead into vent piping on topsides could lead to topsides vapours and gases back-flowing into the piping of the structure. The design shall ensure that this does not create the possibility of flammable gas entering piping within the structure, by means of an inert gas purge within the structure or separation of the vent piping.

### 10.2.8 Safety systems

Safety systems for the protection of personnel and equipment in the concrete structure shall generally be similar to those included in topsides within hazardous and non-hazardous areas; for fire explosions, see ISO 13702 [6]. For the structure and, in particular, the shafts, special consideration shall be given to the consequences of

— lack of natural ventilation,

— the “mine shaft” nature of the structure’s shafts.

NOTE Safety systems commonly utilized in concrete structures include fire, gas and smoke detection, CCTV, active (water spray, deluge, foam, hose reels, $CO_2$) and passive fire protection, resistance to blast, personnel protection devices (clothing, breathing apparatus), fire extinguishers and blankets, and safety shower and eye wash.

### 10.2.9 Decks

Where plated decks are used in dry shafts, either fully gas-tight or with limited free drainage, they shall be provided with sufficient open, piped drainage to handle the greatest operational influx rate of liquid onto the deck (e.g. release of deluge fire water system). Failure panels or hatches may be utilized for abnormal flow rates (e.g. flooding event) to allow liquid to cascade from one deck to another without causing structural failure. The safety of personnel shall be considered in the selection of such devices. Grated decks may also be considered where gas tightness is not required.

### 10.2.10 Elevators

Elevators shall be provided to meet the requirements of the project work specification.

Special consideration shall be given to escape facilities from the elevator cab in the case of breakdown.

Optical sensors, for example, to prevent door closure, shall either be avoided or shall be provided with a manual local override such that elevator operation is not halted by smoke.

Although an elevator is not to be considered as a legitimate means of escape during a fire emergency (see also 10.2.14), it shall be recognized that there could be personnel travelling in the elevator at the time an emergency occurs or, in spite of warnings, that personnel could in any case still attempt to use the elevator as a means of rapid escape.

### 10.2.11 Lifting devices

Hoists, trolleys, runway beams, lifting lugs, drop-out areas, access hatches, etc. provided for maintenance activities shall be included, as necessary, to allow maintenance to be performed in a safe manner.

### 10.2.12 Risers and J-tubes

The routing of risers and J-tubes shall be based on an overall platform safety study and evaluation of risk. The following issues shall be considered:

- location outside, inside or embedded within the concrete structure;
- type of fluid within riser (stabilized crude, sea water, gas);
- type of fluid within flowline pulled into J-tube (well stream, sea water, gas, etc.); J-tubes may also be used for umbilicals to control subsea completions;
- pressure and temperature of the fluid;
- diameter, length, pipe schedule and inventory of riser or flowline;
- location of emergency shutdown valves (subsea external to the structure, within the structure, within the topsides);
- piping within the structure: all-welded or with flanges;
- consequences of fire (pool fire, jet fire) and explosion resulting from leakage or riser rupture.

For multishaft structures, drill shafts are normally located at the “process end” of topsides, while the utility shaft is located at the living quarters end. In such configurations, risers and J-tubes containing flammable fluids are located within the drill shafts and the riser shaft. If the riser shaft is also located at the living quarters end, it should generally not be used for high-pressure gas and well fluids.

Where risers and/or J-tubes are routed on the outside of the caisson and/or shafts, special consideration shall be given to dropped objects protection and boat collision. For J-tubes, consideration shall be given to the consequences of flowline failure within the J-tube.

### 10.2.13 Conductors and shale chutes

Metal sleeves located in the base of the concrete structure, used to guide the penetration of the conductors through the base, shall generally extend to a height above the base sufficient to allow each conductor to be raised and the cutting units of an underreamer to be deployed beneath the conductor in order to drill out the temporary concrete plug.

The design of the structure shall consider the need to cut conductors below the base of the structure in the event that removal of the structure is a project requirement.

The location of shale chute discharges shall be based on a consideration of the need to prevent their effluent entering sea water intakes and forming mounds on parts of the caisson, subsea pipelines, etc. not designed for such actions or requiring visual inspection.

### 10.2.14 Access

Access ways for personnel shall meet the requirements of the project work specification. At least two access/escape routes shall be provided from each deck. An elevator, if present, shall not be considered as an escape route.

Vertical escape access ways (stairs and ladders) shall be enclosed in areas containing hydrocarbons in flanged piping or equipment, unless otherwise specified by the project work specification. In other areas, the need to provide one or more enclosed vertical escape routes shall be evaluated based on an evaluation of the risk, and possible severity, of fire and release of smoke or toxic gases. Smoke ingress into enclosures of escape ways shall be prevented, even if one door is left open, by suitable pressures and flows and/or intermediate doors. Stairs, and possibly elevators, shall be sized to allow evacuation of injured personnel by stretcher.

External valves, if any, should be provided with ROV access docking ports if appropriate.

### 10.2.15 HVAC

HVAC with supply and extract shall be required for all enclosed areas in which personnel are expected to work. The system design shall provide sufficient air changes per hour to meet project requirements and area classification for electrical equipment. Flow patterns within the enclosed areas shall consider both lighter- and heavier-than-air gases and vapours.

Passive fireproofing of ductwork, fire dampers, etc. shall be provided as required by the project work specification.

### 10.2.16 Structure and foundation condition monitoring

System requirements are given in Clause 14.

Cabling from the instrumentation needed for the structure and foundation condition monitoring system shall withstand the sea water hydrostatic pressure. Where cabling passes through a watertight concrete member, watertight penetrations shall be used.

Cabling running in areas open to personnel shall comply with requirements for flammability and release of smoke and toxic gases when subject to fire.

Penetrations for casings through watertight concrete members for such items as pore water pressure measurement, horizontal displacement or settlement of the structure, etc. shall be designed to the same criteria as for other penetrations (e.g. conductors).

### 10.2.17 External markings

Level markings below and above the still waterline shall be included in accordance with the project work specification. Markings shall be of a type recognizable when lit by submerged light (e.g. reflective). In a marine environment a non-fouling surface shall be provided. Markings on horizontal surfaces below water should be designed to remain visible (e.g. being elevated) during the deposition of solids (e.g. sand).

NOTE Markings attached to the structure in a similar manner to embedment plates have proven effective in the past.

### 10.2.18 Other

Other systems such as those for pore water pressure reduction (anti-liquefaction) for removal of the structure shall be designed to meet the project work specification.

## 10.3 Mechanical systems — Temporary

### 10.3.1 General

Systems frequently needed for temporary use include

- air cushion systems,
- construction and temporary ballast water systems,
- skirt evacuation systems,
- grouting systems, and
- systems for monitoring under-keel clearance (e.g. echo sounders), skirt penetration and seabed pressure on the base.

HAZOPs shall be carried out for all temporary systems which can affect the safety of the structure and shall cover all relevant stages of the afloat construction, tows and installation phases. Potential damage caused by dropped objects shall also be considered.

### 10.3.2 Air cushion system

Where an air cushion system is specified to reduce the draught for float-out from the dry dock, and possibly during a period at the deep water construction site, the grout system piping or the skirt evacuation system piping can generally be used. Differences in operating and design pressures shall then be considered in the design of the piping.

The air cushion system shall be sized with sufficient margin for the lift-off operation of the structure in the dry dock to be achieved within the time stated in the project work specification. The possibility of air leakage through maloperation, cracks in concrete, etc. shall be considered.

The air/water interface level can be established by differential pressure measurement, and the design should provide for level measurement of each separate compartment.

System design shall ensure that all air is vented from compartments at completion of use of the air cushion to avoid reduction of the structure's floating stability that would otherwise result.

### 10.3.3 Temporary ballasting/deballasting water system

The structure shall be ballasted during all afloat stages of construction and towing to the offshore site. Ballasting with sea water is required to control inclination and draught. The ballasting system can also be used for pumping out water that enters the structure by means of snow, rain, construction activities, leakage, etc.

The sizing of the temporary ballasting water system shall meet the project work specification with respect to marine operations such as lift-off in the dry dock, deck mating, installation at the offshore site, construction requirements and accidental leakage rates.

It is generally found necessary to design and install more than one system to cover the changing situation during construction when floating construction stage(s) is/are used. The system first utilized can be extended and modified to meet changing needs during construction. The following issues shall be considered in the design.

a) During construction, water within the structure will be dirty and abrasive. Blockage or partial blockage of pump suction piping can occur. The inlets of pump suction piping should not be at low points; thus, a certain volume of water will remain under all conditions.

b) Piping taking suction from one compartment and running through other compartments creates a potential hazard if leakage occurs. Protection against pipe damage shall be considered and shall be provided to the extent necessary to reduce this risk to an acceptable level.

c) Piping that can be exposed to dropped objects or other mechanical damage should be fabricated from ductile materials, such as carbon steel, which will deform rather than fracture when subjected to impact loads. While carbon steel will corrode in water and sea water, this aspect is generally not a problem for the relatively short period of use, provided adequate wall thickness is specified.

d) Simple systems with portable pumps placed directly in compartments, and with all water added by a pumped system, require greater attention from operators, but are inherently safe. Their use, particularly during any initial stages of afloat construction, should be carefully considered.

More complex systems with remote, centralized pumps and controls reduce operational demands, but can create greater possibilities of maloperation leading to uncontrolled tilt or submergence of the structure. Such systems shall be provided with the necessary safeguards, alarms, interlocks, status indication, etc., to provide the high level of reliability and safety required. A minimum of two fail-safe actuated valves in series shall be included in any piping directly routed to sea (i.e. sea water inlet/outlet) which could result in flooding of the structure.

The most critical operations are

— transfer by differential head from one or more compartments to one or more other compartments via manifolded piping, and

— introduction of sea water into one or more compartments by the differential head of the draught of the structure and ballast water level in the compartments.

In establishing the requirements for system safeguards and reliability, the consequences of maloperation shall be considered. Single compartment damage stability and the structural ability of walls to withstand hydrostatic differential ballast water levels shall be considered in the design. The ballasted areas shall be divided into compartments such that flow of ballast water will not cause uncontrolled tilting.

Depending on the consequences of failure, the design of instrumentation associated with temporary ballasting operations should take into account the need for redundancy provided by readily accessible local indicators (water level tubes, gauges, etc.). The effects of structure tilt on levels shall be considered.

Particularly for large-diameter piping, water hammer resulting from rapidly closing valves shall also be considered, as shall the possibility of vibrations caused by control valves, orifices, etc.

The design of temporary systems shall take into account the need to fill the temporary piping running from the utility shaft with grout after the temporary use is completed, to prevent the possibility of leakage of liquid from one compartment to another resulting from corrosion of the piping.

The design of temporary ballast water systems shall take into account the possibility of components such as motors, instrumentation, controls, junction boxes and cables being sprayed or inundated by water.

### 10.3.4 Grouting and skirt evacuation systems

#### 10.3.4.1 Grouting system

Where required to meet foundation and/or structure design requirements, a grouting system shall be designed to fill void spaces between the sea floor and the base of the structure. The system design shall consider the need to displace sea water with grout from the void, without incurring unacceptable dilution from this displacement operation.

NOTE The void space beneath drill shafts is generally left ungrouted.

The number of grout lines and their sizing shall take into account the need to achieve full or partial completion of the grouting operation when this is necessary for the structure in order to resist the environmental actions that could develop during the grouting period.

A means to monitor the grout filling operation shall be provided so that verification of void space filling is obtained. Measurement of injected quantities, differential and/or absolute pressure measurements, facilities to permit visual checks, etc. may be used.

Grouting piping design shall include flushing facilities to allow displacement of grout in cases where grout injection is halted for unexpected reasons, such as adverse weather preventing transfer from a grout vessel, mechanical failure, etc. The flushing facilities shall be designed to overcome the resistance of grout that has gelled within the piping.

#### 10.3.4.2 Skirt evacuation system

The skirt evacuation system is used for

— evacuation of sea water trapped within skirts as they penetrate the seabed during installation of the structure at the offshore site, and

— evacuation of the sea water displaced by grout during the grouting operation.

NOTE The system can also be used as part of an air cushion system.

Sizing of the skirt evacuation piping system shall take into account the need to prevent piping below the skirt tips during penetration by limiting the pressure drop through this piping system. The design shall be based on the maximum rate of penetration, considering the possibility of uneven penetration resulting from seabed characteristics, bathymetry and tilt of the structure.

The skirt evacuation system may be designed to create an underpressure below the base of the structure to provide an additional driving force for skirt penetration. Such designs shall take into account the possibility of reverse piping below the skirt tips.

#### 10.3.4.3 Grouting and skirt evacuation piping

Grouting and skirt evacuation piping systems design shall consider the risks of plugging of, or obstruction to, piping outlets and inlets resulting from the structure's installation. Seabed disturbance can result from skidding of the skirts across the seabed and from soil heave during penetration. The possibility of unplanned flow from one void space to another (around a skirt) shall be considered.

Special care shall be taken with the design of piping as it cannot usually be flanged/plugged below the structure's base and is thus pressurized with sea water during afloat phases of construction. As one or both of these systems will enter the utility shaft, the piping within the utility shaft up to the first block valve will be under pressure. If jumper hoses are to be used below sea level within the utility shaft, non-return valves should be included in the design unless it is demonstrated that a rupture of a hose can be handled without flooding.

Where this piping is of small diameter, it can frequently be embedded in structural concrete members as protection, until it exits into, for example, the utility shaft. If this is not the case and the piping is run in the open inside the structure, then special precautions shall be taken to provide protection from dropped objects.

Where run inside concrete members, the design of those members shall consider the design pressure of the piping and the possibility of leakage.

Where grout and/or skirt evacuation piping is brought inside the utility or other dry shaft, it should be fitted with double valving located as close as possible to the entry point (e.g. within 1 m). Dropped object protection shall also be considered.

While grout piping is generally routed to a dry area within the structure, skirt evacuation piping may be run to the same location, or to the outside of the structure. The location of skirt evacuation outlets on the outside of the structure shall consider the need to prevent sea water intakes, external risers, J-tubes, etc., being exposed to grout during offshore installation of the structure.

#### 10.3.5 Instrumentation for tow and installation of the structure

At installation, under-keel clearance assumes particular significance. It is also possible that bottom clearance can be of concern along certain parts of the tow route to the offshore site.

Suitable instrumentation for measuring under-keel clearance includes echo sounders mounted at the base of the structure, either beneath it or along the perimeter. Consideration should be given to provision of three or four units. This number of units will also provide some redundancy in case of failure while the structure is under construction.

Side scan sonar, for use during the tow to the offshore site, mounted at the base of the structure, can also be useful.

Echo sounders do not always provide sufficiently accurate measurements of under-keel clearance for the final stages of touchdown (less than 2 m clearance). Bottom contact instrumentation utilizing a lever arm with contact pad can be considered. Earth pressure transducers can also provide confirmation of touchdown.

Instrumentation shall be installed to measure penetration depth and inclination in order to assure that the installation criteria are met.

Special care shall be taken with all such instrumentation, together with its cabling, to ensure operation after submergence in sea water. Meticulous functional testing, mechanical completion and protection against subsequent damage are essential.

#### 10.3.6 Other systems

Other systems, such as caisson pressurization (for deep submergence) and the structure docking template, shall be designed to meet the project requirements and relevant clauses of this International Standard.

### 10.4 Attachments and penetrations

#### 10.4.1 Attachments

Attachments to the concrete structure can generally be fixed by embedment plates cast into the concrete. The transfer of actions from the embedment plate into the concrete shall be considered in the design of the concrete members. The mechanism for transfer of transverse shear, moments and pull-out forces at the surface of the embedment plate into concrete shall be carefully developed using shear keys and headed anchor bolts.

#### 10.4.2 Penetrations

Penetrations through concrete members are generally needed for electrical and instrument cables, piping, conductors, risers, J-tubes and foundation condition monitoring instrumentation.

The size required for the penetration and the actions on the concrete member from the mechanical item passing through the penetration shall be considered in the design of the structural member. In the case of sleeved penetrations, the sleeve itself may be used to contribute to the strength of the structural member.

In cases where a watertight seal is required, proven solutions include the use of either a cast-in spool with water stop flange(s) or a cast-in sleeve with end plates welded to both the sleeve and the pipe. Glass-fibre reinforced plastic (GRP) pipe passing through a sleeve depends upon effective grouting of the annulus for a watertight seal. In such cases, a finish should be specified for the external surface of the GRP pipe to be located within the sleeve for that will form an effective bond with the chosen grout.

Deformations of the structure shall be considered in the design of such piping, with attention being given to points where piping embedded in protective concrete enters into structural members.

Penetrations for well conductors and, penetrations for instrumentation for measurement of long-term settlement etc. through watertight concrete members which are temporarily plugged with concrete or grout and drilled through after platform installation are not able to be reinforced within the plug. Shear keys should be provided on the metal sleeve surrounding the plug. The design of the sleeve, keys and plug for conductors should not profit from the weight of the conductor.

#### 10.4.3 Welding

Welding to embedment plates and sleeves shall be performed using welding procedures developed to minimize distortion of the plate and sleeve. Following welding, the interface between plate or sleeve and concrete shall be inspected to verify that spalling, etc. has not reduced the concrete cover of reinforcement.

#### 10.4.4 Corrosion protection

Any subsea connection between dissimilar metals, within or outside the structure, can give rise to galvanic corrosion, and shall be checked for this possibility.

Particular care shall be taken at pipe penetrations where the spool, sleeve and/or pipe are made from a noble metal (titanium, austenitic stainless steel, etc.).

At connections of dissimilar metals, coatings may be used to reduce the area of the cathode (i.e. the more noble metal). Coating of the metal acting as anode (i.e. the less noble metal) near the connection shall be avoided, because the unfavourable area ratio will cause rapid pitting below coating defects (pinholes). Consideration shall also be given to the possibility of reinforcement/prestressing steel acting as an anode to more noble metal penetrations and attachments.

### 10.5 Mechanical systems — Special considerations

#### 10.5.1 Design, installation and testing of piping

The successful completion of piping systems during construction is a critical issue that requires careful design, installation and testing procedures.

Piping which is embedded in concrete (e.g. for grouting, air cushion, skirt evacuation) cannot be examined externally or repaired after encasement. Special attention shall be paid to guarding against installation defects that could allow the penetration of fresh concrete. The piping and its support shall also be sufficiently robust to remain unaffected by concrete placement and vibration. Flushing, pressure testing and positive identification of terminations (inlets/outlets) and routing, with subsequent tagging, performed prior to encasement, shall be performed to ensure a sound final installation. Further flushing immediately following the placement of concrete around the embedded piping should also be considered.

Piping systems operated during construction are exposed to construction dirt and debris, are subject to the possible freezing of water during cold weather and can be subject to damage from construction activities.

Loss or malfunctioning of parts of these systems can have severe consequences for the integrity of the structure through, for example, incomplete grouting as a result of blocked piping jeopardizing foundation stability or the leakage of water into the structure at uncontrolled flow rates.

#### 10.5.2 Design of pipe supports

Special care should be taken with the design of pipe supports as piping can be subjected to forces arising from the deformation of concrete structural members.

Piping commissioned and operated during construction phases shall be designed to accommodate forces from the deformations of structural members supporting the piping. Deformations generally result from changes in the actions on the structure as mass is added, draught is increased, prestressing is applied, water ballast or air cushion are added or removed, environmental temperatures (sea water, air) change, etc.

Piping crossing walls shall be designed to accommodate differential movement of the walls and other components restricting piping movement.

The design of piping encased, or partly encased, in concrete shall consider the structural deformations relative to the various components. Special care shall be taken at interface zones between structural and non-structural concrete as large shear stresses can result from differential displacements at the interface, particularly during deep submergence, mating and installation.

Buoyancy forces acting on piping submerged in ballast water or unset concrete shall be considered.

#### 10.5.3 Design of steel structures

Special consideration shall be given to the design of steel decks supporting rotating equipment. Both the deck itself and the structures supporting the suction and discharge piping shall be designed to prevent vibration of the piping initiated from the rotating equipment.

NOTE Maximum deflection criterion of 1:1 000 has been successfully used on North Sea concrete structures.

#### 10.5.4 Design of equipment

Pressure vessels, tanks and rotating equipment (pumps) shall be designed according to International Standards or national standards where available, and, when not available, to an internationally recognized recommended practice such as is published by the American Petroleum Institute (API).

#### 10.5.5 Dropped object protection

Dropped object protection is required for a number of temporary and permanent systems. Consideration shall be given to the consequences of damage from dropped objects during construction and operation. During construction, the greatest concern is the danger of flooding and, possibly, loss of the structure from dropped objects within the structure rupturing piping operating ballast water, sea water, etc. Rupture of such piping can allow flooding from the sea or flooding of one compartment from another.

Permanent systems within shafts and on the outside of the structure shall be reviewed for the possibility and consequences of dropped objects, and protection shall be provided as deemed necessary. Special consideration should be given to risers, J-tubes and sea water inlets.

## 11 Marine operations and construction afloat

### 11.1 General

The overall objective of this clause is to ensure that the marine operations are performed within defined and recognized safety/confidence levels. The design principles are meant to be applicable on a worldwide basis. However, additional standards, codes and guidelines, in the area in which the marine operations will be performed, shall be taken into account. ISO 19901-6[5]), applies to the marine operations of concrete offshore structures as well, and shall be applied as appropriate.

This clause describes the design principles of marine operations from a point of view of engineering, planning, implementation and documentation for fixed concrete offshore structures. Alternative provisions, methods and requirements can fulfil the intention of this clause and can be applied, provided they can be documented to demonstrate at least the same level of confidence.

Marine operations are non-routine operations of limited duration. Marine operations are normally related to temporary phases of load transfer, transportation, installation and/or securing and removal of units at sea, and include all the transient movements that should be performed during the construction afloat, outfitting, towage and installation of the platform.

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5) To be published.

### 11.2 Engineering and planning

The engineering of marine operations should encompass both the design and analysis of the components, systems or means required for the performance of such operations, as well as the methods ("procedures") developed to achieve them safely. It shall be ensured that all equipment will function and that all activities can be safely performed which are necessary as part of the planned procedure in emergency and accidental situations.

The following shall be considered:

— that the required equipment, vessels and other means are designed and checked for adequate performance with respect to their intended use;

— that adequate redundancy is provided in the equipment, vessels and other means to be used, to cover possible breakdown situations;

— that operating weather conditions, chosen at values smaller than the specified design criteria, are forecast for a period long enough to complete the operation;

— that the operations are planned, in nature and duration, such that accidental situations, breakdowns or delays have a very low probability of occurrence and are all covered by detailed contingency actions;

— that adequate documentation has been prepared for a safe step-by-step execution of the operation, with clear indications of the organization and adopted chain of command;

— that the operations are conducted by suitably experienced and qualified personnel.

## 12 Corrosion control

### 12.1 Introduction

#### 12.1.1 General

Fixed concrete offshore structures associated with production of oil and gas comprise permanent structural components in carbon steel (C-steel) that require corrosion protection, both in the topsides and in the shafts. In addition, shafts and caissons contain mechanical systems such as piping for topsides supply of sea water and for ballast, crude oil storage and export. These piping systems are exposed to corrosive environments both internally and externally. Risers 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 sea water is, however, not covered by this document.

Concrete rebars and prestressing tendons are adequately protected by the concrete itself, provided there is adequate concrete coverage and the type/quality of the concrete is suitable. However, rebar portions freely exposed to sea water in the case of concrete defects, and embedment plates, penetration sleeves and various supports which are freely exposed to sea water or the marine atmosphere, will normally require corrosion protection.

#### 12.1.2 Corrosion zones and environmental parameters affecting corrosivity

A fixed concrete offshore structure will encounter different types of marine corrosion environments. These may be divided into corrosion zones as given in Table 12.

**Table 12 — Corrosion zones**

| External zones | Internal zones |
|---|---|
| External atmospheric zone | Internal atmospheric zones |
| Splash zone | Intermediate zones |
| External submerged zone | Internal submerged zones |
| Buried zone | |

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 sea water during tidal changes and dampened wave action, or during movement of crude oil/ballast water interface levels. 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.

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.

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 can be either to enhance or retard corrosion attack. In the buried and internal submerged zones (i.e. sea water-flooded compartments), oxygen in the sea water 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 can retard corrosion, corrosive metabolites from bacteria can offer an alternative corrosion mechanism.

Corrosion governed by biological activity (mostly bacteria) is referred to as MIC (microbiologically induced corrosion). For most external surfaces exposed in the submerged and buried zones, as well as internal surfaces of piping for sea water and ballast water, corrosion is primarily related to MIC.

### 12.1.3 Forms of corrosion and associated corrosion rates

Corrosion damage to uncoated carbon steel in the atmospheric zone and in the splash/intermediate zones associated with oxygen attack is typically more or less uniform. In the splash zone and the most corrosive conditions for the external atmospheric zone (i.e. high time-of-wetness and high ambient temperature), corrosion rates can amount to 0,3 mm per year, and for internally heated surfaces in the splash zone even higher (up to on 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 carbon steel (i.e. as uniform attack) is normally around 0,1 mm per year or lower. In the submerged and buried zones, corrosion is mostly governed by MIC causing colonies of corrosion pits. Welds are often preferentially attacked. Corrosion as uniform attack is unlikely to significantly exceed about 0,1 mm per year, but the rate of pitting can 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).

In most cases, the static strength of large structural parts is not jeopardized by MIC due to its localized form. The same applies to the strength of pressure-containing piping systems. However, MIC can readily cause leakage in piping by penetrating pits, or initiate fatigue cracking of structural areas subject to cyclic actions.

Galvanic interaction (i.e. metallic plus electrolytic coupling) of carbon steel to e.g. stainless steel or copper-base alloys can enhance the corrosion rates given above. 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.

Very high-strength steels ($f_{yk}$ > 1 200 MPa) and certain high-strength aluminium, nickel and copper alloys are sensitive to stress corrosion cracking in marine atmospheres. If susceptible materials are to be used, cracking should be prevented by use of suitable coatings.

## 12.2 Design for corrosion control

### 12.2.1 General

In a marine environment, the following main measures are used for corrosion control:

- coatings and linings;
- cathodic protection;
- corrosion-resistant materials;
- corrosion allowance.

### 12.2.2 Criteria for design of corrosion control

The initial selection and subsequent detail design of systems for corrosion control should take into account the following main factors:

- regulatory requirements;
- criticality of the overall system and functional requirements to individual components to be protected;
- type and severity of corrosion environment(s);
- design life (and likelihood of lifetime extension);
- accessibility for inspection, maintenance and replacements, including the overall maintenance philosophy;
- suitability, reliability and economy of optional techniques for corrosion control.

### 12.2.3 Coatings and linings

The use of coatings ($<$ 1 mm) and linings ($\geqslant$ 1 mm) is the primary technique for corrosion control in the atmospheric and splash/intermediate zones. Coatings are organic or metallic layers, single or multiple, applied by spraying, brushing or dipping. Linings are layers for corrosion control, mostly in combination with a function for mechanical protection, heat insulation or fire protection. Organic materials for linings are often reinforced (e.g. glass-fibre or flakes). Coating systems include various forms of organic (paint) and certain metallic coatings. Of the latter, zinc layers are applied by hot dipping or thermal spraying. Thermally sprayed aluminium coatings have been used more recently, particularly for more demanding applications.

In the submerged zones, coatings are sometimes applied to reduce the current demand for cathodic protection and to improve the distribution of the cathodic protection current.

The selection of coating and lining systems should be based on proven experience for a specific application and environment. The design shall be in accordance with an International Standard or national standard, if available. Comprehensive field-testing may be required when practical experience is lacking. In the submerged and splash/intermediate zones, coatings and linings shall be selected to ensure compatibility with cathodic protection, specifically in relation to shielding of cathodic protection current, and resistance to cathodic disbonding.

The design of all components intended to be paint coated should take into account relevant measures to ease both the initial application and later maintenance. This may include preference for tubular shapes, rounding of sharp edges, provisions for securing scaffolding, etc. Structural components exposed to sea spray, rain or intermediate wetting internally should be designed to prevent accumulation of moisture, e.g. by using continuous welding and making provisions for drainage.

#### 12.2.4 Cathodic protection

##### 12.2.4.1 General

Cathodic protection can be effected using galvanic (“sacrificial”) anodes, or impressed current from one or more rectifiers. Galvanic anodes are normally preferred for offshore structures due to high reliability and simple design. Impressed current systems are vulnerable to mechanical damage of the anodes and associated cables, particularly in the external submerged zone. They are, in general, less tolerant of shortcomings during design, construction and operation, and require a dependable current source. Impressed current systems have been used for both initial design and upgrading of existing cathodic protection systems based on galvanic anodes.

The possibility of hydrogen-induced stress cracking (HISC) associated with cathodic protection implies that steels with specified minimum yield strength in excess of 720 MPa should not normally be used for critical components without special considerations. This imposes some restrictions on the use of materials for high strength components that receive cathodic protection, e.g. bolting. Furthermore, any welding (or other methods of execution affecting tensile properties and hardness) should be carried out according to a qualified procedure that limits hardness to HV350. This requirement restricts the use of welded structural steels to about 550 MPa.

Galvanic anode systems utilize anodes based on aluminium or zinc. Aluminium is normally preferred for sea water applications, but zinc is sometimes considered more suitable for compartments where anodes could become affected by crude oil.

In the submerged and buried zones, cathodic protection will prevent corrosion damage to rebars and other embedded components in the case of insufficient coverage, poor aggregate quality or other concrete defects. In the splash/intermediate and atmospheric zones, impressed current cathodic protection may be applied to protect rebars damaged by corrosion.

##### 12.2.4.2 Design

The design shall ensure a protection potential within the range –0,80 V to –1,1 V relative to Ag/AgCl/sea water. More negative potentials can be achieved by impressed current systems but can be harmful to coatings, and even to ordinary structural steels due to HISC.

With adequate design, cathodic protection will prevent any form of corrosion damage to external surfaces of components in the submerged and buried zones. This includes prevention of galvanic corrosion due to coupling of dissimilar materials and corrosion in narrow crevices such as in the annulus of piping passing through sleeves in a concrete wall. In the intermediate zones, cathodic protection will only be partly effective. Cathodic protection is possibly not fully reliable in crude oil storage tanks where anodes are intermittently wetted by oil and water. Deposition of drill cuttings in shafts and placing of solid ballast within the structure can also affect the reliability of cathodic protection. Hence, a corrosion allowance could have to be applied in addition, and corrosion-resistant materials should be considered for critical components.

Internal compartments such as shafts and caissons shall be designed to be self-sufficient with cathodic protection. All metallic components within a compartment shall be integrated in the design. The design of cathodic protection for drill shafts shall include any future conductors to be installed during the planned drilling programme.

Cathodic protection in the external submerged zone shall include all external components to receive cathodic protection (risers and riser clamps, embedment plates, etc.) The design shall be based on the estimated surface areas exposed to sea water or sediments. In addition, the design of both external and internal cathodic protection systems shall always allow for current drain to concrete rebars in the buried, submerged and splash/intermediate zones, and externally also to any steel skirts or other exposed surfaces which are not deemed to actually require cathodic protection.

For large structures, it is always convenient, and often necessary, to subdivide the structure into units to be protected. The division may be based on, e.g. depth zones. Each internal compartment shall comprise at least one unit for individual design.

Design of impressed current cathodic protection systems should include extra current capacity (i.e. compared to the calculated current demand) to compensate for a more uneven current distribution from the relatively few, high-output impressed current anodes. Furthermore, redundancy should be included to compensate for some deficiency of individual anodes and rectifiers. The design should include detailed procedures for maintenance (replacement) of anodes and other equipment. Impressed current systems shall have a structure-to-sea water potential monitoring system that is able to verify that cathodic protection is maintained within specified limits at areas closest to, and remotest from, the anodes.

#### 12.2.4.3 Current demand

The current demand ($I_\text{c}$) for cathodic protection of external surfaces in the submerged and buried zones, and for wetted surfaces in internal compartments shall be calculated from

$$I_\text{c} = A_\text{c} f_\text{c}\, i_\text{c}$$

where

$i_\text{c}$ is the design current density;

$f_\text{c}$ is the coating break-down factor for any coated surface ($f_\text{c} = 1$ for bare steel);

$A_\text{c}$ is the actual surface area to be protected.

$I_\text{c}$ shall be calculated as the average current demand $I_\text{c,average}$ to maintain cathodic protection throughout the design life of the system. The initial and final current demands, i.e. $I_\text{c,initial}$ and $I_\text{c,final}$ respectively, required to polarize the relevant surfaces to a protection potential of –0,80 V relative to Ag/AgCl/sea water, shall also be calculated.

Design current densities and coating breakdown factors for calculations of average and initial/final current demands shall be in accordance with a recognized standard.

The total current demands for one unit shall include a calculation of current drain to concrete rebars in the submerged, splash/intermediate and buried zones. Design current densities for calculation of current drain shall be in accordance with a recognized standard.

#### 12.2.4.4 Anode calculation

The design life of cathodic protection systems shall normally be equal to the design life of the structure itself. If the design is based on replacement of anodes, provisions to facilitate retrofitting should be addressed during the design.

Based on the average total current demand for each unit (including current drain), the total net anode mass, $m_\text{T}$, in kilograms (kg) required to maintain cathodic protection throughout the design life, should be calculated from:

$$m_\text{T} = \frac{8760\, t_\text{f}\, I_\text{c,average}}{u \varepsilon}$$

where

$t_f$ is the design life of the CP system, in years;

$I_{c,average}$ is the average current demand, in amperes (A);

$\varepsilon$ is the anode material's electrochemical efficiency, expressed in ampere-hours per kilogram (Ah/kg);

$u$ is the anode's utilization factor.

Design values for $\varepsilon$ and $u$ shall be in accordance with a recognized standard.

From the required total net anode mass $m_T$, a tentative selection of anode dimensions and number of anodes shall be made. It shall subsequently be demonstrated that this selection meets the requirements for initial/final current output $I_a$ (A) and current total capacity $C_a$ (Ah).

The anode current output $I_a$ shall be calculated from Ohm's law according to:

$$I_a = \frac{E^o{}_c - E^o{}_a}{R_a}$$

where

$E°_a$ is the design closed-circuit anode potential, expressed in volts (V);

$E°_c$ is the design protective potential, expressed in volts, matching the initial/final design current densities (i.e. normally –0,80 V relative to Ag/AgCl/sea water);

$R_a$ is the anode resistance, expressed in ohms (Ω).

$E°_a$, $E°_c$ and $R_a$ shall be in accordance with a recognized standard.

The current capacity $c_a$ of an anode is given by:

$$c_a = m_A \varepsilon u$$

where $m_A$ is the net mass per anode in kilograms. The total current capacity $C_a$ thus becomes:

$$C_a = n c_a$$

where $n$ is the number of anodes.

Anode dimensions and net weight shall be selected to match all requirements for current output (initial/final) and current capacity for a specific number of anodes. Calculations shall be carried out to demonstrate that the following requirements are met:

$$C_a = n c_a \geqslant 8\,760\ t_f\ I_{c,average}$$

$$n I_{a,initial} \geqslant I_{c,initial}$$

$$n I_{a,final} \geqslant I_{c,final}$$

The final current output shall be calculated based on the estimated anode resistance when the anode has been consumed to its utilization factor ($u$).

The calculated number of anodes shall be distributed to provide a uniform current distribution, taking into account the current demand of individual surface areas and current drain.

The distribution and fastening of anodes shall ensure adequate electrical continuity to all items that are to receive cathodic protection. This can require dedicated cable connections. Electrical continuity via the rebar systems cannot be relied upon to be fully reliable, although as a conservative approach, full electrical continuity is assumed for current drain to concrete embedded components.

The anodes should preferably be located in the submerged zones, rather than in the buried zone, even if dedicated to protection of buried components. The design of anodes to be located on the outside of the structure should be such as to minimize the potential for mechanical damage, such as snagging of chains, ropes, and umbilicals associated with marine operations.

During any construction afloat, insufficiently protected (or unprotected), temporarily installed steel structures or vessels moored to the structure can cause enhanced consumption of external anodes located below the sea water line (see 12.3.3).

#### 12.2.5 Corrosion-resistant materials

The selection of corrosion-resistant materials for structural components and mechanical equipment shall take into account their anticipated corrosion resistance for the intended application (including resistance to environmentally induced cracking) and compatibility with other materials, mechanical properties and fabricability. In the submerged and buried zones, galvanic corrosion of external surfaces can be efficiently prevented by cathodic protection. However, in the atmospheric/intermediate zones, and internally in piping systems, special precautions can be required to prevent galvanic corrosion. This can include coating of the component with the highest electrochemical potential (i.e. the more “noble” material is to be coated and the less noble component is to be left uncoated) or electric insulation.

In general, the selection of materials and systems for corrosion prevention shall take into account that, for certain items, particularly those located in buried or internal submerged zones, inspection and repairs can be essentially impossible. Hence, materials with intrinsic resistance to corrosion should be selected for certain critical components.

NOTE Cu-base alloys, and Fe- and Ni-base alloys with minimum 17 % Cr plus minimum 2,0 % Mo, are normally fully corrosion-resistant at ambient temperatures in the atmospheric, splash and intermediate zones. For sea water piping systems, austenitic stainless steels and Ni-base alloys with minimum 20 % Cr and 6 % Mo, duplex (ferrite-austenitic) with minimum 25 % Cr and minimum 4 % Mo and, furthermore, titanium are used as piping material for sea water without any practical limitations associated with the maximum allowable flow rate. In the absence of cathodic protection, incipient crevice corrosion has been experienced for these stainless materials but can be prevented by overlay welding of critical surfaces with Alloy 625. Titanium is considered immune to corrosion by sea water. Certain copper alloys are also used for sea water piping but are liable to erosion-corrosion even at moderate flow rates in case of local turbulence, while pitting attack due to MIC can occur at stagnant conditions. Glass-fibre reinforced plastic is another candidate material for sea water piping. The use of carbon steel for such piping will normally require a philosophy for maintenance and replacement, unless the design life is short.

#### 12.2.6 Corrosion allowance

A corrosion allowance, i.e. extra steel thickness to compensate for the effect of metal loss by corrosion, is sometimes used to maintain the required structural capacity. A corrosion allowance will also serve to extend the time for a local penetration associated with pitting attack, causing leakage of fluid. For carbon steel piping systems that are to be exposed to sea water internally/externally for a limited period of time during construction or installation, a corrosion allowance of three millimetres can be adequate.

### 12.3 Fabrication and installation of systems for corrosion control

#### 12.3.1 General

Fabrication procedures can affect the corrosion resistance of materials, in particular for certain corrosion-resistant materials. All fabrication activities involving welding or brazing to structural components or mechanical equipment shall be carried out according to regulatory requirements, applicable codes/standards and approved project-specific procedures and drawings.

#### 12.3.2 Coatings and linings

Coatings and linings shall be applied according to a procedure describing surface preparation, handling of coating/lining materials, application, inspection and repairs. Recommendations for preparation and review of specifications should be given in accordance with a recognized standard.

Quality control shall be performed during surface preparation, coating application and repairs to ensure consistent performance of coatings and linings.

#### 12.3.3 Cathodic protection

Manufacturing and quality control of galvanic anodes should be carried out according to a procedure that defines compositional limits of anode and anode core materials, weight and dimensional tolerances, visual inspection and permissible surface defects, marking and documentation. Short-term electrochemical testing may be specified to verify electrochemical performance on a heat basis.

During construction and outfitting of concrete structures afloat, unprotected submerged temporary structures and vessels can cause an excessive current draw on external galvanic anodes. Where applicable, this should be compensated for by installing temporary extra capacity, e.g. by suspended galvanic anodes or impressed current cathodic protection.

Commissioning of impressed current cathodic protection systems should include detailed structure-to-sea water potential measurements to verify readings from fixed reference electrodes and to confirm an adequate protective potential range for the components to be protected.

#### 12.3.4 Corrosion-resistant materials

Fabrication of corrosion-resistant materials should be carried out with due consideration of how the applicable techniques (e.g. welding, grinding) affect their corrosion resistance and mechanical properties. As an example, improper consumables and insufficient gas shielding can destroy the corrosion resistance of sea water piping in high-alloy stainless steel. When temporarily exposed to sea water during construction or installation and without efficient cathodic protection, ordinary stainless steels such as AISI 316 can suffer severe corrosion damage within a period of weeks.

# 13 Topsides interface design

## 13.1 Introduction

The design of topsides structures is governed by ISO 19901-3 [5].The interface design, as it affects the concrete structure, shall be in accordance with this International Standard.

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

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

Where topsides and concrete structure construction are separate contracts, the owner shall define the interface responsibility. This shall at least clarify who is responsible for input to and from the topsides engineering contractor as part of a technical co-ordination procedure.

## 13.2 Basis for design

As part of establishing and maintaining adequate handling of the topsides/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, paying particular attention to consistency of coordinate systems adopted, and ensure that data are provided for all required limit states and relevant stages in the lifetime of the structure such as

- installation/mating of the topsides,
- transportation and installation of the structure,
- the operating phase, and
- decommissioning.

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

The structural analysis of the concrete structure can consider the topsides to varying levels of detail and sophistication, depending on its effect on the design of different structural parts. Typically, the design of the upper parts of the concrete structure (shaft) is based on finite element analysis comprising also the topsides' stiffness matrix. It is required that the stiffness of the topsides and the action effects imposed by the topsides are represented in sufficient detail to ensure an adequate distribution of action effects between topsides and substructure, as well as within the concrete structure.

The documentation to be provided as a basis for proper interface design shall also cover

- shaft configuration,
- top of shaft layout,
- deck elevation,
- actions to be applied on the top of the concrete offshore structure from the topsides (i.e. topsides weights for design purposes including centre of gravity, etc.);
- tolerances (i.e. for concrete geometry, tie bolts, tendons, bearing tubes, embedment plates, etc.), and
- deck mating tolerances to allow for deformations during load transfer.

## 13.3 Deck/shaft structural connection

Several alternatives are viable for the structural connection between the topsides and the concrete structure. The detailing shall consider initial contact and ensure force distribution as presumed in structural analysis and design.

The design of the intersection between topsides structural components and top(s) of shaft(s) shall take due account of shear forces (friction check) arising from tilt in temporary phases or accelerations of the structure in the operational phase. A compression check on the grout shall be performed. The possibility of uplift shall also be accounted for.

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

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

### 13.4 Topsides — Structure mating

As the selection of the installation method affects both the concrete structure and topsides design, it shall be ensured that such consequences are addressed at an early stage.

Typical items and effects that shall 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 concrete structure for a lift installation of topsides, and
- design of installation aids for both lift and float-in installations.

### 13.5 Transportation, tow-to-field

Accelerations and tilting angles in intact and damaged conditions shall be accurately defined, and the consequences for design of topsides, structure and their connection shall be dealt with.

## 14 Inspection and condition monitoring

### 14.1 General

This clause specifies requirements and recommendations for inspection and condition monitoring of concrete offshore structures and indicates how these requirements and recommendations can be achieved. Alternative methods can also fulfil the intent of these requirements and can be applied, provided they can be demonstrated and documented to provide the same level of safety and confidence.

It describes how inspection and condition monitoring can be planned, implemented and documented for fixed concrete offshore structures.

### 14.2 Objective

The inspection and 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 and condition monitoring activities is to ensure that the structure is suitable for its intended purpose throughout its design service life.

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 due to 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 maintenance and/or repair works.

## 14.3 Personnel qualifications

Personnel involved in inspection planning and condition monitoring, as well as in assessment of the findings, shall have relevant competence with respect to design of marine concrete structures, concrete materials technology, execution of concrete structures, as well as specific experience in the application of inspection techniques and the use of inspection instrumentation and equipment. Owing to the fact that each offshore structure is unique, inspectors shall familiarize themselves with the primary design and operational aspects before conducting an inspection.

Inspectors shall have adequate training appropriate for supervisors, divers or ROV-operators as specified in accordance with national requirements where applicable.

## 14.4 Planning

### 14.4.1 General

The planning of inspection and condition monitoring activities shall be based on

- the function of each structural member,
- the exposure to damage,
- the vulnerability to damage, and
- the accessibility for inspection.

### 14.4.2 Basis for planning of inspection and condition monitoring

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.

If values for actions, action effects, erosion or foundation behaviour are highly uncertain, the installation shall be equipped with instrumentation for measurement of environmental conditions, dynamic motions, strains, etc. to confirm the applicability of governing design assumptions. Significant changes to equipment and storage/ballast operations should be identified and recorded.

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. repair) or abandonment. The structure should also be monitored to detect minor damage and defects, which can develop to a critical situation.

### 14.4.3 Programme for inspection and condition monitoring

The first programme for inspection and condition monitoring shall provide an initial assessment of the condition of the structure, as described in 14.4.4.2, i.e. the assessment should have an extent and duration which, as far as possible, provide a total description of the condition of the structure (design verification). The programme for inspection 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.

#### 14.4.4 Inspection and condition monitoring intervals

##### 14.4.4.1 General

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 the 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 assessing the safety of the personnel involved, with respect to defined acceptable risks and consequences of failure.

##### 14.4.4.2 Initial inspection and condition monitoring

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 parts.

In the operational phase, more information will become available and the knowledge about the initial condition can be updated.

##### 14.4.4.3 Periodic inspection and condition monitoring during operation

Inspection and condition monitoring of the structure shall be carried out regularly in accordance with the established programme for inspection and condition monitoring.

Assessment of the condition of the structure 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 14.5. 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 can indicate time-dependent deterioration processes.

##### 14.4.4.4 Special inspection and condition monitoring

Inspection and condition monitoring should be conducted after direct exposure to a design environmental event (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 events (boat collision, falling object, etc.).

In the event of change of use, extension of the design service life, 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 of the programme to the changed conditions and shall be subjected to modification as required. Risk to the environment shall be included.

##### 14.4.4.5 Inspection and assessment prior to removal

Based on a removal programme, an assessment of the structural integrity should be carried out prior to removal. The need for this assessment, and the extent of the assessment and inspection required will depend heavily on the period elapsed since the last periodic or special inspection. As a minimum, however, this assessment needs to consider safety of personnel.

### 14.5 Documentation

The efficiency and integrity of the inspection and condition monitoring activities are dependent on the validity, timeliness, extent and accuracy of the available inspection data.

To facilitate periodic inspection as specified in the programme for inspection and condition monitoring, the following documents/information shall be recorded:

— data from the design, construction and installation phase (summary report);

— basic information about each inspection performed (e.g. scope of work, important results, available reports and documentation).

Up-to-date inspection summaries shall be retained by the owner/operator. Such records shall describe the following:

— the tools/techniques employed;

— the actual scope of work (including any field changes);

— the inspection data collected including photographs, measurements, videotapes, etc.;

— the 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.

## 14.6 Important items related to inspection and condition monitoring

### 14.6.1 General

Inspection of concrete offshore structures normally includes a survey of the different parts of the structure, including the atmospheric zone, the splash and the tidal zones and the important areas of immersed concrete. It is generally recognized that the splash zone is the most vulnerable to corrosion.

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.

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.

It is also important to differentiate between the extent of assessment and frequency of inspection for different structural parts. The function of each structural member will play a role in establishing the extent and frequency of assessment. The exposure or vulnerability to damage of each member should be considered when establishing priorities for assessment. The accessibility for assessment can also be highly variable. The atmospheric zone provides the least difficult and the submerged zone the most difficult access. However, the splash zone usually provides the most severe environmental exposure and a greater likelihood of accidental impact for many concrete marine structures. Therefore, the condition monitoring plan shall consider the function of each structural part and provide further consideration of access and exposure of the part. Focusing on critical structural parts located in high exposure areas of the structure leads to efficiency in monitoring.

### 14.6.2 Atmospheric zone

Inspection and condition monitoring should focus on detecting possible damage or defects caused by

- structural design and construction imperfections,
- environmental actions,
- mechanical actions,
- static and dynamic operational actions,
- altered operational conditions,
- chloride ingress,
- geometric anomalies, such as construction joints, penetrations, embedments,
- subsidence,
- impacts.

Typical defects will be

- deformation/structural imperfections,
- cracks,
- corrosion of reinforcement,
- damaged coatings,
- freeze/thaw damage,
- spalling and delaminations,
- local impact damage.

### 14.6.3 Splash zone

In addition to the aspects listed for the atmospheric zone, the inspection and condition monitoring should focus on

- effects due to alternating wetting and drying of the surface, and
- marine growth.

### 14.6.4 Submerged zone

In addition to the aspects listed for the atmospheric and splash zones, the inspection and condition monitoring 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 cuttings or sediments if such build-up covers significant parts of the structure,

— movement in bottom sediments,

— mechanical outfitting,

— tension cable anchor points,

— debris,

— settlement, and

— the corrosion protection system (anodes).

#### 14.6.5 Internal

The inspection of the internal parts shall focus especially on

a) detecting any leakage,

b) biological activity,

c) temperature, composition of sea water and pH values in connection with oil storage,

d) detecting any corrosion of reinforcement, and

e) concrete cracking.

The presence of bacterial activity, such as sulfate-reducing bacteria (SRB), and pH shall be evaluated, considering the quality and thickness of the concrete cover. Necessary actions against the possible harmful effect of bacterial activity shall be evaluated.

#### 14.6.6 Concrete durability

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 that are important but are unlikely to change significantly with time, such as permeability and cover to reinforcement, and

— those 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 in petroleum storage areas.

Chloride profiles should be measured in order to establish the rate of chloride ingress through the concrete cover. Either total chloride ion content or water-soluble chloride content should be measured. However, the method chosen should be consistent throughout the life of the structure. These profiles can be used for estimating the time to initiation of corrosion attack of reinforcement in the structure.

#### 14.6.7 Corrosion protection

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.

In 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.

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.

Inspection of corrosion control based on use of corrosion-resistant materials can be integrated with visual inspection of the structural or mechanical components associated with such materials.

NOTE Experience with fixed concrete offshore structures constructed with materials as recommended in this document is very good, however 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 process, cell measurements of the electro-potential of the reinforcement can provide some indication of corrosion activity. These techniques are useful for detecting potential corrosion in and above the splash zone but have limited application under water because of the low resistance of sea water.

It has been established that under many circumstances underwater corrosion of the reinforcement does not lead to spalling or rust staining. The corrosion products are of a different form and can be washed away from cracks, leaving no evidence on the surface of the concrete of buried corrosion of the reinforcement. However, when the reinforcement is adequately cathodically protected any corrosion ought to be prevented. In cases where cathodic protection of the reinforcement can be limited, the absence of spalling and rust staining at cracks in the concrete cover ought not to be taken as evidence for no corrosion.

## 14.7 Inspection and condition monitoring types

### 14.7.1 General

The extent and choice of methods of inspection and condition monitoring can vary, depending on the location and function of the actual structure/structural part. In the choice of inspection methods due consideration shall be given to reducing the risk associated with the inspection activity itself. The main techniques for underwater use 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.

NOTE Concrete in sea water develops a surface layer, consisting mainly of aragonite and brucite that provides some protection to ingress of chlorides, etc. Cleaning of the surface can remove this protective layer and hence any cleaning needs to be done with caution.

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.

The following types of inspection shall be considered.

a) **Global visual inspection**

Global visual inspection is an examination of the total structure to detect obvious or extensive damage such as impact damage, wide cracks, settlements, tilting, etc. The inspection can be performed at a distance, without direct access to the inspected areas, for instance by use of binoculars. Prior cleaning of the 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 a 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 inspection item can be needed.

c) **Non-destructive inspection/testing**

Non-destructive inspection/testing is a close inspection by electrical, electrochemical or other method 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, used to detect hidden damage or to assess the mechanical strength or parameters influencing concrete durability.

e) **Instrumentation-based condition monitoring**

In areas with limited accessibility, or for monitoring of action effects, corrosion development, etc., additional information can be provided by the use of instrumentation-based condition monitoring. The instrumentation can be temporary or permanent. Sensors should preferably be fitted during fabrication. The sensors will be strain gauges, pressure sensors, accelerometers, corrosion probes, etc.

### 14.7.2 Structural monitoring and structural safety systems

The structure may be instrumented in order to record data relevant to pore pressure, earth pressure, settlements, subsidence, dynamic motions, strain, inclination, corrosion of reinforcement, temperature in oil storage, etc.

The data could be beneficial to the condition monitoring.

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 in case of storms, these monitoring systems shall be inspected regularly.

## 14.8 Marking

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, of the regions in which defects are most prone to occur and of parts of the structure expected to become, or known to 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.

## 14.9 Guidance for inspection of special areas

### 14.9.1 General concrete surface

Poor-quality concrete, or concrete containing construction imperfections, should be identified during the initial condition assessment and monitored for subsequent deterioration. Surface imperfections of particular importance include poorly consolidated concrete and rock pockets, spalls, delaminations and surface corrosion staining.

The emphasis for the monitoring will be to detect and monitor damage caused by overstressing, abrasion and environmental exposure.

Overstressing is often evidenced by cracking, spalling, concrete crushing and permanent distortion of structural members. Not all cracking is the result of structural overload. Some cracking can be the result of creep, restrained drying shrinkage, plastic drying shrinkage, finishing, thermal fluctuations and thermal gradients through the thickness of the member. 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.

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 deformations occur. Passive cracks are benign in that they do not increase in severity with time. Design codes and recommendations provide guidance on critical crack widths that signal concern for the ingress of chloride ions and the resulting corrosion of embedded reinforcing 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.

Concrete crushing, spalling and delamination also require careful determination of the cause. Crushing is generally associated with flexural overload, axial compression or impact. Spalling and delamination can be either load-related or caused by severe corrosion of embedded reinforced steel. The appropriate repair method for these distress types will vary considerably depending upon the actual distress cause.

NOTE A number of methods have been developed to measure the strength of concrete *in situ*. These include ultrasonic pulse velocity, gamma ray backscatter, impact hammers and the Windsor probe. They are well developed for the inspection of land-based concrete, but have limited experience for under water concrete.

### 14.9.2 Steel transition ring/concrete interface

This interface is the main load transfer point between steel topsides and the concrete structure, and should preferably be examined for structural integrity annually. The examination should include the load transfer mechanism (flexible joints, rubber bearings, bolts and cover) and the associated ring beam.

The concrete interface should be inspected for evidence of overstress and corrosion of embedded reinforcement steel. Corrosion-potential surveys can be used to detect ongoing corrosion that is not visible by visual inspection alone.

### 14.9.3 Construction joints

Construction joints in the concrete structure represent potential structural discontinuities. Water leakage and corrosion of reinforcement are possible negative effects. As a minimum, the monitoring programme should identify construction joints located in high stress areas and monitor the performance with respect to evidence of

a) leakage,

b) corrosion staining,

c) local spalling at joint faces, which indicates relative movement at the joint,

d) evidence of poorly placed and compacted concrete, such as rock pockets and delaminations,

e) joint cracking or separation.

### 14.9.4 Penetrations

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 particularly 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.

### 14.9.5 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.

### 14.9.6 Embedment plates

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 can lead to deterioration of the reinforcing steel, which is in contact with the embedments.

#### 14.9.7 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 is associated with areas that provide a permeable path through which salt water flow can take place. Continuous flow of saline and oxygenated water can cause corrosion of the reinforcement and washout of cementitious paste with an ensuing weakening effect of the reinforced concrete matrix. In such areas, adequate emphasis needs to be placed on the detection of local loss of reinforcement section due to chloride-induced (black) corrosion. Attention should be given to the surface and the perimeter of patched areas for evidence of shrinkage cracking and loss of bond to the parent concrete surface.

#### 14.9.8 Splash zone

The splash zone can experience damage from impact of supply vessels, etc. and can also deteriorate from ice formation with ensuing spalling in surface cavities where concrete has been poorly compacted.

Even where high-quality concrete has been 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 actions resulting from the movement of pack ice relative to the structure, the heavily abraded concrete surface can cause an increase in applied global lateral actions. Repairs to these surfaces should be made as soon as possible to prevent further deterioration and structural overload.

#### 14.9.9 Debris

Drill cuttings can build up on the cell tops and/or against the side of the structure and should be assessed for

— lateral pressures exerted by the cuttings, and

— whether they cause an obstruction to inspection.

Removal of drill cuttings needs to be assessed accordingly.

Debris can cause structural damage through impact, abrasion or by accelerating the depletion of cathodic protection systems. Also, it poses a danger to diving activities and precludes examination if allowed to accumulate. Particular vigil needs to be maintained for impact damage covered by debris.

#### 14.9.10 Scour

Scour is the loss of foundation-supporting soil material and can be induced by current acceleration round the base of the structure or by “pumping” effects caused by wave-induced dynamic rocking motion. It can lead to partial loss of base support and unfavourable redistribution of actions.

#### 14.9.11 Differential hydrostatic pressure (drawdown)

Structural damage or equipment failure can lead to ingress of water and affect the hydrostatic differential pressure. This can necessitate call for special inspection before and during drawdown.

#### 14.9.12 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 can be required.

#### 14.9.13 Sulfate-reducing bacteria (SRB)

SRBs occur in anaerobic conditions where organic material is present (such as hydrocarbons). The bacteria produce $H_2S$ (hydrogen sulfide) as their natural waste which, in large enough amounts, will cause a lowering of pH value of the cement paste in the concrete. Favourable conditions for SRB growth can be present in unaerated water in, for example, the water-filled portion of shafts and cells. An acidic environment can cause concrete softening and corrosion of reinforcement. An inspection of a concrete surface likely to be affected by SRB activity is difficult to undertake. Some guidance can be obtained by adequate monitoring of SRB activity and pH levels.

#### 14.9.14 Post-tensioning

Tendons are usually contained within ducts which are grouted. Inspection of tendons is therefore very difficult using conventional inspection techniques.

Post-tensioning anchorage zones are commonly areas of complex stress patterns. Because of this, considerable additional reinforcement steel is used to control cracking. In many cases, the reinforcing steel is very congested, and this condition can lead to poor compaction of concrete immediately adjacent to the anchorage. Also, the anchorages for the post-tensioning tendons are generally terminated in prestressing pockets in the structure, and the recess is fully grouted after tensioning and before launch.

Experience has 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. Where evidence exists for potential distress, a more detailed visual inspection supplemented by impact sounding for delaminations should be performed to determine if the anchorage is distressed. The visual inspection should focus on corrosion staining, cracking and large accumulations of efflorescence deposit.

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 recognized as local concrete cracking resulting from redistribution of stress and should be investigated upon discovery. Total loss of prestress can result in member collapse. Design documents should be reviewed to establish the arrangement and distribution of cracking that could be expected to result from partial loss of prestress. This information should be documented with the inspection records and made available to the inspection team.

## 15 Assessment of existing structures

### 15.1 General

This clause gives procedures for the assessment of existing fixed concrete offshore structures to demonstrate their fitness-for-purpose.

Assessment is an integral part of the evaluation phase of an inspection and monitoring programme. An assessment shall be undertaken if any of the initiators specified are triggered. An assessment shall consider all relevant available data, including the summary report on the structural design.

The owner shall maintain and demonstrate the fitness-for-purpose of the structure for its specific site conditions and operational requirements, based on the principles given in this International Standard.

A structure that complies with the requirements given in Clauses 5 to 13 of this International Standard may be considered fit-for-purpose. Demonstration of adequate fitness-for-purpose may include justified deviation from Clauses 5 to 13, or may be achieved by modifications to either the structure or its operation (i.e. prevention and mitigation measures).

NOTE Further guidance can be found in ISO 19902, which gives a more detailed outline of the procedure for assessment of existing fixed steel structures.

## 15.2 Structural assessment initiators

An existing structure shall be assessed to demonstrate its fitness-for-purpose if one or more of the conditions given in a) or b) exist.

a) Changes from the original design or previous assessment basis, including

   1) addition of personnel or facilities such that the platform exposure level is changed to a more onerous level,

   2) modification to the platform, such that the magnitude or disposition of the permanent, variable or environmental actions on a structure are more onerous,

   3) more onerous environmental conditions and/or criteria,

   4) more onerous component or foundation resistance data and/or criteria,

   5) physical changes to the structure's design basis, e.g. excessive scour or subsidence, and

   6) inadequate deck height, such that waves associated with previous or new criteria will impact the deck, and provided such action was not previously considered.

b) Damage to, or deterioration of, a primary structural component:

   1) minor structural damage may be accepted on the basis of appropriate local analysis without performing an assessment;

   2) cumulative effects of multiple damage shall be documented and included in an assessment.

Exceedance of the original design service life is not, in itself, an assessment initiator except that the effect on the fatigue life shall be considered. Inspection of the structure shall be undertaken to ensure that time-dependent degradation (i.e. corrosion, etc.) has not become significant.

# Annex A
(informative)

# Regional information

## A.1 Introduction

This annex contains clauses for various regions of the world for which regional experts have developed information. For the region or country concerned, each clause in the annex supplements the provisions, information and guidance given in the main body of ISO 19903. Each clause may be considered to constitute the information required for the regional implementation of this International Standard in the particular region or country defined.

The regional information generally provides regional and national data that can include regional environmental conditions and local design, construction and operating practices. Additionally, the regulatory framework for the region or country concerned can be explained.

## A.2 Canada

### A.2.1 Description of region

The geographical basis for this annex is the region bounded by the continental shelf margins of Canada. The region encompasses both shallow water and deepwater areas of offshore Canada that are either ice-free regions (Pacific Ocean off the west coast of British Columbia) or regions that may be subjected seasonally to the presence of sea ice and icebergs. Sea ice can be present in the Beaufort Sea, offshore Newfoundland and Labrador, in the Gulf of St. Lawrence, as well as offshore Nova Scotia, although the occurrence of sea ice in the offshore Nova Scotia area is rare. Icebergs are typically encountered in the waters on the north and east coasts of offshore Newfoundland and Labrador.

### A.2.2 Regulatory framework in Canada

Oil and gas exploration and production activities in Canada's non-Accord Frontier Lands (defined as the Northwest Territories, Nunavut, Sable Island and its submarine areas, and areas not within a province adjacent to the coast of Canada to the outer edge of the continental margin or to a distance of two hundred nautical miles, whichever is greater, but excluding the offshore areas of Nova Scotia and Newfoundland and Labrador), are governed by the *Canada Oil and Gas Operations Act* and the *Canada Petroleum Resources Act*. *The Canada Oil and Gas Operations Act* and certain elements of the *Canada Petroleum Resources Act* are administered by the *National Energy Board* (NEB) in all of the non-Accord Frontier Lands.

Oil and gas exploration and production activities in Canada's Accord Frontier Lands [defined as offshore areas in the *Canada — Nova Scotia Offshore Petroleum Resources Accord Implementation Act* (PRAIA) and the *Canada — Newfoundland Atlantic Accord Implementation Act* (AAIA)] are governed by the PRAIA and AAIA and mirror the provincial Accord Implementation Acts respectively. These Acts are administered by joint federal-provincial offshore petroleum boards. In the Nova Scotia Offshore Accord area the regulator is the *Canada-Nova Scotia Offshore Petroleum Board* (C-NSOPB), and in the Newfoundland and Labrador Offshore Accord area the regulator is the *Canada-Newfoundland and Labrador Offshore Petroleum Board* (C-NLOPB).

For the offshore areas, the three boards (NEB, C-NSOPB and C-NLOPB) are responsible for the regulation of petroleum activities including

— issuance of authorizations for offshore exploration and development activities,

— health and safety of workers,

- protection of the environment during petroleum activities,
- management and conservation of petroleum resources,
- compliance with the provisions of the laws that deal with employment and industrial benefits by the offshore petroleum board in the Accord area, by the Department of Indian Affairs and Northern Development for non-Accord Frontier Lands north of 60° north and by the Department of Natural Resources for non-Accord Frontier lands south of 60° north, and
- resource evaluation and data collection and distribution.

## A.2.3 Technical requirements for Canada

### A.2.3.1 General

Until the publication of ISO 19906 [7], all requirements for the design of structures for ice and iceberg loads shall be in accordance with CAN/CSA-S471-04 [8].

### A.2.3.2 Reference standard for design

CAN/CSA-S474-04[9] shall be used as the reference standard, supplemented by CAN/CSA-S471-04.

As referenced in the note to 8.2.1, NS 3473.E[2] has been widely used for the design of fixed offshore concrete platforms and is deemed to meet the requirements of 8.2.1. Therefore, NS 3473.E may be used for the design of concrete structures *in lieu* of CAN/CSA-S474-04.

If NS 3473.E is to be used as the reference standard, it shall be accompanied by the appropriate load and resistance factors to provide levels of safety and serviceability that are at least equal to those provided by the requirements of CAN/CSA-S474-04 supplemented by CAN/CSA-S471-04.

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[2] NS 3473.E, *Design of concrete structures, Design and detailing rules* (Norwegian Standard/English translation, 6th edition)

[3] EN 1992 (all parts), *Eurocode 2*: *Design of concrete structures*

[4] API 2000, *Venting Atmospheric and Low-Pressure Storage Tanks: Nonrefrigerated and Refrigerated,* 5th edition, April 1998, American Petroleum Institute

[5] ISO 19901-3, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 3: Topsides structure* 6)

[6] ISO 13702, *Petroleum and natural gas industries — Control and mitigation of fires and explosions on offshore production installations — Requirements and guidelines*

[7] ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures* 6)

[8] CAN/CSA-S471-04, *General Requirements, Design Criteria, the Environment, and Loads*

[9] CAN/CSA-S474-04, *Concrete Structures*

[10] ISO 19904-1, Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars

[11] ISO 19904-2, *Petroleum and natural gas industries — Floating offshore structures — Part 2: Tension leg platforms* 6)

6) Under preparation.

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