INTERNATIONAL STANDARD

ISO 19901-3

Second edition
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Petroleum and natural gas industries — Specific requirements for offshore structures —

Part 3: Topsides structure

Industries du pétrole et du gaz naturel — Exigences spécifiques relatives aux structures en mer —

Partie 3: Superstructures

Reference number
ISO 19901-3:2014(E)

Contents

Page

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 19901-3 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*.

This second edition cancels and replaces the first edition (ISO 19901-3:2010), which has been technically revised.

ISO 19901 consists of the following parts, under the general title *Petroleum and natural gas industries — Specific requirements for offshore structures*:

— *Part 1: Metocean design and operating considerations*

— *Part 2: Seismic design procedures and criteria*

— *Part 3: Topsides structure*

— *Part 4: Geotechnical and foundation design considerations*

— *Part 5: Weight control during engineering and construction*

— *Part 6: Marine operations*

— *Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

— *Part 8: Marine soil investigations*

A future Part 9 dealing with structural integrity management is under preparation.

The first edition of ISO 19901-3:2010 included a number of serious typographical errors. A 'Corrected' version of the first edition was issued in December 2011. This 'Corrected' version first edition was subsequently issued by some national standards organisations. To ensure all national standards bodies issue a 'Corrected' version of the document, TC 67/SC 7 decided to produce a second edition of 19901-3 which incorporates the following changes from the original issue in 2010:

— in 4.1, the symbol S_d for design internal force or moment has been added;

— in 8.1, Formulae (7), (8) and (9) have been amended to include symbol S_d and the second paragraph has been reworded to reflect the changes in the equations;

— in 9.18, first paragraph, new values have been given for variable action for the grating and plating as well as for the contribution of personnel to the total variable action allowance;

— in A.7.10.4.2.2, the text has been reworded and Formula (A.1) has been amended, in line with the modifications in 8.1;

— in A.8.1, Formula (A.5) has been corrected by changing "max" to "min";

— in B.2, Table B.1, the value of Young's modulus has been amended so as to be in accordance with the default value recommended in ISO 19902;

— in Tables B.3, B.4, B.5, B.7, B.8 and B.9, some values have been updated to reflect the change in Young's modulus;

— in B.3.3, Table B.4, the symbol for utilization has been corrected;

— in B.4.5, Table B.10, all values for compression and for compression and bending have been amended, as well as the value for the minimum ratio;

— in B.4.5, first and second paragraphs, the building code correspondence factor has been amended and a sentence about its applicability has been added;

— in Annex C, Table C.1, the existing building code correspondence factor has been amended and a second correspondence factor, relating to CSA S16-09, has been added;

— in the Bibliography, Reference[3] has been updated with a more recent edition; references in the text (see A.5.2, A.8.3.1, A.8.3.2, A.8.3.3 and A.8.3.4) have been updated accordingly.

In producing the second edition the following additional minor corrections have been applied to the 2011 'Corrected' version of the first edition:

— in 9.5.3.4 the units of the area-imposed action corrected to kN/m^2;

— in 9.6.2 the description of off-lead and side-lead in Table 5 improved;

— in A.7.10.4.2.3 the reference to section A.7.10.2.4 changed to A.7.10.4.2.4;

— in A.11.3 minor text correction;

— in Annex B Table B.1, symbols for bending amplification reduction factor corrected to $C_{m,y}$ and $C_{m,z}$

ISO 19901 is one of a series of International 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*

— 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 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*

— ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*

— ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

Introduction

The series of International Standards applicable to types of offshore structure, 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 the 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 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.

This part of ISO 19901 has been prepared for those structural components of offshore platforms which are above the wave zone and are not part of the support structure or of the hull. Previous national and international standards for offshore structures have concentrated on design aspects of support structures, and the approach to the many specialized features of topsides has been variable and inconsistent, with good practice poorly recorded.

Historically, the design of structural components in topsides has been performed to national or regional codes for onshore structures, modified in accordance with experience within the offshore industry, or to relevant parts of classification society rules. While this part of ISO 19901 permits use of national or regional codes, and indeed remains dependent on them for the formulation of component resistance equations, it provides modifications that result in a more consistent level of component safety between support structures and topsides structures.

In some aspects, the requirements for topsides structures are the same as, or similar to, those for fixed steel structures; in such cases, reference is made to ISO 19902, with modifications where necessary. Annex A provides background to, and guidance on, the use of this part of ISO 19901, and is intended to be read in conjunction with the main body of this part of ISO 19901. The clause numbering in Annex A follows the same structure as that in the body of the normative text in order to facilitate cross-referencing.

Annex B provides an example of the use of national standards for onshore structures in conjunction with this part of ISO 19901.

Regional information on the application of this part of ISO 19901 to certain specific offshore areas is provided in Annex C.

In International Standards, the following verbal forms are used:

— “shall” and “shall not” are used to indicate requirements strictly to be followed in order to conform to the document and from which no deviation is permitted;

— “should” and “should not” are used to indicate that, among several possibilities, one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required, or that (in the negative form) a certain possibility or course of action is deprecated but not prohibited;

— “may” is used to indicate a course of action permissible within the limits of the document;

— “can” and “cannot” are used for statements of possibility and capability, whether material, physical or causal.

Petroleum and natural gas industries — Specific requirements for offshore structures —

Part 3: Topsides structure

1 Scope

This part of ISO 19901 gives requirements for the design, fabrication, installation, modification and structural integrity management for the topsides structure for an oil and gas platform. It complements ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, which give requirements for various forms of support structure. Requirements in this part of ISO 19901 concerning modifications and maintenance relate only to those aspects that are of direct relevance to the structural integrity of the topsides structure.

The actions on (structural components of) the topsides structure are derived from this part of ISO 19901, where necessary in combination with other International Standards in the ISO 19901 series. The resistances of structural components of the topsides structure can be determined by the use of international or national building codes, as specified in this part of ISO 19901. If any part of the topsides structure forms part of the primary structure of the overall structural system of the whole platform, the requirements of this part of ISO 19901 are supplemented with applicable requirements in ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906.

This part of ISO 19901 is applicable to the topsides of offshore structures for the petroleum and natural gas industries, as follows:

— topsides of fixed offshore structures;

— discrete structural units placed on the hull structures of floating offshore structures and mobile offshore units;

— certain aspects of the topsides of arctic structures.

This part of ISO 19901 is not applicable to those parts of the superstructure of floating structures that form part of the overall structural system of the floating structure; these parts come under the provisions of ISO 19904-1. This part of ISO 19901 only applies to the structure of modules on a floating structure that do not contribute to the overall integrity of the floating structural system.

This part of ISO 19901 is not applicable to the structure of hulls of mobile offshore units.

This part of ISO 19901 does not apply to those parts of floating offshore structures and mobile offshore units that are governed by the rules of a recognized certifying authority and which are wholly within the class rules.

Some aspects of this part of ISO 19901 are also applicable to those parts of the hulls of floating offshore structures and mobile offshore units that contain hydrocarbon processing, piping or storage.

This part of ISO 19901 contains requirements for, and guidance and information on, the following aspects of topsides structures:

— design, fabrication, installation and modification;

— in-service inspection and structural integrity management;

— assessment of existing topsides structures;

— reuse;

— decommissioning, removal and disposal;

— prevention, control and assessment of fire, explosions and other accidental events.

This part of ISO 19901 applies to structural components including the following:

— primary and secondary structure in decks, module support frames and modules;

— flare structures;

— crane pedestal and other crane support arrangements;

— helicopter landing decks (helidecks);

— permanent bridges between separate offshore structures;

— masts, towers and booms on offshore structures.

2 Normative references

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

ISO 2631-1, *Mechanical vibration and shock — Evaluation of human exposure to whole-body vibration — Part 1: General requirements*

ISO 2631-2, *Mechanical vibration and shock — Evaluation of human exposure to whole-body vibration — Part 2: Vibration in buildings (1 Hz to 80 Hz)*

ISO 13702, *Petroleum and natural gas industries — Control and mitigation of fires and explosions on offshore production installations — Requirements and guidelines*

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

ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*

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 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*

ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900, ISO 19902 and the following apply.

3.1
abnormal value
design value of a parameter of abnormal severity used in accidental limit state checks in which a structure is intended not to suffer complete loss of integrity

Note 1 to entry: Abnormal events are typically accidental and environmental (including seismic) events having probabilities of exceedance of the order of 10^{-3} to 10^{-4} per annum.

[SOURCE: ISO 19900:2013, definition 3.1]

3.2
accidental situation
design situation involving exceptional conditions of the structure or its exposure

EXAMPLE Impact, fire, explosion, loss of intended differential pressure.

[SOURCE: ISO 19900:2013, definition 3.2]

3.3
active fire protection
system of fire protection that reacts to a fire by discharging water or an inert or reactive substance in the vicinity of the fire to extinguish it

Note 1 to entry: There is a possibility that such a system fails to operate as designed.

3.4
caisson
appurtenance used for abstracting water from the sea or as a drain

3.5
conductor
tubular pipe extending upward from or beneath the sea floor containing pipes that extend into the petroleum reservoir

[SOURCE: ISO 19900:2013, definition 3.12]

Note 1 to entry: A conductor is generally vertical, and is continuous from below the sea floor to the wellbay in the topsides and can be laterally supported in both the support structure and topsides structure. The vertical support is in the seabed.

Note 2 to entry: In a few cases, conductors are rigidly attached to the topsides or to the support structure above sea level. In these cases, the conductor's axial stiffness can affect the load distribution within the overall structure.

3.6
critical component
structural component, failure of which would cause failure of the whole structure, or a significant part of it

Note 1 to entry: A critical component is part of the primary structure.

[SOURCE: ISO 19902:2007, definition 3.12]

3.7
design accidental action
accidental action with a probability of occurrence greater than 10^{-3} to 10^{-4} per year

3.8
design service life
assumed period for which a structure is used for its intended purpose with anticipated maintenance, but without substantial repair being necessary

[SOURCE: ISO 19900:2013, definition 3.16]

3.9
design situation
set of physical conditions representing real conditions during a certain time interval, for which the design demonstrates that relevant limit states are not exceeded

[SOURCE: ISO 19900:2013, definition 3.17]

3.10
design value
value derived from the representative value for use in the design verification procedure

[SOURCE: ISO 19900:2013, definition 3.18]

3.11
explosion
rapid chemical reaction of gas or dust in air

Note 1 to entry: An explosion results in increased temperatures and pressure impulses. A gas explosion on an offshore platform is usually a deflagration in which flame speeds remain subsonic.

[SOURCE: ISO 19902:2007, definition 3.17]

3.12
exposure level
classification system used to define the requirements for a structure based on consideration of life safety and consequences of failure

Note 1 to entry: 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.

[SOURCE: ISO 19900:2013, definition 3.20]

3.13
extreme value
value of a parameter used in ultimate limit state checks, in which a structure's global behaviour is intended to stay in the elastic range

Note 1 to entry: Extreme values and events have probabilities of exceedance of the order of 10^{-2} per annum.

[SOURCE: ISO 19902:2007, definition 3.19]

3.14
load case
compatible load arrangements, sets of deformations and imperfections considered simultaneously with permanent actions and fixed variable actions for a particular design or verification

[SOURCE: ISO 19902:2007, definition 3.29]

Note 1 to entry: Load arrangements are the identification of the position, magnitude and direction of a free action.

3.15
mitigation
action taken to reduce the consequences of a hazardous event

EXAMPLE Provision of fire or explosion walls; use of water deluge on gas detection; structural strengthening.

3.16
nominal value
value assigned to a basic variable determined on a non-statistical basis, typically from acquired experience or physical conditions

[SOURCE: ISO 19900:2013, definition 3.29]

3.17
owner
representative of the company or companies owning or leasing a development

[SOURCE: ISO 19900:2013, definition 3.34]

3.18
passive fire protection
PFP
coating on the surface of a structural component that improves the structural component's resistance to fire

Note 1 to entry: Some PFP can produce toxic fumes in fires.

3.19
platform
complete assembly including structure, topsides, foundations and stationkeeping systems

[SOURCE: ISO 19900:2013, definition 3.35]

3.20
regulator
authority established by a national governmental administration to oversee the activities of the offshore oil and natural gas industries within its jurisdiction, with respect to the overall safety to life and protection of the environment

Note 1 to entry: The term *regulator* can encompass more than one agency in any particular territorial waters.

Note 2 to entry: The regulator can appoint other agencies, such as marine classification societies, to act on its behalf, and in such cases, *regulator* as it is used in this International Standard includes such agencies.

Note 3 to entry: In this International Standard, the term *regulator* does not include any agency responsible for approvals to extract hydrocarbons, unless such agency also has responsibility for safety and environmental protection.

[SOURCE: ISO 19902:2007, definition 3.40]

3.21
representative value
value assigned to a basic variable for verification of a limit state

[SOURCE: ISO 19900:2013, definition 3.38]

3.22
return period
average period between occurrences of an event or of a particular value being exceeded

Note 1 to entry: The offshore industry commonly uses a return period measured in years for environmental events. The return period in years is equal to the reciprocal of the annual probability of exceedance of the event.

[SOURCE: ISO 19900:2013, definition 3.40]

3.23
riser
tubular used for the transport of fluids between the sea floor and a termination point on the platform

Note 1 to entry: For a fixed structure the termination point is usually the topsides. For floating structures, the riser can terminate at other locations of the platform.

[SOURCE: ISO 19900:2013, definition 3.41]

Note 2 to entry: A riser can be supported both laterally and vertically in the topsides structure and transmit actions from thermal effects, wave action, permanent and variable actions and variations in fluid flow to the topsides structure.

3.24
robustness
ability of a structure to withstand accidental and abnormal events without being damaged to an extent disproportionate to the cause

[SOURCE: ISO 19900:2013, definition 3.42]

3.25
safety-critical element
SCE
item of structure, piping or equipment, the failure of which can result in major accidents or which is provided to prevent or mitigate against them

EXAMPLE Primary structure, pressure-containing equipment, blow-down and other safety systems, vessels and pipework containing hazardous materials, fire and gas detection systems, supports for SCE.

3.26
structural component
physically distinguishable part of a structure

EXAMPLE Column, beam, stiffened plate, tubular joint, or foundation pile.

[SOURCE: ISO 19900:2013, definition 3.46]

3.27
support structure
structure supporting the topsides

Note 1 to entry: The support structure can take many forms including fixed steel (see ISO 19902), fixed concrete (see ISO 19903), floating (see ISO 19904-1), mobile offshore units (see ISO 19905-1), or the various forms of arctic structures (see ISO 19906).

3.28
topsides
structures and equipment placed on a supporting structure (fixed or floating) to provide some or all of a platform's functions

Note 1 to entry: For a ship-shaped floating structure, the deck is not part of the topsides.

Note 2 to entry: For a jack-up, the hull is not part of the topsides.

Note 3 to entry: A separate fabricated deck or module support frame is part of the topsides.

[SOURCE: ISO 19900:2013, definition 3.52]

4 Symbols and abbreviated terms

4.1 Symbols

a acceleration

A accidental action

b spacing of stiffeners

D_e equivalent quasi-static action representing dynamic response effects to the extreme environmental action, E_e

D_o equivalent quasi-static action representing dynamic response effects to the operating environmental action, E_o

E quasi-static environmental action

E_e extreme quasi-static environmental action due to wind, waves and current

E_o quasi-static environmental action due to wind, waves and current for an operating condition under consideration (see 7.3.4)

F_d design action

F_G vertical action due to self-weight of a crane

F_H horizontal action due to off-lead and side-lead on a crane

F_r representative action

F_{rhl} representative hook load of a crane

F_W maximum operating wind action on a crane

$F_{W,ext}$ extreme wind action on a crane

g acceleration due to gravity

G permanent action

I explosion impulse

l span or length

K_c building code correspondence factor

p instantaneous explosion overpressure

$p(t)$ variation of overpressure with time

P probability

Q variable action

R resistance

R_D design resistance

R_K representative resistance

S internal force or moment

S_d design internal force or moment

t time from ignition of an explosion

t_d duration of explosion pressure pulse

T fundamental period of vibration of a component or structure

$T_{C,max}$ maximum allowable temperature in a component

δ thickness of a structural component, plate, or finite element

γ partial safety factor

γ_f partial action factor

γ_{FD} partial damage design factor

γ_R partial resistance factor

Δ deflection

ε_{cr} critical average strain

σ stress

θ temperature

4.2 Abbreviated terms

AISC American Institute of Steel Construction

ALARP as low as reasonably practicable

ALE abnormal level earthquake

ALS accidental limit states

API American Petroleum Institute

ASD allowable stress design

AVM anti-vibration mounting

CFD computational fluid dynamics

CTOD crack tip opening displacement

CVI close visual inspection

DAF dynamic amplification factor

DLB ductility level blast

ELE extreme level earthquake

FEA finite element analysis

FLS fatigue limit states

FPSO floating production storage and off-loading

FSO floating storage and off-loading

GVI general visual inspection

LRFD load and resistance factor design

MPI magnetic particle inspection

MTOW maximum take-off weight

PFP passive fire protection

SCE safety-critical element

SDOF single degree of freedom

SLB strength level blast

SLS serviceability limit states

ULS ultimate limit states

UR utilization ratio

WSD working stress design

5 Overall considerations

5.1 Design situations

Design situations include all operational requirements, temporary conditions, environmental conditions and accidental and abnormal conditions which could affect the design. Adequate planning shall be undertaken before detailed design is started in order to obtain a workable and economical topsides layout and structure to perform its functions. The initial planning shall include the determination of all criteria upon which the design of the topsides will be based.

An important consideration for most topsides structures is the extent and magnitude of possible fire and explosion actions, which can be affected by the structural and equipment layouts, congestion and containment, and the initial planning shall include these layout considerations.

5.2 Codes and standards

5.2.1 Limit state and allowable stress design philosophies

In general, the ISO 19900 series[1)] of International Standards follows the requirements of ISO 2394[63] and as such is intended to be a limit state design standard. Limit state methods are also known as load and resistance factor design (LRFD) methods, particularly in North America.

Within the ISO 19900 series, ISO 19902 and ISO 19903 are explicit limit state design standards. ISO 19904-1 allows allowable stress design (ASD) or limit state design methods, but limit state methods for ships are only currently being finalized in LRFD formats.

NOTE ASD methods are also known as working stress design (WSD) and permissible stress design methods.

The intent of this part of ISO 19901 is that limit state methods should be used where possible, but, where the supporting structure is designed using ASD methods, such as for floating structures, ASD methods may also be used for the topsides structure using a current ASD code compatible with that used for the supporting structure.

The specific requirements and guidance in this part of ISO 19901 apply to the use of limit state (or LRFD) codes. Where ASD codes are used, the requirements for partial action and partial resistance factors do not apply, but all other requirements still apply.

1) The ISO 19900 series consists of the following International Standards: ISO 19900, ISO 19901 (all parts), ISO 19902, ISO 19903, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. These standards have all been prepared by ISO/TC 67/SC 7 for offshore structures for the petroleum and natural gas industries.

5.2.2 Use of national codes and standards

The detailed design for a topsides structure shall be based on national or regional building codes. These should normally be those for the nation or region in which the platform is to be located, but may, with the agreement of the owner and the regulator, be those from other nations or regions. The standards used for fabrication should be consistent with and compatible with those used for design.

NOTE As stated in Clause 1, this part of ISO 19901 does not apply to those parts of floating offshore structures and mobile offshore units that are governed by the rules of a recognized classification society and which are wholly within the class rules.

In order to realize a similar level of reliability to that implicit in other standards in the 19900 series, the action factors shall be taken from the relevant standard in the ISO 19900 series for the support structure and shall be used unmodified. Resistance factors in the national or regional building code shall be modified by the application of a building standard correspondence factor (see 8.1).

Floating structures and production jack-ups that are registered as vessels are also subject to any requirements of the prospective flag state or any classification society acting on behalf of the flag state.

5.3 Deck elevation and green water

Air gap requirements for fixed structures are addressed in ISO 19901-1, ISO 19902 and ISO 19903. No component of the topsides structure or equipment shall be within the design air gap unless explicitly designed to withstand possible hydrodynamic actions and to transmit such actions through the topsides to the support structure. These actions should be identified to the support structure designer as early as possible.

For floating structures, and for monohulls in particular, increasing the height of topsides modules above the main deck is a trade-off between reducing the potential effects of explosion pressures, increasing accessibility, and reducing stability. These floating structures can also be inundated in severe weather if the top of the wave crest is higher than the deck of the structure. The inundating water, known as green water, can run along the deck and impact equipment and structures on the deck. The actions associated with possible green water shall be evaluated and any vulnerable topsides structures shall be designed to withstand these actions.

5.4 Exposure level

The exposure level for the topsides structure shall be the same as for the support structure and shall be determined in accordance with the criteria given in ISO 19900. Additional guidance is provided in ISO 19902, ISO 19903, 19904-1, ISO 19905-1 and ISO 19906.

NOTE There are three categories of exposure level: L1 structures are manned during design conditions or have high consequences of failure; L2 platforms are not expected to be manned during governing design conditions and have medium consequences of failure; and L3 platforms are normally unmanned and have low consequences of failure.

5.5 Operational considerations

5.5.1 Function

The structural engineer shall be aware of the functional requirements of the platform including drilling, process, access, safety and auxiliary systems. In particular, the integrity of the structure shall comply with the platform's safety philosophy and any defined performance standards.

5.5.2 Spillage and containment

Provision for handling spills, overflows and potential contaminants should be provided. A deck drainage system shall be considered that collects and stores liquid spillages and overflows for subsequent handling.

5.6 Selecting the design environmental conditions

The design environmental conditions (metocean, ice and seismic) for the topsides shall be those selected for the support structure. Less onerous environmental conditions can be acceptable for specific short-term operations.

The wind speed shall be modified depending upon the dimensions and elevation of the part of the structure or the component being considered (see ISO 19901-1).

EXAMPLE The short-term installation of specific pieces of equipment can be considered with lower environmental conditions if an assessment of the risks and consequences of exceeding the environmental criteria are considered.

5.7 Assessment of existing topsides structures

Any assessment of existing topsides structures to confirm that they comply with this part of ISO 19901 or are fit for purpose shall be performed in accordance with the assessment requirements of ISO 19900, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate.

5.8 Reuse of topsides structure

Existing topsides structures may be removed and relocated for use at a new location. When this is considered, the topsides structure shall be evaluated in accordance with the requirements of ISO 19900, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate, for the use (including exposure level) and conditions that are applicable at the new location. Any repairs or modifications that are necessary shall be in accordance with the requirements of this part of ISO 19901.

5.9 Modifications and refurbishment

Where modifications or refurbishment of an existing topsides structure are planned, the structure shall be assessed for the revised configuration in accordance with the assessment requirements of ISO 19900, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate. All changes shall be documented (see Clause 11). In some cases, more advanced techniques than are usual for design can be required to allow economical strengthening or stiffening schemes to be developed and justified. Such techniques are outside the scope of this part of ISO 19901.

6 Design requirements

6.1 General

This clause presents the overall minimum requirements for design of topsides. The general principles on which structural design requirements are based are given in ISO 19900.

6.2 Materials selection

Materials shall be selected that have a performance compatible with the design code. ISO 19902 gives specific requirements and guidance for the selection of carbon steel for topsides structure. Clause 10 addresses alternative materials.

6.3 Design conditions

The topsides shall be designed to resist permanent and variable actions; wind, wave, and current actions; earthquake actions; temperature and deformation effects, temporary conditions and accidental conditions – all of which can occur during its service life. These actions shall include both actions directly applied to the topsides and also the effects of actions on the supporting structure (such as waves, currents and earthquakes). In addition, actions due to the motions of the support structure shall be considered; these are particularly significant for floating structures.

The nominal values of these actions, or the derivation of these values, are given in Clause 7. Each mode of operation of the platform, such as drilling, production, work-over, or anticipated combinations thereof, shall be explicitly considered. In areas where icing can occur, the effects of both the weight of ice accretion and the increase in effective dimensions of components due to the ice, resulting in increased wind actions, shall be included.

6.4 Structural interfaces

Particular attention shall be paid to the following:

— interfaces between different structures in order to ensure adequate alignment when fabrication tolerances are taken into account;

— compatibility of stiffnesses, distortions and displacements during fabrication, installation and in-service conditions.

The effects of displacements on different structures supported on one or more separate structures shall be considered.

EXAMPLE The flexing of a ship-shaped floating structure can cause significant differential movements between adjacent modules supported on the hull of the structure.

6.5 Design for serviceability limit states (SLS)

6.5.1 General

The serviceability of the topsides structures can be affected by excessive relative displacement or vibration (vertical or horizontal). Limits for either can be dictated by

a) discomfort to personnel,

b) integrity and operability of equipment or connecting pipework,

c) control of deflection of supported structures, e.g. flare structures and telecommunication masts,

d) damage to architectural finishes, or

e) operational requirements for drainage (free surface or piped fluids).

The vibration limits are specified in 6.5.2 and the deflection limits are specified in 6.5.3.

6.5.2 Vibrations

6.5.2.1 Sources of vibration

All sources of vibration shall be considered in the design of the topsides structure. As a minimum, the following shall be reviewed for their effect on the structure:

a) operating mechanical equipment, including that used in drilling operations;

b) vibrations from variations of fluid flow in piping systems, in particular slugging;

c) vortex-induced vibrations on slender tubular structures due to wind;

d) global motions due to environmental actions on the total platform structure;

e) vibrations due to earthquake and accidental events.

6.5.2.2 Design limits

Design limits for vibration shall be established from operational limits set by equipment suppliers and the requirements of personnel comfort and health and safety.

The design limits for horizontal and vertical vibration effects on personnel shall not exceed those given in ISO 2631-1 and ISO 2631-2. More onerous limits can be required by the owner or by the regulator.

6.5.2.3 Long-period vibrations

Large cantilevers (whether formed by simple beams or trusses) forming an integral part of the topsides, but excluding masts or booms, shall normally be proportioned to have a natural period of less than 1 s in the operating condition.

6.5.2.4 Dynamic analysis and avoidance of resonance

Where necessary, analytical techniques shall be used to assess the dynamic response of various parts of the topsides to ensure that resonance is avoided. The dynamic behaviour of large cantilevers can be calculated by eigenvalue analysis. Such analysis should include unfactored static and imposed actions. Where heavy rotating machinery is installed (such as variable speed pump skids, compressors, etc.), three-dimensional vibration analysis should be performed. To avoid resonance, the cantilevered local structure should be designed such that the natural frequencies of the deck do not lie between 0,65 times and 1,5 times the operating frequency of the equipment supported.

6.5.2.5 Deflections

The final deflected shape, Δ_{max}, of any element, structural component or structure comprises three parts as follows:

$$\Delta_{max} = \Delta_1 + \Delta_2 - \Delta_0 \tag{1}$$

where

Δ_0 is any pre-camber (hogging) of a beam or element prior to the addition of any permanent or variable actions;

Δ_1 is the deflection from permanent actions after applying the actions;

Δ_2 is the deflection from the variable actions and any time-dependent deformations from permanent actions.

The maximum values for vertical deflections are given in Table 1.

Table 1 — Maximum vertical deflections

Structural component	Maximum deflection	
	Δ_{max}	Δ_2
Floor beams	$\frac{l}{200}$	$\frac{l}{300}$
Cantilever beams	$\frac{l}{100}$	$\frac{l}{150}$
Deck plate thickness	—	2δ or $\frac{b}{150}$ [a]

l span

δ deck thickness

b stiffener spacing

a Whichever is smaller.

More onerous limits can be required by the owner or by the regulator or can be specified for individual items of equipment.

Lower limits can be necessary to limit ponding of surface fluids and ensure that drainage systems function correctly; ponding should be avoided by cambering deck plating in areas susceptible to icing of any surface water.

The alignment of telecommunications equipment can be critical for their reliable operation and due consideration should be given to maintaining the required tolerances for such equipment.

Horizontal deflections shall generally be limited to 0,3 % of the height between floors. For multi-floor modules, the total horizontal deflection shall not exceed 0,2 % of the total height of the topsides structure. More onerous limits can be necessary to limit pipe stresses.

Higher deflections can be acceptable for cladding panels and other components where serviceability is not compromised by deflection.

6.6 Design for ultimate limit states (ULS)

To obtain design actions, an action factor shall be applied to each of the representative applied actions in the combinations given in Clause 7. The action factors are given in the relevant International Standard in the ISO 19900- series, or other document relevant to the supporting structure, and are described in Clause 7.

The combination of factored representative actions causes amplified internal forces and moments, *S*.

A resistance factor is applied to the representative strength of each component to determine its design strength. Each component shall be proportioned to have sufficient factored strength to resist *S*. The appropriate strength and stability criteria shall be taken from the appropriate national or international building code and shall be modified by a correspondence factor to account for any differences in approach between the building code and the International Standards in the ISO 19900- series to become the formulae for the representative strength of the component. This is to ensure that a similar level of reliability for topsides design are achieved to that implied in other International Standards in the ISO 19900- series.

In some conditions, particularly during construction and installation, the internal forces should be computed from unfactored representative actions and then modified by appropriate action factors to arrive at *S* (see the relevant International Standard in the ISO 19900- series).

6.7 Design for fatigue limit states (FLS)

The design actions to be used in the FLS are addressed in the International Standard in the ISO 19900-series that is applicable to the support structure. Additional considerations for specific systems are given in Clause 9. The fatigue methodology in ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate, shall be followed in the design of the topsides structure; in cases where the support structure standard does not provide adequate fatigue guidance for a topsides structure, the requirements of ISO 19902 shall apply.

In place of more detailed assessment, the fatigue damage design factors can be taken from Table 2.

Table 2 — Partial damage design factors, γ_{FD}

Failure critical component	Inspectable	Not inspectable
No	2	5
Yes	5	10

Where the topsides structure is subjected to a long sea transportation prior to installation on the support structure, particular attention shall be given to the fatigue performance of structural details that do not normally see significant fatigue actions.

6.8 Design for accidental limit states (ALS)

ALS are addressed in 7.10, where requirements and recommendations are given for determining the conditions and actions, the partial action factors and the partial resistance factors.

6.9 Robustness

The topsides shall incorporate robustness through consideration of the effects of all hazards and their probabilities of occurrence to ensure that damage is not disproportionate to the cause. Damage from an event with a reasonable likelihood of occurrence shall not lead to complete loss of integrity of the structure. In such cases, the structural integrity in the damaged state shall be sufficient to allow a process system close-down, or a safe evacuation, or both. Framing patterns that provide alternative load paths are preferred.

Qualitative assessments to evaluate measures to improve the robustness of critical tertiary structure, such as pipe supports and equipment anchorages, shall be considered before detailed analysis. For a new topsides, a walk-down study shall be carried out in the fabrication yard or shortly after installation, but before production starts.

Walk-downs are methodical, on-site, visual evaluations of existing structures and equipment as installed. The walk-down scope shall include

— planning for the walk-down,

— preparation of walk-down documentation,

— a screening evaluation to determine zones of potential severe vibration and to identify structures and equipment most at risk,

— the walk-down itself,

— post-walk-down assessment, and

— reporting and recommendations.

Planning shall include an assessment to identify areas and components most at risk. Post-walk-down assessment shall include simple calculations to determine the adequacy of anchorages where there is doubt as to their suitability. Any remedial actions necessary shall be identified, reported and implemented as quickly as possible.

6.10 Corrosion control

The design of structural details shall, whenever possible, avoid corrosion traps and provide for free drainage of liquids.

The design of corrosion protection shall be compatible with the design assumptions for the topsides structure. Corrosion-related aspects to be considered include

a) the corrosion allowance (if any) considered in the design,

b) the design life and requirement for planned maintenance,

c) access for maintenance of corrosion protection systems,

d) the protection of details sensitive to crevice corrosion (e.g. bolted connections and the interface between piping and pipe supports),

e) the protection of voids vulnerable to corrosion (e.g. by plugging vent holes in pipe supports after welding),

f) the specification of requirements for corrosion protection, and

g) the avoidance of galvanic corrosion (e.g. between carbon steel framework and aluminium helidecks and between carbon steel framework and stainless steel process pipework and vessels).

Where structural components are also used for fluid storage, e.g. diesel tanks within crane pedestals, a suitable corrosion control system shall be installed.

Where a corrosion allowance is incorporated in any component, the allowance shall be documented for use in inspection planning and assessment (see Clause 14).

Particular attention shall be paid to the prevention of water leakage and subsequent corrosion under lagging systems and under passive fire protection (PFP) systems.

Further requirements and recommendations for corrosion control are given in Clause 12. In general, the thickness of a painted steel deck will be governed by the need for stiffness and to prevent excessive weld-induced distortion rather than by corrosion considerations.

6.11 Design for fabrication and inspection

The designers shall be familiar with, and anticipate likely methods of, fabrication, welding and erection to execute the design and they shall provide a design which accommodates these through the provision of appropriate material thicknesses, clearances, access and stability at all stages of construction.

The design shall be prepared with a clear understanding of the level of in-service inspection and maintenance planned during the topsides structure's life. Where the integrity of the topsides structure during its design life requires mandatory in-service inspection, provision for access for such inspection shall be included.

The design assumptions with respect to in-service inspection shall be clearly recorded and communicated to the fabricator and owner.

The design intent shall be followed during construction and variances shall be resolved without compromising the design intent.

The designers shall communicate the extent, type and rejection criteria for all non-destructive inspections. Where performance level (e.g. fatigue performance) depends on the achievement of particular standards in construction, the designer shall ensure that these are communicated.

NOTE Requirements are given elsewhere in this part of ISO 19901 for materials (see Clause 10), quality control, quality assurance and documentation, welding and fabrication inspection (see Clause 12).

6.12 Design considerations for structural integrity management

During the design, fabrication, inspection, transportation and installation of the topsides structure, sufficient data shall be collected and compiled for use in preparing in-service inspection programmes, possible topsides modifications, etc. Where a topsides structure has fatigue-sensitive components or other critical areas, these shall be identified and the information used in the preparation of in-service inspection programmes.

The design of equipment supports and skids shall provide adequate access to the structure to facilitate inspection and maintenance (e.g. painting) of primary and secondary steelwork and of equipment

6.13 Design for decommissioning, removal and disposal

6.13.1 General

Decommissioning and removal requirements shall be addressed during the topsides structure design phase, particularly for fixed platforms and deep draught floating platforms. Where the preferred removal option requires the use of special features, these should be considered for inclusion in the topsides structure during its fabrication. The platform's structural integrity management system should prevent in-service structural modifications that can prejudice later removal.

6.13.2 Structural releases

Consideration should be given to designing secondary structures between modules and elsewhere that are supported from one side only so as not to depend on temporary supports during dismantling.

The design of module support points, anti-vibration mountings and equipment supports shall consider access requirements for future disconnection.

6.13.3 Lifting appurtenances

Lifting attachments for installation of the topsides structure should be retained for subsequent use during decommissioning. Where attachments are removed during installation or during the service life of the topsides structure, consideration shall be given to facilitating their reattachment or replacement for subsequent removal.

The design should allow for periodic access for inspection.

6.13.4 Heavy lift and set-down operations

The dynamic impact factors used in design should allow for a removal case involving set-down onto a barge that could be more severe than for installation, if this seems probable.

7 Actions

7.1 General

A topsides structure can be exposed to a number of design situations throughout its design service life. These include

— extreme conditions of wind, waves and currents,

— normal operating conditions of wind, waves and currents,

— fabrication,

— transportation,

— installation,

— fatigue: pre-installation and during the design service life,

— accidental situations including fire, explosions, ship impact and dropped objects,

— abnormal conditions of wind, waves and currents,

— earthquakes, and

— dismantling/removal.

Each of these design situations comprises several actions such as permanent, variable and environmental actions, deformations, temperature effects and accidental events, each with appropriate partial action factors.

General guidance on the design situations is given in ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate.

7.2 In-place actions

Each topsides structural component shall be assessed for internal force (action effect), S, resulting from the design action, F_d. The design action shall be derived from the following combinations of actions:

a) maximum permanent and variable actions, G_1, G_2, Q_1, and Q_2;

b) extreme environmental actions, G_1, G_2, Q_1, E_e and D_e, together with any actions resulting from associated support structure movements;

c) operating environmental actions, G_1, G_2, Q_1, Q_2, E_o and D_o, together with any actions resulting from associated support structure movements, where

G_1 is the permanent action imposed on the topsides structure by the self-weight of the topsides structure with associated equipment and other objects (see ISO 19902); in addition, any actions due to the misalignment of structures, such as between the topsides structure and the supporting structure, are part of G_1;

G_2 is the permanent action imposed on the topsides structure by self-weight of equipment and other objects that remain constant for long periods of time, but which can change from one mode of operation to another, or during a mode of operation (see ISO 19902);

Q_1 is the variable action imposed on the topsides structure by the weight of consumable supplies and fluids in pipes, process vessels, tanks and stores, the weight of transportable tanks and containers used for delivering supplies, the weight of ice accretions, and the weight of personnel and their personal effects (see ISO 19902); in addition, any actions due to the movement of supporting structures not due to environmental effects, such as trim of a floating production storage and off-loading (FPSO) and the effects of cargo loading, including flexure of the supporting structure due to such effects, are part of Q_1;

Q_2 is the short-duration variable action imposed on the topsides structure from operations such as the lifting of drill string, lifting by cranes, liquids in pipes and process vessels for pressure testing, machine operations, mooring of an adjacent ship to the platform, and helicopters (see ISO 19902);

E_e is the extreme quasi-static environmental action on the topsides structure and any environmental action effects transmitted through the supporting structure (see ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate); in addition, any actions due to the movement of supporting structures due to extreme environmental effects, such as roll of an FPSO, including any consequent flexure of the supporting structure due to such effects, are part of E_e;

D_e is the equivalent quasi-static action on the topsides structure representing dynamic response to the extreme environmental action (see ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate);

E_o is the environmental action on the topsides structure and any environmental action effects transmitted through the support structure for environmental conditions limiting a particular operation (see 7.3.4); in addition, any actions due to the movement of supporting structures due to the operating environmental effects, such as roll of an FPSO, including any consequent flexures of the supporting structure due to such effects, are part of E_o;

D_o is the equivalent quasi-static action on the topsides structure representing dynamic response to the operating environmental action, E_o.

The values of G_1, G_2, Q_1 and Q_2 are often not well defined at early stages of the design process, and the potential lack of accuracy should be taken into account (see A.7.2). In a similar manner, potential variation of the centre of gravity of the topsides during the design process should be considered, particularly with respect to design for loadout, transportation and installation of the topsides.

7.3 Action factors

7.3.1 Design action for in-place situations with permanent and variable actions only

The design action, F_d, for the in-place design situation due to maximum permanent and variable actions only shall be calculated using Formula (2).

$$F_d = \gamma_{f,G1} G_1 + \gamma_{f,G2} G_2 + \gamma_{f,Q1} Q_1 + \gamma_{f,Q2} Q_2 \tag{2}$$

The partial action factors, γ_f, shall be the same as those used for the support structure design or assessment. See ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 or ISO 19906, as appropriate.

7.3.2 Design actions for equipment testing

Tanks, pipework and pressure systems can be subjected to hydrostatic and pressure testing at various stages of a platform's life. The additional actions due to any weight of liquid used for the test shall be included in the short-duration variable action, Q_2.

7.3.3 Design action for in-place situations due to extreme environmental actions

The design action, F_d, for the in-place design situation due to extreme environmental situation actions for topsides on fixed structures shall be calculated using Formula (3).

$$F_d = \gamma_{f,G1} G_1 + \gamma_{f,G2} G_2 + \gamma_{f,Q1} Q_1 + \gamma_{f,Q2} Q_2 + \gamma_{f,Ee} \left(E_e + \gamma_{f,D} D_e \right) \tag{3}$$

For this check, G_2, Q_1 and Q_2 shall be the maximum values associated with the particular operating situation being considered.

The design action for floating structures shall be calculated in a similar manner, which includes allowing for the rotations and accelerations due to the vessel motions.

The partial action factors, γ_f, shall be the same as those used for the support structure design or assessment. See ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, or ISO 19906, as appropriate.

When the internal forces due to permanent and variable actions oppose those due to wind, wave and current actions in extreme environmental conditions, the design action, F_d, shall be calculated in accordance with Formula (4) using reduced partial action factors for the permanent and variable actions.

$$F_d = \gamma_{f,G1} G_1 + \gamma_{f,G2} G_2 + \gamma_{f,Q1} Q_1 + \gamma_{f,Ee} \left(E_e + \gamma_{f,D} D_e \right) \tag{4}$$

For this check, G_2 and Q_1 shall exclude any parts associated with the mode of operation considered that cannot be ensured of being present during the operational conditions.

The partial action factors, γ_f, shall be the same as those used for the support structure design or assessment (see ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, or ISO 19906, as appropriate) for the case when the internal forces due to permanent and variable actions oppose those due to wind, wave and current actions in extreme environmental conditions.

NOTE 1 The appropriate partial action factors for the environmental action depends on the exposure level, the long-term environment at the offshore location of the platform, and the geometrical and structural properties of the structure considered.

NOTE 2 ISO 19902 specifies a value of $\gamma_{f,Ee}$ of 1,35 where no other information is available.

7.3.4 Design actions for in-place situations with operating environmental actions

Platform operations are often limited by environmental conditions and differing limits can be set for different operations. Examples of operations that can be limited by environmental conditions include

— drilling and workover,

— crane transfer to and from supply ships,

— crane operations on deck,

— deck and over-the-side working,

— deck access, and

— helicopter operations.

Each operating situation that is restricted by environmental conditions shall be assessed as demonstrated in Formulae (5) and (6), in which E_o and D_o represent the environmental actions limiting the operations.

The design action, F_d, for in-place situations involving platform operations on fixed structures shall be calculated using Formula (5).

$$F_d = \gamma_{f,G1} G_1 + \gamma_{f,G2} G_2 + \gamma_{f,Q1} Q_1 + \gamma_{f,Q2} Q_2 + \gamma_{f,Eo} \left(E_o + \gamma_{f,D} D_o \right) \quad (5)$$

For this check, G_2, Q_1 and Q_2 shall be the maximum values associated with the particular operating situation being considered.

When the internal forces due to permanent and variable actions oppose those due to wind, wave and current actions in operating environmental conditions, the design action, F_d, shall be calculated in accordance with Formula (6) using reduced partial action factors for the permanent and variable actions.

$$F_d = 0{,}9 G_1 + 0{,}9 G_2 + 0{,}8 Q_1 + \gamma_{f,Eo} \left(E_o + \gamma_{f,D} D_o \right) \quad (6)$$

For this check, G_2 and Q_1 shall exclude any parts associated with the mode of operation considered that cannot be ensured of being present during the operational conditions.

The design action for floating structures shall be calculated in a similar manner, which includes allowing for the rotations and accelerations due to the vessel motions.

NOTE ISO 19902 allows a value of $\gamma_{f,Eo}$ of 0,9 × 1,35 approximately 1,20 where no other information is available.

7.4 Vortex-induced vibrations

For the fabrication, transportation and in-place phases, an assessment of the possibility of vortex-induced vibrations due to wind on exposed structural components shall be undertaken.

The possibility of fatigue due to vortex-induced vibrations on lattice structures (e.g. flare booms and drilling derricks) and exposed pipework shall be considered.

7.5 Deformations

Internal forces due to imposed deformations can arise from the effects of fabrication tolerance, foundation settlement and the indeterminate effects of transportation and lift. Internal forces can also occur from operational or accidental thermal effects.

Where a primary topsides structure is supported by a multi-column gravity base structure, the movements and deformations of the column tops can result in significant indirect actions applied to the topsides structure. For this reason, the support structure and the topsides structure should normally be analysed together for both the ULS and FLS.

All such actions or action effects shall be considered in combination with appropriate operating and environmental actions to ensure that serviceability and ultimate limit states are not exceeded.

The hull of a monohull is usually much stiffer than the topsides structure and, as it sags and hogs, considerable deformations can be introduced at the topsides structure level. It is important that the differences between essentially static behaviour due to ballast and cargo loading, and dynamic behaviour due to environmental effects are understood.

7.6 Wave and current actions

Although wave and current actions mainly affect the supporting structure, directly, there are frequently indirect actions on the topsides due to the displacements and deformations of the supporting structure. All wave and current effects on the supporting structure, appurtenances (e.g. conductors, risers, caissons, etc.) and the topsides shall be included in the calculation of the environmental actions on the topsides, including those listed below.

a) For fixed platforms:

- lateral accelerations of the topsides due to deformations of the supporting structure;
- framing actions due to horizontal actions on the supporting structure;
- particular attention shall be given to topsides on multi-leg concrete platforms in which the wave actions can act in differing directions on different legs, resulting in various forces and moments through the topsides in conditions well below the extreme wave heights.

b) For floating platforms:

- translational accelerations due to sway, surge and heave of the supporting structure;
- rotational accelerations due to roll, pitch and yaw of the supporting structure;
- rotation of the topsides due to roll and pitch with the consequent effects on the directions of actions;
- particular attention shall be paid to the distortions of the supporting structures and the consequent effects on the support points of the topsides structure.

Large actions can result when sea water strikes a platform's deck and equipment. Where insufficient air gap exists, when wave run-up against large diameter legs and columns strikes the deck, or when water inundates the deck for floating structures (green water effects), then all actions resulting from the water flow including buoyancy, inertia, drag and slam shall be taken into account. See ISO 19901-1, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 or ISO 19906, as appropriate.

7.7 Wind actions

Guidance on wind speeds including wind profiles and gust durations is given in ISO 19901-1. The derivation of the wind action on the topsides should follow the methodology in the selected building code (see 5.2.2).

7.8 Seismic actions

7.8.1 General

The topsides structure shall be considered for earthquake conditions as part of the overall platform consisting of the support structure with its foundation, where applicable, and the topsides. The requirements are given in ISO 19901-2. Additional guidelines for components of the topsides are given in 7.8.2.

Topsides structure, equipment piping, and other deck appurtenances shall be designed and supported such that seismic actions induced by the design seismic event can be resisted and displacements can be restrained so that no unacceptable damage to the equipment, piping, appurtenances and their supporting structures occurs.

Special consideration shall be given to the design of restraints for critical piping and equipment whose failure could result in personnel injury, hazardous material spillage, pollution, or hindrance to emergency response.

Design acceleration levels shall include the effects of overall platform dynamic response, and, if appropriate, local dynamic response of the deck and the appurtenance itself. Due to the platform's dynamic response, the design acceleration levels are typically much greater than the ground motions and hence greater than those commonly associated with the seismic design of similar onshore processing facilities.

7.8.2 Minimum lateral acceleration

A minimum design lateral acceleration of 0,2 g shall be applied as an extreme level earthquake (ELE) to the topsides, including equipment, and supporting framework for all structures (except seismic risk category 1, see ISO 19901-2), including those in seismic zone 0.

7.8.3 Equipment and appurtenances

In general, most types of properly anchored equipment and appurtenances are sufficiently stiff for their lateral and vertical responses to be calculated directly from maximum computed deck accelerations, since local dynamic amplification of the appurtenances themselves is negligible. However, in some circumstances the flexibility of the topsides can affect the natural frequency of equipment and appurtenances.

For relatively stiff equipment and appurtenances, in which the mass is small compared to that of the topsides structure and which can reasonably be treated as single degree of freedom (SDOF), a simplified uncoupled analysis may be performed using the following steps:

— from prior modal analysis of the overall platform (see ISO 19901-2), extract the accelerations, a_S, at the equipment support location;

— multiply the equipment or appurtenance mass by the resulting acceleration and design its supports for the resulting actions.

A more rigorous analysis shall be undertaken if any of the following apply:

a) the equipment or appurtenance mass is greater than 5 % of the total platform mass;

b) the equipment or appurtenance has dynamic characteristics or its supporting structure affects its vibration;

c) the SDOF natural period of the equipment or appurtenance exceeds 1,25 times the period of a significant mode of the complete structure.

Where more rigorous analysis is required, it shall be undertaken by

— an uncoupled analysis with deck-level floor response spectra, or

— coupled analysis methods.

Equipment and appurtenances that typically require a more rigorous analysis include drilling rigs, flare booms, vent and communications towers, deck cantilevers, tall process vessels, large unbaffled tanks, bridges and cranes.

Coupled analyses that properly include the dynamic interactions between the equipment or appurtenance and the topsides structure result in more accurate and often lower design accelerations than those derived using uncoupled floor response spectra.

Drilling and well servicing structures shall be designed for earthquake actions in accordance with an appropriate standard (see A.7.8.3) and shall be tied down or restrained at all times except when being moved.

7.9 Actions during fabrication and installation

7.9.1 General

The primary objective of 7.9 is to ensure that a topsides structure begins its design service life with its designed strength and structural integrity intact. Installation encompasses the operations of moving the topsides components from the fabrication site (or prior offshore location) to the support structure, and installing them to form the completed platform. Clause 13 contains further details on installation procedures.

7.9.2 Fabrication

The sequence of construction and any temporary erection conditions, including any jacking and weighing conditions shall be considered to ensure that the ULS requirements are met during all temporary conditions. Individual support reactions during fabrication depend on the stiffness of the topsides structure and of the supporting foundation. Internal forces in the topsides structure due to uneven support points shall be determined.

7.9.3 Loadout, transportation and installation

Specific requirements, recommendations and guidance for marine operations, including loadout, transportation and installation are given in ISO 19901-6.

7.10 Accidental situations

7.10.1 General

Prevention, detection, control and mitigation of accidental situations arising from hazards shall be considered in the design in order to promote inherently safe topsides. Implementing preventive measures has historically been, and will continue to be, the most effective approach in minimizing the probability of occurrence of an event and the resultant consequences of the event. The owner or operator responsible for the overall safety of the platform shall identify the hazard management issues to be considered. Accidental situations shall be identified and assessed by means of hazard analysis performed in accordance with ISO 19900. A suitable screening process is shown in Figure 1. See 7.10.2.

For topsides structures, the accidental and abnormal situations considered shall include

a) explosion,

b) fire,

c) vessel collision,

d) impact from dropped and swinging objects, from projectiles, and from broken cables and wire,

e) helicopter impact (emergency landing or crash), and

f) for floating structures, the effects of accidental flooding due to compartment damage, etc.

Methods to prevent accidents and to control and mitigate their consequences shall be considered during the design of the topsides structure and in laying out the facilities and equipment so as to minimize the probability of occurrence and the effects of accidental events. Control and mitigation measures, including those to protect safety-critical elements (SCE) against fire and explosion actions, shall conform to ISO 13702.

The facilities layout, including equipment positioning, shall be arranged to minimize the exposure of personnel to accidental events and their consequences. Where the process of risk assessment identifies a hazard, the topsides structure shall be designed to have sufficient capability to resist or contain its effects so as to reduce risks to personnel and the environment to as low as reasonably practicable (ALARP). Robustness and inherent safety are important design considerations. Provision of adequate structural strength and ductile deflection capability as inherent safety measures to enhance the survivability of the platform shall be considered for the design of the topsides structure, equipment and SCE attached to it.

The main load-bearing primary structure, which is one of the SCE and is fundamental to the support of the temporary refuge, the life boats and other components essential to the safety of the personnel shall be designed to retain sufficient integrity during accidental situations and to maintain sufficient integrity to provide

— protection to personnel for a duration sufficient to effect their evacuation, and

— protection to the environment for a duration sufficient to effect containment of hydrocarbon spillages from process equipment.

In addition, the protection of the asset and the costs of failure should be considered with respect to importance to the owner and to the relevant national authorities.

An accidental situation can directly or indirectly impose actions, including drag actions, deflections and strong vibration on structural components and SCE, such as blow-down systems, emergency shut-down systems, deluge systems, processing systems and piping. Structural discipline engineers shall work closely with engineers of other disciplines (mechanical, electrical, process, etc.), including safety engineers experienced in performing hazard analyses, as part of the owner or operator’s safety management system, as described in ISO 13702, to ensure that the likely response of the topsides structure and equipment is suitably assessed.

Interaction between the topsides structure and the support structure shall be considered. For a floating platform, suitable means of structurally decoupling the topsides structure from the hull and deck shall be provided where this is necessary to reduce the internal forces resulting from accidental and abnormal actions to acceptable levels.

The structural system shall be designed to resist accidental actions to ensure that the main safety functions of the topsides structure are not so impaired as to lead to either unacceptable loss of integrity of the structure or escalation causing its partial collapse. ALS design verification shall be carried out by considering, for each accidental situation, a representative value that reduces risks to ALARP. The probability of exceeding this representative accidental situation shall be no greater than about 10^{-4} per year. This probability level can be taken as indicative of an order of magnitude since the data basis for accurate determination of this small exceedance probability can be limited and include considerable uncertainties.

Items of equipment essential to the survival of the topsides (i.e. SCE) shall be assessed for structural resistance to accidental actions. The assessment shall include supports to such equipment and any associated critical pipework.

Imposed and self-weight actions shall be included in the assessment of the integrity of the structure and its response. Partial action and resistance factors may be set to unity for the ALS. The ALS assessment shall also include appropriate consideration of the integrity of the damaged structure (after-damage design situation) in assessing its capability to resist appropriate environmental actions. An assessment is necessary where the resistance of the structure has been significantly reduced by the structural damage caused by the accident. Criteria for this assessment are given in ISO 19902.

7.10.2 Evaluation of accidental situations

7.10.2.1 General

Three risk levels are considered in this part of ISO 19901, as described in Table 3.

Table 3 — Description of risk levels

Risk level	Description
1. Low risk	Insignificant or minimal risks that can be eliminated from further consideration (risk level 1).
2. Medium risk	Risks shown to be acceptable using ALARP criteria (risk level 2).
3. High risk	Risks requiring further mitigation or modification of platform functions or manning philosophy, to reduce the probability of occurrence or the consequences of an event, or both (risk level 3).

The assessment process is intended to be a series of evaluations of specific events that could occur for the selected platform over its intended service life and service function(s).

A risk assessment process, as shown in Figure 1, shall be undertaken as described in this subclause to

a) initially screen those platforms considered to be low risk (risk level 1), thereby not requiring mitigation measures,

b) study the probabilities, accidental action magnitudes and consequences, and the costs of mitigating measures, and

c) determine which mitigation measures are necessary and re-evaluate the risk levels when ALARP criteria cannot be met without mitigation.

The assessment process as detailed in Figure 1 comprises a series of tasks to be performed to identify the risk to a platform from fire or explosion actions, from impact loading, or from compartment flooding and to perform a suitable structural assessment.

The assessment tasks listed below should be read in conjunction with Figures 2 and 3 and with Table 4.

— Task 1: For each accidental situation in turn, estimate the order of magnitude of the probability of the event. Determining the probability of accidental events can be based on a review of statistics of occurrences at other locations, for example the occurrence of leaks from particular types of flanges in pipework coupled with the probability of an ignition, but the limitations and inaccuracies in such estimates should be understood.

— Task 2: For the accidental event, estimate the potential for injury from the descriptions in Table 4 and consequently determine the risk level 1, 2 or 3 for the event under consideration. In many cases, it can be necessary to undertake a structural assessment of the event to determine the potential consequences (see Figures 2 and 3). For events with risk level 1, the assessment is complete and the next accidental event shall be considered.

— Task 3: For risk levels 2 and 3, further studies shall be undertaken to better define the risks, consequences, possible mitigation measures and the costs of those mitigation measures. Mitigation measures can include changes to hardware or to procedures to reduce the likelihood of the event, as well as changes to structure or equipment to reduce the consequences of the event. Having determined and costed possible mitigation measures, the costs shall be compared against the benefit in risk reduction to determine if the ALARP criteria are satisfied.

— Task 4: Where necessary, mitigation measures shall be undertaken and the risk assessment repeated.

7.10.2.2 Probability of occurrence and severity of accidental events

The probability of occurrence of a fire, explosion, impact loading or flooding event is associated with its origin, point of ignition (fires and explosions), source of impact (other accidental events), location of flooding and escalation potential. The type and presence of a hydrocarbon source can also be a factor in event initiation or event escalation. The significant events requiring consideration and their probability of occurrence levels are normally defined from hazard analysis.

Factors affecting the probability of occurrence and severity of an accidental event include the following.

— **Product type**

Product type, i.e. gas, condensate, light or heavy crude, shall be considered in evaluating the probability of occurrence.

— **Operation type**

The types of operations being conducted on the platform shall be considered when evaluating the probability of occurrence. Operations to be considered shall include drilling, production, supply boat operations and personnel transfer. Production operations are defined as those activities that take place after the successful completion of the wells; they include separation, treatment, measurement, transportation to shore, operational monitoring, modification of facilities and maintenance. Simultaneous operations include two or more activities.

— **Decks, ceiling and boundary walls**

The potential of a platform deck, ceiling, wall or other physical barrier to confine a vapour cloud is important. Whether a platform configuration is open or closed should be considered when evaluating the probability of an event occurring. Most platforms in mild environments, such as the US Gulf of Mexico, are open allowing natural ventilation. Platform decks in more severe climates (e.g. Alaska or the North Sea) are frequently enclosed, resulting in increased probability of containing and confining explosive vapours and higher explosion overpressures if ignited.

— **Equipment type**

The complexity, amount and type of equipment are important. Separation and measurement equipment, pump and compression equipment, heating equipment, generator equipment, safety equipment and their piping and valves shall be considered when evaluating the probability of occurrence of an event.

— **Equipment congestion**

Turbulence generated by equipment, structure, piping and cable trays, etc. can cause high overpressures in the event of an explosion with or without the presence of confining boundaries.

— **Platform location**

The proximity of the platform to shipping lanes can increase the potential for collision with passing vessels.

— **Compartmentation**

For floating structures, the increase in draught and the degree of listing following the accidental flooding of one or more watertight compartments can be influenced by the size of the compartments and any common-mode failures in the pipework or controls to the compartments.

— **Other**

Other factors, such as the frequency of supply boat operations, the type and frequency of personnel training, etc., shall be considered, as necessary.

7.10.2.3 Risk assessment

7.10.2.3.1 General

Accidental events of fire and explosion are assigned overall risk levels for a particular platform using Table 4 in accordance with ISO 13702. The risk levels are based on probabilities of accidental events and the likely consequences of those events.

Accidental events for platforms with risk levels 1 and 2 shall be considered as load cases for structural design.

7.10.2.3.2 Risk matrix

The risk level matrix in Table 4 provides a means of determining the acceptability of the risks of particular accidental events and is primarily a means of screening low-risk events that do not need further investigation from those that require more detailed investigation and possibly mitigation measures. Documentation of the screening and detailed assessment processes is required, particularly with regard to the conservatism of the data used and the sensitivity of the results to changes in assumptions.

Table 4 — Indication of risk level for accidental events

Annual frequency of occurrence f		**Possible consequences to human safety and to the environment**					
		C1	C2	C3	C4	C5	C6
		No significant risk to personnel and No significant risk to environment	Lost time injury but No significant risk to environment	Worst of: Serious harm to individuals or Limited loss of oil or chemicals to the sea	Worst of: Single fatality or Major loss of oil or chemicals to the sea	Worst of: Several fatalities (2 to 5) or Major loss of oil or chemicals to the sea	Worst of: Multiple fatalities (>5) or Loss of platform
Frequent	$1 \leq f$	Risk level 2	Risk level 2	Risk level 3	Risk level 3	Risk level 3	Risk level 3
Occasional	$10^{-1} \leq f < 1$	Risk level 1	Risk level 2	Risk level 3	Risk level 3	Risk level 3	Risk level 3
Infrequent	$10^{-2} \leq f < 10^{-1}$	Risk level 1	Risk level 2	Risk level 2	Risk level 3	Risk level 3	Risk level 3
Rare	$10^{-3} \leq f < 10^{-2}$	Risk level 1	Risk level 1	Risk level 2	Risk level 2	Risk level 3	Risk level 3
Very rare	$10^{-4} \leq f < 10^{-3}$	Risk level 1	Risk level 1	Risk level 1	Risk level 2	Risk level 2	Risk level 3
Improbable	$f < 10^{-4}$	Risk level 1	Risk level 1	Risk level 1	Risk level 1	Risk level 2	Risk level 2

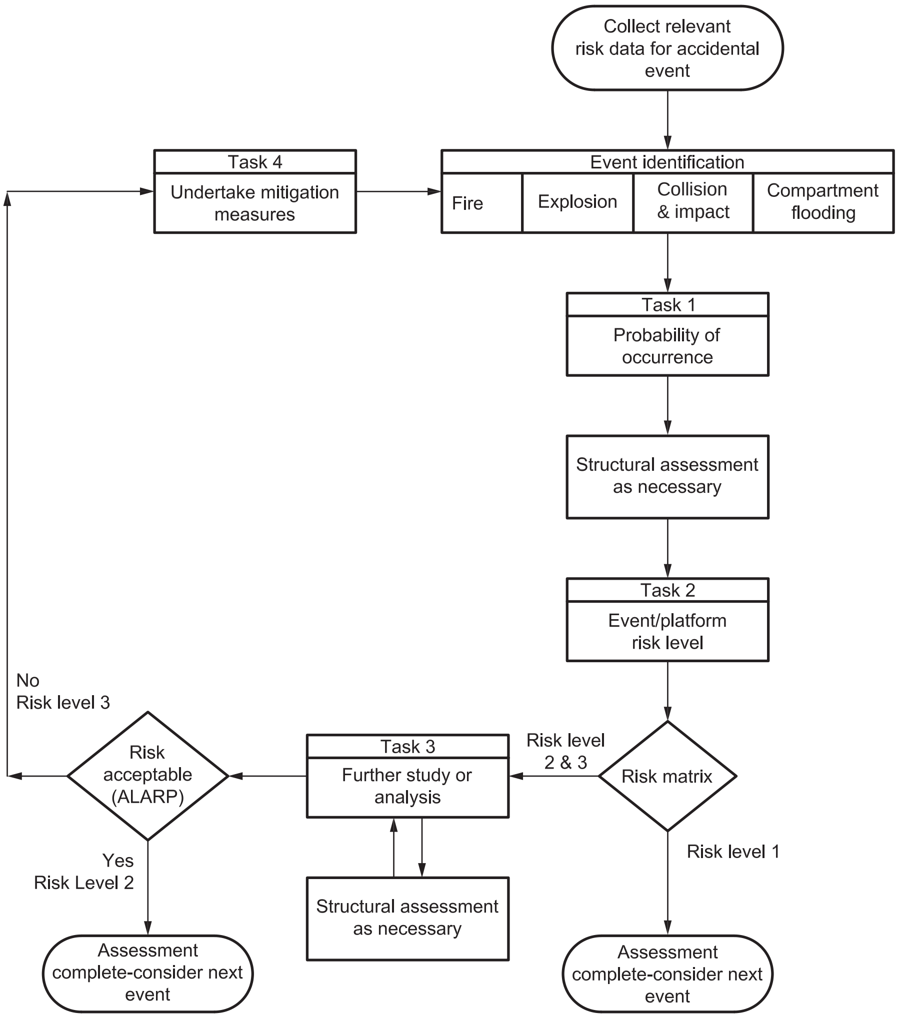
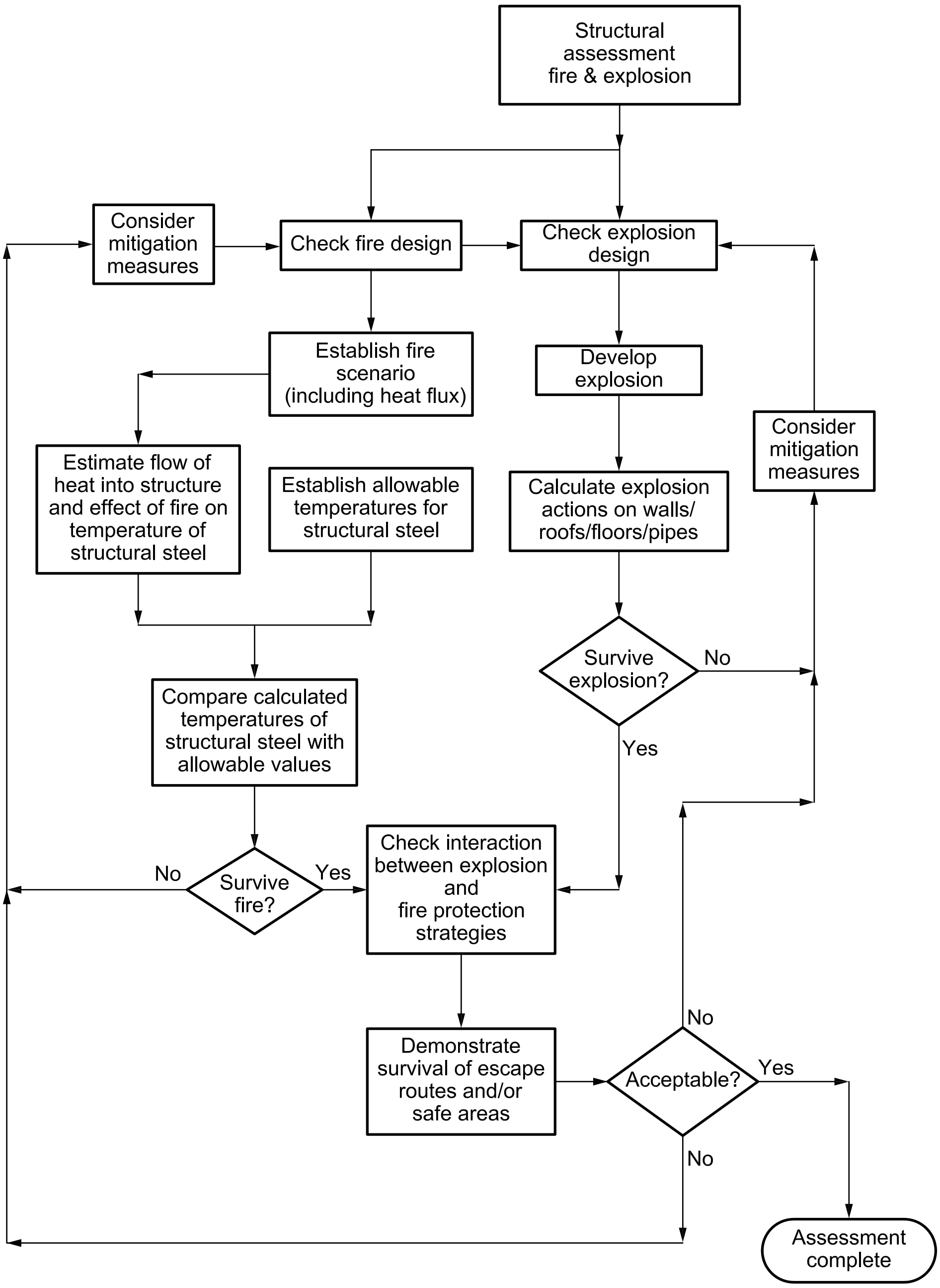
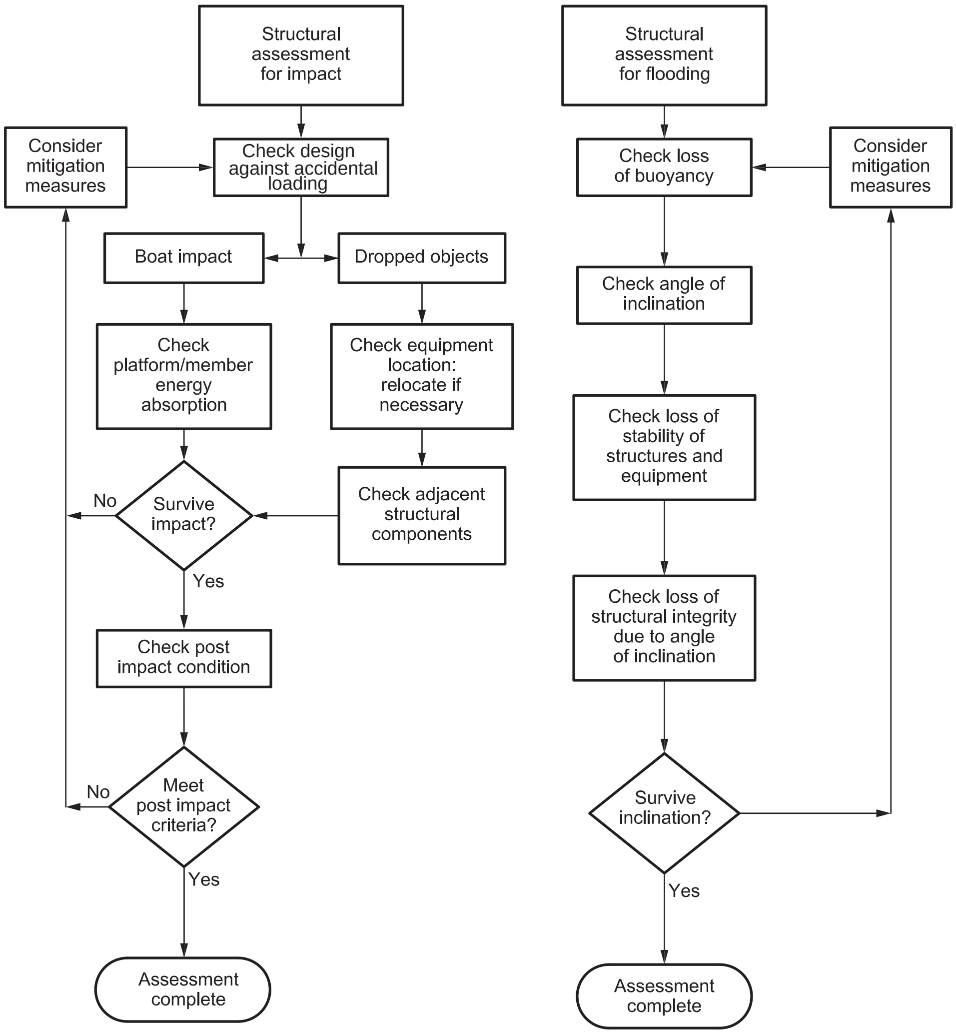


Figure 1 — Assessment of accidental events



NOTE Survival indicates no loss in serviceability of SCE.

Figure 2 — Detailed structural assessment for fires and explosions



NOTE Survival indicates no loss in serviceability of SCE.

Figure 3 — Detailed structural assessment for impact and accidental flooding events

7.10.3 Hydrocarbon incidents

Explosion and fire events can lead to equipment damage, or to partial or total collapse of topsides and other structures, or to both damage and collapse, resulting in loss of life, or in environmental pollution, or in both loss of life and environmental pollution.

Designing topsides to control the risks associated with explosions and fires requires a multi-disciplinary approach to developing and implementing a suitable safety management process. Steps in this process can include

— defining global systems and component performance standards for the topsides SCE,

— assessing the probability of hydrocarbon leaks and minimizing them,

— assessing the probability of ignition sources and minimizing them,

— optimizing the layout of equipment and structures to minimize the severity of potential explosion actions, or fire actions, or both explosion and fire actions,

— considering the use of mitigation systems to minimize the severity of potential explosion actions, or fire actions, or both explosion and fire actions,

— quantifying the potential explosion and fire design actions,

— designing SCE and structures with the necessary inherent safety to satisfy predetermined performance criteria, and

— demonstrating by suitable and sufficient fire and explosion assessments that safe areas exist and that sufficient escape routes are available to satisfy the performance criteria to survive any design accidental action.

The operator or owner shall define their risk acceptance criteria in advance of assessment and analysis.

NOTE In many cases, the steps listed above are undertaken by fire and explosion specialists in association with laying out the facilities.

7.10.4 Explosion

In many cases, particularly for larger and more complex platforms in hostile environments, it is not practicable to design the topsides to withstand the highest conceivable explosions that could occur and, consequently, a balance should be found between the probability of explosion of differing magnitudes and the provision of sufficient resistance to withstand the explosion.

Explosion scenarios shall be developed as part of the process hazard analysis. Assessment of explosions shall be performed in accordance with ISO 13702. For each topsides area, an exceedance curve should be drawn, showing the probability of a explosion overpressure exceeding a particular value. The explosion overpressure at the limit of significant probability, often taken as a probability of 10^{-4} per year, should be the minimum value used for design explosion overpressure.

Five major, controllable parameters influence explosion overpressure. These are

— confinement by walls, decks and larger equipment,

— congestion due to equipment, piping, structure and cable trays,

— size of combustible gas-air cloud formed by the hydrocarbon release,

— composition and concentration of the gas-air cloud formed by the hydrocarbon release, and

— location of ignition.

As part of the detailed explosion assessment process described in Figure 2, confinement shall be suitably represented, congestion shall be sufficiently detailed and representative gas-air clouds (including variation in location of ignition source within the cloud) shall be used. The latter requirement poses the largest challenge. Two possible approaches are to use:

a) worst-case gas clouds containing stoichiometric mixes, where it is certain or at least highly probable that the resulting actions are conservative;

b) a distribution of gas clouds with associated probabilities, where the resulting actions and their probabilities can be presented as a series of curves showing a range of overpressures with associated probabilities: this approach is particularly suitable when probabilistic acceptance criteria are set.

The explosion assessment shall demonstrate that the escape routes and safe areas survive.

7.10.5 Fire

If the assessment process identifies that a significant risk of fire exists, fire should be considered as a load case. Fire scenarios shall be developed as part of the process hazard analysis. Fire shall be considered a design accidental event if assessment identifies that the probability of a significant fire is greater than about 10^{-4} per year. Fire as a design accidental event may be treated using the techniques presented in ISO 13702.

The structural assessment shall demonstrate that the escape routes and safe areas are maintained to allow sufficient time for platform evacuation and emergency response procedures to be implemented.

7.10.6 Explosion and fire interaction

Fires and explosions can both occur during the same overall event, for example a leak can cause a gas cloud to form which can explode when it meets an ignition source; following the resulting explosion, the original leak can remain but as a fire. It is less likely that an explosion follow a fire unless additional inventory is released due to the fire. The explosion and fire analyses shall be considered together and the effects of one on the other shall be carefully analysed. Examples of explosion and fire interaction can be found in A.7.10.6.

7.10.7 Vessel collision

If an assessment identifies that a significant (i.e. potentially unacceptable) risk of vessel collision exists, then the effects on the topsides structure and equipment of a ship impacting the support structure shall be taken into account. This is particularly important where a module support frame or integrated deck contributes to the reserve strength of the platform as a whole. The effects of the resulting strong vibration shall also be investigated, as described in 7.10.10.

7.10.8 Dropped and swinging objects and projectiles

Certain locations, such as in and around crane lay-down areas, are more likely to be subjected to dropped or swinging objects. The probability of occurrence can be reduced by training and by following safe handling practices.

The consequences of damage can be minimized by considering the location and protection of facilities and critical platform areas. Operational procedures should limit the exposure of personnel to overhead material transfer.

The platform shall survive the impact from dropped objects being lifted in accordance with operational procedures (i.e. within crane limits and following any restrictions on heights of lifts above decks).

The potential impact depends on the functional activities taking place on the topsides. An assessment shall be carried out to determine the cumulative probability of occurrence. Events that can cause unacceptable effects, but cannot be disregarded due to their frequency of occurrence, shall be considered design accidental events. Design measures shall be taken to protect the platform's safety functions against any such accidental events or the activities shall be modified to reduce the probability of occurrence.

7.10.9 Loss of buoyancy

For floating structures, there are minimum criteria to be considered for compartment damage (see ISO 19904-1). The effect of any resulting inclination (heel and trim) shall be checked to ensure that there is no loss of stability or integrity of the structure, including supports to equipment.

7.10.10 Strong vibration

Strong vibration shall be considered explicitly an accidental event in the design of new exposure level L1 and L2 platforms and in the assessment of existing exposure level L1 and L2 platforms. The causes of strong vibration can include accidental shock actions from gas explosions, ship collision, helicopter emergency landing or crash, sudden failure of loaded cables, extreme weather and seismic events. The response of offshore structures to seismic events is considered separately in 7.8.

Vibrations can propagate from the site of an initiating event through the structure to affect other parts of the topsides, such as the flare boom, drilling derrick and helideck, as well as the platform's safety systems. Failure or excessive deformation of individual components that could lead to failure of safety-critical systems shall be considered. Safety systems that can be vulnerable to damage from strong vibration include emergency shutdown systems, emergency power supplies and communications, fire and gas detection systems, fire protection systems and evacuation, escape and rescue equipment.

Pipework and risers can leak when subjected to excessive relative deflections between modules or to large dynamic actions. Module supports and their connections can fail when subjected to large horizontal actions. Connections of equipment items to their support structures can fail. Control equipment, including computers and microprocessors, and telecommunications equipment, can be particularly susceptible to high-frequency vibration.

Supports for pipework, cables and equipment can be liable to large deformations or collapse. Severe deformations in the accommodation, escape routes or at the helideck can impair evacuation, escape and rescue.

Modifications or remedial work to increase the robustness of the topsides structure, including strengthening support structures and access platforms, and their connections to reduce the effects of shock shall be considered.

7.11 Other actions

7.11.1 Drilling operations

Drilling operations result in actions transmitted from a derrick to the topsides. Those most commonly occurring are described below.

a) Drilling actions: sufficient drill rig positions shall be considered in combination with appropriate environmental actions to ensure that all maximum forces in the supporting structures are identified. Reverse environmental actions with associated minimum actions shall also be considered to check stability and uplift.

b) Increased action from the drilling derrick when it is required to pull on a stuck drill string shall be considered.

c) Actions from skidding the derrick substructure over the skid base and the skid base over any integrated deck skid beams shall consider sufficient locations in order to allow for the maximum stresses resulting from all possible relative positions of the structures to be evaluated. Horizontal frictional forces and bearing stresses shall be checked, including racking components that can result from a stuck jack.

d) Under extreme environmental conditions certain drilling operations are suspended. This constraint may be taken into account in assessing the appropriate combinations of actions.

e) Certain drilling operations require the temporary support of heavy drill strings from the floor of the supporting structure. The need to support such actions shall be identified and considered.

7.11.2 Conductors

Supports for conductors shall allow for (axial) movement from thermal growth (including the effects of thermal growth of the well string) and differential settlement of the platform and conductor. Radial movement of the conductors within the guides should be minimized to reduce the effects of lateral movements and impacts. Actions from waves and from current and flow-induced vibrations on conductors can be reacted at the cellar deck guide and these actions shall be taken into account. Drilling operations can require the temporary support of vertical actions from conductors at the cellar deck. The need to support such actions shall be identified and considered.

7.11.3 Risers

In addition to normal actions due to weight, supports for risers shall take into account possible actions resulting from waves, current, flow-induced vibrations, thermal growth and the dynamic reaction to slugs of fluid moving in the risers. Radial movement of the risers within the guides should be minimized to reduce the effects of lateral movements and impacts.

7.11.4 Caissons

In addition to normal actions due to weight, supports for caissons shall take into account possible actions resulting from waves, load variations from pump reaction and fluid contents, and the particular actions associated with offshore erection or completion. The effects of internal/external corrosion are a major cause of caisson failure and shall be accounted for. Radial movement of the caissons within the guides should be minimized to reduce the effects of lateral movements and impacts.

7.11.5 Maintenance, mechanical handling and lifting aids

Design of the structure shall take into account the actions resulting from equipment maintenance, variable actions from hydrostatic pressure testing and mechanical handling. In particular, the principal routes and means for moving heavy equipment shall be identified and the supporting structures assessed to ensure that these actions, in combination with those normally acting, do not result in unacceptable combinations. Care shall be taken to ensure that the actions from trolley wheels do not locally yield deck plate, resulting in ponding.

Where lifting aids (runway and lifting beams, padeyes, etc.) are attached to a primary or secondary structure, the effects on the strength and stability of the structure shall be considered. Particular attention shall be paid to the potential negative influence on local stability of webs and flanges caused, for instance, by differential displacements of beam supports.

7.11.6 Bridge supports

Topsides structures can be required to carry permanent and variable actions from bridges to other structures, including temporary construction, drilling and other equipment.

The following potential actions shall be considered:

a) variation in point of application resulting from extreme tolerances in platform position;

b) actions resulting from installing and removing temporary bridges;

c) actions resulting from any bridge-imposed constraints on differential displacement of linked platforms in all degrees of freedom, giving consideration to in-phase and out-of-phase wave actions at different wave periods;

d) actions from the differential constraint of any pipework carried over the bridge;

e) any jacking actions that can be applied during maintenance operations, such as bridge-bearing change-out.

8 Strength and resistance of structural components

8.1 Use of local building standards

The general requirement in the ISO 19900- series is that for the structure as a whole, and for each component:

$$R_{\mathrm{D}} \geq S_{\mathrm{d}} \tag{7}$$

or

$$\frac{R_{\mathrm{K}}}{\gamma_{\mathrm{R}}} \geq S_{\mathrm{d}} \tag{8}$$

where

R_{D} is the design value of the resistance;

S_{d} is the design value of the internal force or moment due to factored actions;

R_{K} is the representative resistance of the structure or component;

γ_{R} is the partial resistance factor.

This part of ISO 19901 allows the use of representative actions, F_{r}, with the partial action factors, γ_{f}, from other International Standards in the ISO 19900- series, to determine the design value of the internal force or moment, S_{d}. It also allows the use of any appropriate national or regional building standard or classification society rules for the derivation of the representative component resistances, R_{K}, in conjunction with appropriate partial resistance factors, γ_{R}.

The balance between the partial action factors and the partial resistance factors differs between the various building standards and the ISO 19900- series. Therefore, for topsides structures design and assessment, Formula (8) is modified to become:

$$K_{\mathrm{c}} \times \frac{R_{\mathrm{K,code}}}{\gamma_{\mathrm{R,code}}} = \frac{R_{\mathrm{K,ISO\ 19902}}}{\gamma_{\mathrm{R,ISO\ 19902}}} \geq S_{\mathrm{d}} \tag{9}$$

where K_{c} is a building code correspondence factor that matches the design resistance(s) of the building standard to the design resistance(s) of ISO 19902.

K_{c} can be different for the various national and regional building standards or classification society rules and shall be evaluated to give equivalent reliability to that implicit in the ISO 19900- series.

A procedure which may be used to determine a value of K_{c} is given in A.8.1. Annex C gives values that may be used with certain building standards.

8.2 Cylindrical tubular member design

The requirements for the design of cylindrical tubular members shall be taken from ISO 19902 or from the national or regional standard for buildings or bridges that is being used to design other topsides components.

8.3 Design of non-cylindrical sections

8.3.1 General

The strength equations to be used in the design of non-cylindrical structural shapes shall be taken from the national or regional standard being used in conjunction with the appropriate partial action factors (see 7.3).

8.3.2 Plate girder design

Plate girders shall be designed in accordance with a suitable code of practice (see A.8.3.2). Design methodologies based on developing tension fields in the web of the plate girder shall not be used where large penetrations would inhibit such development or where the necessary anchorages at the ends of plate girders do not exist (e.g. due to lack of plating continuity through legs). Where stress concentrations, such as abrupt changes in section, penetrations, jacking slots, etc. are necessary, their effect on fatigue and fracture shall be considered. Unstiffened plate girders shall have web thicknesses of not less than 1,25 % of the web depth or 6 mm, whichever is greater.

8.3.3 Box girder sections

Box girders shall be designed in accordance with a suitable code of practice (see A.8.3.3). Particular attention shall be paid to internal stiffening of the box girder with diaphragms or other components to facilitate a cost-effective fabrication and to mitigate the effects of welding and stress-induced distortions.

Specific issues that shall be addressed in design include

— the effect of high local bending forces,

— warping,

— distortion,

— distortional warping, and

— the effect of shear lag upon the distribution of elastic stresses.

8.3.4 Stiffened plate structures

The webs of longitudinally stiffened plate girders and box girders shall be designed in accordance with an appropriate standard (see A.8.3.4).

8.3.5 Stressed skin structures

Stressed skin structures may be designed on the basis that the plating resists shear forces only and that all axial forces are carried by the framing (see A.8.3.5). If the stressed skin structure is exposed to cyclic actions, the possible detrimental effects from repeated buckling shall be considered.

8.4 Connections

8.4.1 General

Connections shall be designed in accordance with the same national or international building code as used for the topsides structural components, except for connections situated between cylindrical tubular members, where the provisions of ISO 19902 shall apply.

Connections should be designed to transfer the full strength of the adjoining members, unless structural releases are part of the design. It shall be demonstrated that the ductility associated with the failure modes of the connection is acceptable.

8.4.2 Restraint and shrinkage

Design details shall minimize any constraint of ductile behaviour and excessive concentration of welding. Details shall allow simple access for the placing of weld metal.

Connections shall be designed so as to minimize, insofar as practicable, stresses due to the contraction of the weld metal and adjacent base metal upon cooling. Particular care is required where shrinkage strains in the through-thickness direction can lead to lamellar tearing in highly restrained connections. See A.8.4.2.

8.4.3 Bolted connections

Bolted connections can be safe and efficient, but have historically been avoided in the design of new primary topsides structure because of concerns about crevice corrosion. Where structures are not exposed to direct contact with sea spray, and where welding is undesirable, bolted connections can be satisfactory.

A bolted connection can form an extremely simple and cost-effective connection for the modification of existing topsides. With appropriate safety procedures, bolted connections can be effected on live production platforms without a hot work permit.

In general, bolted connections designed in accordance with any normal building code are satisfactory (see A.8.4.3). High-strength friction grip bolted connections shall be used whenever the connection is expected to experience significant and sustained fluctuating stresses.

When bolted connections are used, the following requirements and recommendations apply.

— The effective length of the bolt (between the underside of the head and the nut) shall be sufficiently long to minimize the consequence of creep in reducing the tension in the bolt.

— The reduction in pretension due to creep shall be calculated and shall include a creep equivalent to 50 % of the initial pretension. Care shall be taken to ensure that bolt hole edges are rounded and that the connection is completely sealed with a durable sealant, or a durable paint system, or both.

— Washers shall be used beneath both bolt heads and nuts to minimize coating damage.

— Corrosion protection of bolted connections shall receive appropriate attention. All surfaces that are in contact or inaccessible after assembly should be coated in aluminium spray. Bolts shall be protected by a high-protection system; galvanizing can be used in many cases but can be unsuitable for high-strength bolts. Cadmium-plated bolts shall be avoided as they can emit a lethal toxic fume when heated.

— Regular inspection of bolted connections should be specified.

8.5 Castings

Castings may be used in place of otherwise complex fabricated components (e.g. padears, supporting structures, transition components, etc.). The design of complex geometries requires the use of suitable numerical analysis and specifications for both design and manufacture shall be prepared. The specifications shall address acceptance criteria for stresses and for the extent of plastic strain in regions above nominal yield, differentiating between stresses within and outside bearing areas. In complex castings subjected to significant fatigue actions, an evaluation of the local peak stress shall be made to evaluate the fatigue performance of the component.

The material properties of the casting shall be compatible with the adjacent materials to ensure weldability and avoidance of corrosion. Weldability tests shall be undertaken prior to the casting being manufactured.

9 Structural systems

9.1 Topsides design

9.1.1 General

The topsides shall be investigated for all appropriate design situations and load cases. Permanent and variable actions may be treated as a series of discrete action combinations representing the range of anticipated platform operations, taking account of variable area actions and skid beam reactions.

Structural analysis of the topsides shall include an adequate representation of the support structure to ensure that the effects of the support structure stiffness are incorporated and that any support structure actions (e.g. environmental) transferred to the topsides are included.

9.1.2 Topsides on concrete support structures

In analysis, particular attention shall be paid to the interaction between steel topsides and concrete support structures (see 7.6). Depending on the details of this interface, the deck can comprise part of a portal frame resisting environmental actions and can be subject to internal actions due to differential movements. In general, the support structure designer should design the steel to concrete connection, and an overlapping interface within the body of the primary topsides structure should be agreed between the topsides and support structure designers.

Attention should be given to the response of the integrated platform to wave actions and the magnitude of any fatigue-inducing actions introduced into the topsides shall be assessed. The subdivision of a topsides structure into small modules can reduce wave-induced stresses.

Concrete exhibits significant creep under sustained actions and has an elastic modulus that varies significantly with time. The topsides designer should therefore seek specific advice from the support structure designer on the values of elastic modulus that should be used in any analyses.

The assumptions shall be communicated to those subsequently responsible for platform operations such that significant variations in the actions shall be cause for assessment.

9.1.3 Topsides on floating structures

Particular attention shall be paid to the interaction between topsides and hull structures for mobile and floating structures. Deformations of the hull under environmental actions and varying cargo and ballast conditions can be significant and shall be considered in the design of supports. The use of sliding or elastomeric bearings at the topsides/hull interface can be required.

9.2 Topsides structure design models

9.2.1 General

Internal forces in structural components are normally derived using an indeterminate, three-dimensional structural analysis methodology. In general, a linear-elastic model of the topsides structure is sufficient. The stiffness of the support structure can influence the distribution of forces within the topsides.

9.2.2 Support structure model for topsides design

The support structure shall be modelled in sufficient detail to ensure that simplifications of the boundary conditions for the support structure do not influence the topsides behaviour. It can be necessary to include the foundations of bottom-founded support structures in this model. This model may be simplified, but shall be sufficient to represent the vertical and horizontal stiffness of the support structure together with its inertial response and major variations in operational action effects.

9.2.3 Topsides model for topsides design

The topsides structure can be modelled as one or more independent structures to represent the actual sequence of fabrication and installation. The interaction between the separate structures shall be considered and differential deflections allowed for where these can have a significant effect on the performance of the structures, particularly for serviceability conditions. The sequence of stressing introduced into rigid jointed structures arising from fabrication and erection shall be considered.

Where the model is used to represent pre-service conditions, appropriate boundary conditions shall be applied to represent the stiffness of the supports and, for transportation conditions, appropriate accelerations and displacements shall be applied.

9.2.4 Modelling for design of equipment and piping supports

Local supports for piping and individual items of equipment can generally be analysed in isolation. However, where relative displacements and deflections between pieces of equipment or between modules could affect the integrity of the interconnecting pipework; such pipework and its supports shall be checked for the effects of interaction. For piping bridging between modules, deflections due to environmental, accidental and seismic actions shall be considered.

9.3 Support structure interface

9.3.1 Responsibility

All assumptions at the interface between the topsides and support structures shall be documented to ensure that the designs are not compromised by any differences in assumptions.

9.3.2 Static strength design

Care shall be taken to ensure that the governing conditions for the connection of the topsides to the support structure are correctly identified. The governing case for this connection can be the requirement to ensure global integrity for the whole platform under accidental actions. The design shall be checked to ensure that simplifying assumptions at the interface do not mask the most onerous condition for the connection.

9.3.3 Fatigue design

In any circumstances in which the internal forces in the connection between the topsides structure and the supporting structure vary significantly, a fatigue analysis shall be undertaken. The structural models to be used for fatigue analyses shall reflect all stiffnesses at their expected values. Significant stiffnesses from non-structural components like piping, cladding, etc. shall generally be considered in the model. Stiffness contributions, such as grouted legs, corrosion allowance and soil stiffness, shall be considered with their expected values. If the stiffness is uncertain, parametric studies shall be undertaken to establish a safe estimate. For floating and compliant supporting structures, the fatigue analysis shall include the effects of the distortions of the supporting structure, the stiffnesses of the supporting points and the changes in the direction of actions due to the motions and deformations of the supporting structure.

9.4 Flare towers, booms, vents and similar structures

This subclause gives requirements that apply to separate structures where varying actions constitute a major proportion of total actions.

Flare towers, booms and other structures can be susceptible to global and local resonant responses due to

— global and local wind actions,

— thermal actions from the flare including thermal cycling,

— seismic actions,

— accidental actions, and

— the indirect effects of wave and current actions on the support structure.

Global modes of vibration can be due to vortex-induced vibrations of major components of the structure, or pipework supported by it, or both. The structures shall be checked to establish the natural modes of vibration of the structure as a whole and of critical components of the structure. Guidance on such structures is given in A.9.4.

The static and dynamic behaviour of such structures can be substantially influenced by accumulations of snow and ice and these shall be taken into consideration for the design. Both wind actions and variable actions can be increased by such accumulations. These shall be combined with actions generated by wind having a representative return period of, typically, 10 years.

The thicknesses and densities of snow and ice accumulations shall be determined from site-specific environmental data. Simplified build-up profiles for calculation purposes may be applied.

The designer of the flare boom and the designer and manufacturer of the flare tip should liaise to ensure compatibility between their designs and maintainability of the flare tip.

9.5 Helicopter landing facilities (helidecks)

9.5.1 General

The design shall meet the requirements of the regulating authority for aviation in the region in which the platform is to be installed.

The helicopter landing and take-off area and any parking area provided shall be of sufficient size and strength and laid out so as to accommodate the greatest size and greatest dimensions of any helicopter anticipated to be used.

The helideck structure shall be designed to resist, without disproportionate consequences, the impact from an emergency landing of a helicopter anywhere within the designated landing and take-off area. Whether or not a designated parking area is provided, the design shall allow the design helicopter to be parked anywhere on the accessible helideck surface. Both the ULS and SLS shall apply.

Environmental conditions around the helideck, particularly wind flow and turbulence affected by adjacent structures, equipment and process plant, can influence the actions on, and controllability of, helicopters during landing and take-off and shall be considered (see A.9.5.1).

9.5.2 Construction

The helideck and its supporting structure may be fabricated from steel, aluminium alloy or other suitable materials and shall be designed and fabricated to appropriate standards. Where differing materials are used, the detailing of the connections shall be such as to avoid galvanic corrosion.

The landing surface shall be watertight and have a minimum slope of 1:100. To prevent any environmental impact of spilt aviation fuel, the helideck shall drain all rainwater and other liquids to the closed drain system on the platform.

9.5.3 Design actions and resistances

9.5.3.1 Design situations

Helicopter actions shall be applied together with other permanent, variable and environmental actions as appropriate. Combinations of actions shall be evaluated for

— an emergency landing situation (see 9.5.3.3), and

— a helicopter at-rest situation (see 9.5.3.4).

9.5.3.2 Design requirements

The structure shall be designed for the SLS and ULS conditions appropriate to the structural component being considered, as follows:

— deck plate and stiffeners:

  — ULS under all conditions;

  — SLS for permanent deflection following an emergency landing;

— helideck supporting structure:

  — ULS under all conditions;

  — SLS.

The supporting structure, deck plate and stringers shall be designed to resist the effects of local wheel or skid actions, acting in combination with the other permanent, variable and environmental actions. Helicopters shall be considered to be located on the deck so as to maximize the internal forces in the component being considered.

Deck plate and stiffeners shall be designed to limit the permanent deflection (deformation) under helicopter emergency landing actions to no more than 2,5 % of the clear width of the plates between supports.

Webs of stiffeners shall be assessed locally under wheels or skids and at the supports, so as not to fail under landing gear actions due to emergency landings.

Tubular structural components forming part of the supporting structure shall be checked for vortex-induced vibrations from wind.

9.5.3.3 Helicopter emergency landing situation

The following actions shall be included in helicopter emergency landing situations.

a) **Helicopter dynamic actions** (undercarriage local actions)

   The dynamic helicopter landing action shall be taken as the design collapse load of the undercarriage and shall be increased by a structural response factor to account for the sympathetic response of the helideck structure. The factor to be applied for the design of the helideck framing depends on the natural frequency of the deck structure. Unless values based upon particular undercarriage behaviour and deck frequency are available, a minimum structural response factor of 1,3 shall be used.

b) **Area-imposed actions**

   A general area-distributed action of 0,5 kN/m^2 shall be applied to allow for minor equipment left on the helideck and for any snow and ice actions.

c) **Permanent and variable actions on the helideck structure and fixed appurtenances**

The self-weight of the helideck structure and fixed appurtenances supported by each structural component concerned shall be evaluated.

Concentrated horizontal imposed actions equivalent in total to half the maximum take-off weight (MTOW) of the helicopter shall be applied at the locations of the main undercarriages and distributed in proportion to the vertical actions at each point. These shall be applied at deck level in the horizontal direction that will produce the most severe load case for the structural component being considered.

d) **Environmental actions**

1) **Wind actions on the helideck**

Wind actions on the helideck structure shall be applied in the direction which, together with the horizontal imposed actions, produces the most severe load case for the structural component considered. The wind speed to be considered shall be that restricting normal (non-emergency) helicopter operations at the platform. Any vertical (up or down) action on the helideck structure due to the passage of wind over and under the helideck shall be considered.

2) **Inertial actions due to platform motions**

The effect of accelerations and dynamic amplification arising from the predicted motions of the fixed or floating platform in a storm condition with a 10-year return period shall be considered.

9.5.3.4 Helicopter-at-rest situation

The following actions shall be included in helicopter-at-rest situations.

a) **Helicopter static actions** (undercarriage local actions)

All parts of the helideck accessible to helicopters shall be designed to support an action equal to the MTOW of the helicopter at any location. This shall be distributed at the undercarriage locations in proportion to the position of the centre of gravity of the helicopter, taking account of possible different positions and orientations of the helicopter.

b) **Area-imposed actions**

To allow for personnel, freight, refuelling equipment and other traffic, snow and ice, rotor downwash, etc., a general area imposed action of 2,0 kN/m^2 shall be included.

c) **Horizontal actions from a tied-down helicopter, including wind actions**

Each tie-down shall be designed to resist the calculated proportion of the total wind action on the helicopter imposed by a storm wind with a minimum one-year return period.

d) **Self-weight of helideck structure and fixed appurtenances**

e) **Environmental actions**

1) **Wind actions on the helideck**

The 100-year return period wind actions on the helideck structure shall be applied in the direction which produces the most severe load case for the structural component considered.

2) **Inertial actions due to platform motions**

The effect of accelerations and dynamic amplification arising from the predicted motions of the fixed or floating platform in a storm condition with a 10-year return period shall be considered.

9.5.3.5 Representative strengths and partial resistance factors

Strength formulations shall be taken from ISO 19902 or the applicable national or regional building standard using appropriate partial resistance factors together with the building code correspondence factor, K_c.

9.5.3.6 Safety net arms and framing

Safety nets for personnel protection shall be installed around the landing area except where adequate structural protection against falls already exists. The safety net shall be strong enough to withstand and contain without damage a 100 kg weight being dropped from a height of 1 m. Where lateral or longitudinal supporting bars are provided to support the net structure, these should be arranged and constructed to avoid causing serious injury to persons falling on them.

9.5.3.7 Helicopter tie-down points

Sufficient flush fitting tie-down points shall be provided for the types of helicopter for which the landing area is designed. These should be so located and be of such construction as to secure the helicopter in severe weather conditions. They shall take into account the inertial forces resulting from any movement of the platform (see 9.5.3.4).

9.5.4 Reassessment of existing helidecks

Alternative requirements and criteria to those specified in 9.5.1 to 9.5.3 for the type of aircraft governing the design may be adopted provided they are approved by the regulatory authority for aviation in the region in which the offshore platform is located. Any alternative requirements and criteria shall be derived from a suitable engineering assessment using information from the manufacturer of the design helicopter. A range of potential landing conditions shall be considered, including failure of one engine at a critical point in the take-off or landing, and the likely behaviour of the helicopter's landing gear and the response of the helideck structure.

Provided the helideck was originally designed and constructed to appropriate standards, and if accepted by the appropriate regulatory authority, minor local deformations of the deck plate and stiffeners as a result of emergency and other heavier-than-normal landings that do not affect aircraft safety can be permitted during service, as long as the structural integrity of the helideck as a whole remains intact and no significant pools of flammable liquids (e.g. aviation fuel) can be expected to form in any deformed deck plating.

9.6 Crane support structure

9.6.1 General

The crane support structure comprises the crane pedestal and its connections to the topsides primary steelwork. It does not include the slew ring or its equivalent, or the connections between the slew ring and the pedestal. Advice shall be taken from the crane manufacturer as to the actions likely to be imposed on the crane support structure

Crane support structures shall, where practical, be attached at an intersection of topsides structure primary framing with minimal eccentricities and be connected to a minimum of two main deck elevations. The pedestal shall be included in the analytical model of the primary structure as its stiffness can have a significant effect on load distribution. When located in accordance with this guidance, the crane support performance is generally governed by static actions with negligible dynamic amplification. Such structures are, however, subject to fatigue actions and shall always be checked to ensure that fatigue life is satisfactory for the required service conditions.

The maximum rotation at the top of the pedestal relative to the primary framing should not exceed 1° for the most onerous static load case. Where this criterion cannot be met, the dynamic response shall be checked.

9.6.2 Static design

The following separate cases shall be considered for the design of crane support structures.

a) **Crane working in calm conditions**

$$F_G + F_{rhl} + F_H \qquad (10)$$

where

F_G is the vertical action due to self-weight of the crane;

F_{rhl} is the greater of the representative hook load, including all applicable factors, taken from ISO 19901-6, and the rated load multiplied by the crane manufacturer's dynamic coefficients;

F_H is the horizontal action due to off-lead and side-lead (see below).

b) **Crane working at maximum operating wind**

The maximum operating wind can be different for platform lifts and for sea lifts (i.e. lifts to or from an adjacent vessel), and can also vary depending on the weight being lifted.

$$F_G + F_{rhl} + F_H + F_W \qquad (11)$$

where F_W is the maximum operating wind action on the crane.

c) **Crane at rest, in extreme wind conditions**

For this situation the crane boom may be considered resting in a boom support (where fitted).

$$F_G + F_{W,ext} \qquad (12)$$

where $F_{W,ext}$ is the extreme wind action on the crane.

d) **Crane collapse failure condition**

This situation is included to ensure that in event of a gross overload of the crane, causing collapse of any part of the crane structure (most commonly the boom or A-frame), no damage to the crane support structure is suffered and progressive collapse is prevented.

The crane support structure shall be designed such that its strength exceeds the collapse strength of the crane.

The crane manufacturer's failure curves, for all crane conditions, shall be used to determine the worst actions on the pedestal. It shall be assumed that the maximum lower-bound failure moment of the weakest component will place an upper bound on the forces and moments to which the pedestal can be subjected.

The design moment for the crane failure condition shall be taken as the lower-bound failure moment described above, multiplied by a safety factor of 1,3.

The action factors used with cases a) to d) above shall be those for normal operating conditions (see Clause 7).

For cases a) and b), F_{rhl} shall be selected in accordance with the maximum rated load applicable over the range of maximum and minimum crane radii, for sea and platform lifts.

For cases b) and c), the most onerous wind directions shall be checked.

For floating structures, the effects of roll and pitch on the direction of the forces in the crane support structure shall be accounted for.

The crane pedestal and its components shall be designed to resist safely the forces and moments from the most onerous load case applicable to the prevailing sea state, together with associated off-lead and side-lead actions. These values shall be obtained from the crane manufacturer, and the angles used shall be not less than the values given in Table 5.

Table 5 — Minimum off-lead and side-lead angles

Angle		Description
Type	Minimum value	
Off-lead angle	6°	Angle to vertical of hoist rope measured in the vertical plane containing the pedestal and boom
Side-lead angle	3°	Angle to vertical of hoist rope measured in the vertical plane normal to the plane containing the pedestal and boom

9.6.3 Dynamic design

It is not normal practice for the supporting structure of offshore cranes to be subjected to a dynamic analysis. The process of fatigue design (see 9.6.4) incorporates an average dynamic amplification factor (DAF) for all lifts; this has been found to provide satisfactory results in practice.

A dynamic analysis shall be undertaken where

— the design is unconventional,

— the maximum rotation at the plane of support exceeds the limit of 1° and experience or engineering judgement indicates that the performance of the crane supporting structure can be adversely affected by its dynamic response to actions,

— seismic situations can result in significant accelerations (see 7.8),

— a crane is being replaced by a larger crane than that for which the crane support structure was originally designed, or

— for floating structures, when a crane will be used in circumstances different from that for which it was originally designed, e.g. for a conversion of a trading tanker to an FPSO or to a floating storage and off-loading (FSO) structure.

The dynamic analysis model shall be sufficiently detailed to ensure that the coupled response of the crane and its supports are realistically represented. Any mechanical damping device incorporated within the design of the crane shall be taken into consideration.

Where appropriate, the results of the dynamic analysis may be used to modify the fatigue design process described in 9.6.4.

9.6.4 Fatigue design

The fatigue design shall, where possible, be based on a rational assessment of the planned use of the crane throughout the topsides life. In the absence of such an assessment a fatigue loading spectrum may be used, based on any appropriate national or international code of practice.

The fatigue damage design factor for the crane support structure shall be based on the safety criticality and accessibility for in-service inspection of each component.

All connections in the pedestal shall be designed to minimize stress concentration factors that are likely to result in an excessive reduction of the fatigue endurance of the pedestal.

Off-lead and side-lead actions, wind and other environmental actions are normally insignificant with regard to the fatigue analysis, but shall be assessed on a case-to-case basis.

Special attention shall be given to any environmental action effects on floating structures that can cause fatigue damage to the crane support structures.

The fatigue analysis shall be based on the expected crane usage combined with manufacturer's recommended dynamic coefficients; a minimum of 25 000 lifting cycles shall be considered.

For each cycle of lift, the full stress range in a component resulting from picking up the load, slewing and set-down shall be considered.

The fatigue calculations shall use S-N curves appropriate to the details used, with stresses enhanced by any major geometric discontinuity, e.g. for the presence of a structural opening, taken into account.

9.7 Derrick design

Design of the derrick structure shall be carried out in a manner consistent with the requirements of this part of ISO 19901, taking full account of the type of supporting structure and including all direct and indirect actions due to the supporting structure; all temporary and short-term situations shall be considered, including situations such as jarring. Design standards for derricks are given in A.9.7.

As part of the assessment of accidental actions on a platform, the risk and consequence of collapse of the derrick structure shall be considered.

The reactions at the base of the derrick structure, in all directions, and the need for tie-downs of the derrick structure shall be considered for all conditions including seismic and abnormal storm conditions.

9.8 Bridges

The design of bridges shall account for the design displacements of the structures that they connect. Bridges shall be fitted with longitudinal, lateral or rotational bearings at support points, arranged to both constrain the bridge from movement relative to supporting structures and minimize the transmission of forces into the bridge from displacements of the support points.

The location, level and depth of bridge structures shall take due account of potential hazards to helicopter and supply boat operations.

Where bridges accommodate escape routes, enclosed fire-rated escape tunnels can be required. Any such escape tunnels shall be explosion-rated for credible far-field explosion effects and suitably ventilated during fires.

Bridges that accommodate escape routes shall be suitably sited, or protected from falling objects and debris during accident conditions, or both.

Bridges that can be subjected to explosion actions shall be provided with suitable restraints for both lateral and vertical action effects. Both initial action effects and rebound action effects shall be considered.

Actions, whether direct or indirect, applied to the bridge that can result in vibrations of the bridge or of equipment and piping on the bridge shall be considered.

Where bridge collapse is allowable in certain accidental situations, including fire, the structural arrangement shall be such that the potential collapse modes do not endanger structures and systems that are required to survive these events.

Local support arrangements shall be detailed to accommodate longitudinal movement, including thermal expansion due to fires if applicable.

9.9 Bridge bearings

Bridge bearings shall be designed to accommodate all actions under operating and design accidental situations. At locations where relative movement between bridge and support structure is to be accommodated, the following actions and action effects shall be included:

— fatigue and wear due to fluctuating actions and movements;

— extreme displacements under ULS and ALS situations, including their effect on assessed actions;

— extreme actions under ULS and ALS situations;

— the most severe combination of both translational and rotational tolerances of the bridge supports.

The design and fire protection of bearing systems shall consider inspection and change-out of bearing components in service, including the provision of jacking and lifting points, as necessary.

9.10 Anti-vibration mountings for modules and major equipment skids

Where modules or equipment skids are supported on anti-vibration mountings (AVMs), these shall be designed for the most unfavourable combination of actions and displacements under ULS, extreme environmental conditions and accidental situations, including fire.

Where the AVMs themselves cannot accommodate certain actions, supplementary guidance and restraint systems shall be provided and these shall be dimensioned to withstand the same actions.

Due considerations shall be given to the design and fire protection of AVMs and their support systems for inspection and change-out of components in service.

9.11 System interface assumptions

The design of the topsides involves complex interfaces with process equipment and plant. Liaison with the other technical disciplines shall be undertaken as part of the design process; in particular, the assumptions made by equipment suppliers about the behaviour of the structure shall be verified. There is a significant potential for these interfaces to compromise structural design assumptions, such as:

a) the location of continuous trough drains penetrating deck plate can compromise lateral support of beam flanges;

b) uncontrolled attachments of minor equipment or utilities to fatigue-sensitive structures can increase potential fatigue damage;

c) penetrations in decks or walls can compromise membrane action required to resist accidental actions;

d) the criticality of components of the topsides structure can depend on their interface with the process plant where the consequence of failure can result in the release of hydrocarbons and consequent fire or explosion;

e) components supporting plant or pipework can be exposed to extremely low temperatures from process operations or blow-down in an emergency situation;

f) spillage of damaging fluids, for example liquid nitrogen, is likely to cause cracking of steel plate and underlying supporting steelwork.

These considerations can affect design, material and fabrication (welding).

The design process shall ensure that interfaces are monitored and the results of this process are clearly recorded.

9.12 Fire protection systems

Fire protection is used to protect personnel and safety-critical structure and equipment from the effects of heat for sufficient time to allow evacuation of personnel from the area. Safety-critical structures shall be identified and are likely to include any primary structure that cannot be shown to be structurally redundant, as well as structures supporting walkways, decks and muster areas, etc. that can be used for evacuation.

Active fire protection (water deluge or foam spray) or passive fire protection (cementitious coatings, or intumescent coatings or fire-resistant panels) shall be specified depending on location and use. Where active fire protection is specified, the effects of possible enhanced corrosion rates on structures subjected to wetting during testing shall be considered. Where passive fire protection is likely to be wetted frequently or for long periods, the top (weather) coat shall be designed to withstand such conditions. The passive fire protection shall be designed to withstand the effects of any direct radiation to which it can be subjected, both during normal or upset operating (blow-down) and accidental situations.

Further information can be found in ISO 13702.

9.13 Penetrations

Penetrations and access openings may be included in structural components providing the capacity of the component is not compromised. Openings shall be included to allow access for inspection of the surrounding structure, including stiffeners and reinforcement, where necessary.

The effects of penetrations and cut-outs on both static and fatigue strength shall be considered. Openings may be provided with reinforcement (e.g. lips, single- or double-sided rings), as necessary, designed to carry the internal forces around the opening. Alternatively, the reinforcement can be designed by reference to experimental or numerical data.

9.14 Difficult-to-inspect areas

Consideration shall be given at the design stage to the accessibility of all parts of the structure for inspection, cleaning and coating by appropriate positioning and detailing of structural components in relation to the adjacent structure and equipment.

9.15 Drainage

Areas where ponding can occur shall be minimized and adequately drained. Where there is a potential for such areas to be fouled with oil, adequate provision shall be made for drainage to a closed-drain system. Arrangements for cleaning to eliminate or reduce any hazards to the environment and to health and safety shall be implemented before any discharge to the sea. In areas susceptible to freezing weather, the possibility of ponding on decks and walkways shall be avoided to prevent slipping hazard.

9.16 Actions due to drilling operations

The consequences of operational impulse actions and vibration on SCE, protective coatings, etc. shall be considered at the design stage of the project.

9.17 Strength reduction due to heat

Any possible reduction in the strength and stiffness of structures sited near heat-producing facilities, such as flares and exhaust ducts, shall be avoided. Where it is not reasonably practicable to relocate the structure and associated facilities, suitable design measures and thermal protection shall be provided and the resultant effects on the structure considered.

9.18 Walkways, laydown areas and equipment maintenance

Walkways and access ways shall be designed to support a variable action for personnel access of 5 kN/m^2 for the design of the grating or plating and for the design of the supporting structure, but the total allowance for the variable actions due to personnel, their personal effects and hand tools need not exceed 1,5 kN per individual of the maximum number of persons on the platform.

Laydown areas for the storage of containers and other equipment transported to the platform shall be designed with sufficient capacity to support the functioning of the platform. The total laydown requirement and arrangement is dependent on the size, functions and manning of the platform. A laydown control system shall be operated to ensure that no laydown area can become inadvertently overloaded. Each laydown area shall be designed to withstand impact from a dropped object, the impact energy of which shall be derived from the lift height and the maximum capacity of the cranes serving the laydown area; it shall be assumed that all the impact energy is applied at one point.

Maintenance areas shall be provided adjacent to any equipment likely to require heavy maintenance. The size and weight capacity of the maintenance area depends on the nature of the equipment and the size and weight of any components that can be required to be removed and replaced.

All laydown and maintenance areas shall be clearly marked with signage to show the maximum laydown capacity and such information shall also be presented in the platform operating manual.

9.19 Muster areas and lifeboat stations

In addition to the uniform allowance for variable actions on walkways and access ways, a higher allowance can be required at muster areas, lifeboat stations and other locations at which personnel can congregate during an emergency. The total variable action for personnel in these areas should allow for at least twice the number of persons for which the muster area, lifeboat station or other escape equipment is intended. Lifeboat supports should be designed to withstand the full capacity of the lifeboat davits or other supporting system.

10 Materials

10.1 General

Most offshore platforms have topsides structures fabricated from carbon steels and this practice is expected to continue. However, stainless steel, aluminium, fibre-reinforced composites and timber have all been successfully used in offshore structures and some considerations on their use are given in this clause. ISO 19902 provides detailed requirements for structural steel for offshore use.

Where a new material that has not previously been used for a particular function is considered, it shall be carefully evaluated with, as a minimum, the following issues being addressed:

a) strength, toughness, stiffness and durability;

b) behaviour at elevated temperatures: flammability, surface spread of flame, emission of smoke and toxic combustion products;

c) resistance to environmental degradation, including various forms of corrosion;

d) compatibility with other materials (e.g. the risk of galvanic corrosion for a specific application);

e) consequential effects on other parts of the topsides, e.g. increased deflection resulting in higher pipe stresses;

f) resistance to fatigue;

g) maintenance requirements;

h) weight;

i) whole life-cycle cost;

j) availability of material of consistent quality complying with recognized standards supported by reliable certification.

10.2 Carbon steel

The selection of carbon steels for the topsides structures shall follow the requirements of ISO 19902 for materials. Two methods are presented in ISO 19902 for determining the steel specifications required together with associated welding, fabrication and inspection requirements. These two methods are referred to as

— the material category (MC) method;

— the design class (DC) method.

For both methods, the requirements for welding, fabrication and inspection of carbon steel in ISO 19902 shall be followed. For the material category method, the minimum strength group and toughness classes for components of the topsides structure are given in Table 6. For the design class method, the steel toughness level and inspection category are given in Table 7.

Table 6 — Material category — Material selection for topsides

Component location in topsides		Strength group	Toughness class[a]		
			MC1	MC2	MC3
Deck legs	Connections up to 50 mm thick	II	CV2Z	CV2Z	CV2[d]
	Connections greater than 50 mm thick	II	CV2ZX[b]	CV2Z[c]	CV2[d]
	Elsewhere	I	CV2	CV1 or C	NT
		II	CV2	CV1	CV1
Deck truss	Chords	I	—	NT	NT
		II	CV2	CV2 or CV1	CV1
		III	CV2	—	—
	Diagonals	I	CV2 or CV1	CV1 or NT	NT
		II	CV2 or CV1	CV1	CV1 or C
Girders	Flange at connections and panel points	II	CV2ZX[b]	CV2Z	CV2[d]
		III	CV2ZX[b]	CV2Z	—
	Other flange, web, stiffeners	I	CV2	CV1 or NT	NT
		II	CV2	CV1	CV1
		III	CV2	CV2	—
Secondary structure	Bracing and floor beams (redundant)	I	NT	NT	NT
		II	CV1	NT	NT
Crane pedestal		II	CV2ZX[b]	CV2Z	CV2[d]
Lifting points	Padeye main plates and attachment points	II	CV2ZX[b]	CV2Z	CV2[d]

[a] Where two toughness classes are given, the higher class is recommended for tension structural components greater than 25 mm thick.

[b] CV2ZX includes mandatory crack tip opening displacement (CTOD) testing if greater than 50 mm thick.

[c] For connections greater than 75 mm thick, consider CV2ZX.

[d] Specify steel with a low sulfur content below 0,006 %.

Table 7 — Design class approach — Typical minimum selection for topsides structures

Joint/component	Design class (DC)	Steel toughness level	Inspection category[a]
Main vertical support trusses, longitudinal and transverse			
Lift nodes/complex/butt welds	1	CV2Z or CV2ZX	A or B
Butt and fillet welds in shear or compression			C
Lift nodes/simple/butt welds	2	CV2 or CV2ZX	A or B
Butt and fillet welds in shear or compression			C
Support nodes/butt welds in tension	2	CV2Z or CV2ZX	A or B
Butt and fillet welds in shear or compression			C
Columns and diagonals to lift nodes	2	CV2	
Butt welds to node			A or B
All other columns and diagonals	4	CV2	
Butt and fillet welds			B or C
Stiffeners/butt welds	4	CV1	B
Stiffeners/fillet welds			C
Remaining truss systems and deck structures			
Horizontal bracing/transverse butt welds	4	CV1	B or C
Fillet and longitudinal welds			C
Deck beams/girders/transverse butt welds	4	CV1	B or C
Longitudinal welds			C
Bulkheads (plates and stringers)	4	CV1	
All welds			C
Grating support beams/all welds	4	CV1	C
In-deck support beams and tanks/all welds	4	CV1	C or D
Mezzanine structures/all welds	4	CV1	C or D
Outfitting steel			
Crane pedestal and pedestal support	4	CV2	
Butt welds			B or C
Crane boom rest/all welds	5	CV1	C
Major support structures/all welds	4	CV1	C or D
Minor support structures/all welds	5		D
Lay-down and infill structures/all welds	5	NT	C or D
Runway beams and lift lugs/butt welds	4	CV2 or CV1	B or C
General outfitting structures/butt welds	4 or 5	CV1 or NT	D or E
Partial penetration and fillet welds			E
Installation aid structure			

NOTE For welds in tension, the most severe inspection category applies.

[a] Local areas of welds with high utilization shall be marked with frames showing areas for mandatory NDT when partial NDT are selected. Inspection categories depend on access for in-service inspection and repair.

Table 7 *(continued)*

Joint/component	Design class (DC)	Steel toughness level	Inspection category[a]
Main vertical support trusses, longitudinal and transverse			
Installation bumper and guide/all welds	5	NT	D
Rigging platforms/all welds	5	NT	D
Tuggerline padeyes/all welds	4	CV1	C

NOTE For welds in tension, the most severe inspection category applies.

a Local areas of welds with high utilization shall be marked with frames showing areas for mandatory NDT when partial NDT are selected. Inspection categories depend on access for in-service inspection and repair.

10.3 Stainless steel

10.3.1 General

Stainless steels generally exhibit excellent corrosion resistance and this is the main reason for their selection. However, these steels can be subject to corrosion under certain conditions although this can be minimized by paying attention to grade selection and detailed design. Consideration shall be given to the risk of galvanic corrosion of connected materials, particularly with carbon steel.

The stainless steels used offshore generally retain higher strengths at elevated temperatures than carbon steels. They can also provide exceptional ductility and energy-absorbing characteristics. The avoidance of a corrosion allowance and low maintenance requirements can lead to the economic selection of stainless steel as a structural material. Typical offshore applications include cable trays and ladders, ventilation louvers, floor panels, fire and explosion walls, ladders, walkways and module cladding.

Product availability, particularly for shapes, is such that greater use is made of cold-formed, welded or extruded sections.

10.3.2 Types of stainless steel

There are many types of stainless steel and these fall into five main groups, classified according to their metallurgical structure (i.e. the austenitic, ferritic, martensitic, duplex and precipitation-hardening groups). Not all of these are suitable for structural applications, particularly where welding is required. The austenitic stainless steels are the most useful group for offshore structural applications. The most common alloy used is 17Cr 12Ni 2Mo steel (more usually referred to as 316 steel with the low-carbon variant 316L steel).

Austenitic steels can be strengthened by work hardening. Welding and heat treatments will partially anneal such strengthened materials resulting in some loss of the strength enhancement.

Compatible fastenings shall be selected to avoid corrosion problems. Bolting materials are covered in ISO 3506[29].

10.3.3 Material properties

The density of stainless steel is dependent on the properties of the alloying elements but may be taken as 8 000 kg/m^3 for grade 316 steels.

As a first approximation, Young's modulus may be taken as 195 000 N/mm^2.

Austenitic stainless steels have lower thermal conductivity, but higher thermal expansion than ferritic steels, including structural carbon steels. The effects of differential thermal expansion shall be considered in design. These thermal properties can also lead to greater welding distortion in austenitic stainless steel components, even where careful jigging is used during fabrication.

10.4 Aluminium alloys

10.4.1 General

Not all aluminium alloys are resistant to marine corrosion and careful material selection is required. Appropriate alloys have excellent corrosion resistance in marine environments, but are liable to galvanic corrosion when combined with other materials, including structural steels, stainless steels and copper alloys. Electrical isolation is generally required, often obtained by using insulating packers at connections to carbon steel.

The properties of aluminium alloys can be severely degraded by welding and this shall be allowed for in the design of connections.

Aluminium loses strength and stiffness rapidly when subjected to heat.

Aluminium alloys have found successful applications in the construction of living quarters modules, helidecks, crane boom rests and general decking.

10.4.2 Types of aluminium

The two most common types of aluminium alloy used for offshore structures are the heat-treatable 6XXX series, specifically 6082, and the non-heat-treatable 5XXX series, specifically 5083, which obtains its increased strength from work hardening. Both materials are susceptible to loss of strength in the heat-affected zone of a welded connection.

For welded structures, alloys should be used in the annealed condition and be selected from materials with a strength not exceeding 130 N/mm^2 at the specified 0,2 % strain.

Higher-strength alloys can be considered for non-welded construction.

10.4.3 Material properties

Typical properties of aluminium alloys are as follows:

— density: 2 700 kg/m^3

— Young's modulus: 7×10^4 N/mm^2

— yield strength (6082 alloy): 130 N/mm^2

— yield strength (5083 alloy): 220 N/mm^2

Aluminium has a high heat conductivity and specific heat. It melts at 550 °C and loses 50 % of its strength at 225 °C. Its thermal expansion is twice that of steel.

10.4.4 Thermite sparking

Thermite (aluminium-iron oxide) sparking can occur when iron oxide (rusty steel) comes into contact with aluminium. It requires specific circumstances to produce a thermite spark of appreciable energy; for this to represent a hazard, it has to occur in combination with an explosive gas/air mixture. When aluminium is used for structural applications, the operations manual or other documentation shall contain warnings and advice that precautions should be taken to prevent thermite sparking, and the structure itself shall be labelled with warnings.

NOTE Thermite sparking is also called frictional sparking and incendive sparking.

10.5 Fibre-reinforced composites

Fibre-reinforced composites can be produced with a wide range of properties, including high strength, and with considerable resistance to fire. A wide range of resin binders and fibres are used and the technology has been developing rapidly.

Due to the large variation in material properties, there is a paucity of design codes for use of these materials and their suitability is usually determined by type-testing to meet performance criteria.

Fibre-reinforced composites have been successfully used in the production of floor grating, hand railing and ladders, lightweight fire and explosion-resistant panels, and for reinforcement and repair of carbon steel sections.

Fibre-reinforced composites are often electrically non-conductive and any conductive and metallic objects attached to fibre-reinforced composites should be independently earthed where necessary.

In fire conditions, fibre-reinforced composites can give off toxic fumes and the risks from such fumes shall be considered.

10.6 Timber

The use of timber in offshore topsides structures has generally been restricted to the protection of weather decks from dropped objects and damage from pipe handling. It has been found effective against dropped objects when sandwiched between two steel sheets.

Because timber is generally flammable, a problem exacerbated by its ability to soak up hydrocarbon spills, it shall not be used in confined hazardous areas.

Timber properties are highly variable and anisotropic. Design should be undertaken in accordance with appropriate codes and standards.

11 Fabrication, quality control, quality assurance and documentation

11.1 Assembly

11.1.1 General

The requirements for fabrication, quality control, quality assurance and documentation given in ISO 19902 shall be followed with the additional requirements given below.

Fabrication, other than welding, shall be in accordance with a national or regional fabrication specification that complements the design code. Fabrication tolerances shall be compatible with design assumptions. In some situations, tighter than normal tolerances are required and these shall be documented on the drawings.

11.1.2 Grating

Joints in grating shall occur only at points of support, unless other appropriate details are specified.

11.1.3 Landing and stairways

Landing elevations and landing and stairway locations shall be within 50 mm in plan of the drawing dimensions unless required to align with other access ways or equipment, in which case the mismatches in elevation and alignment shall not exceed ± 4 mm.

11.1.4 Temporary attachments

Any temporary attachments to the topsides structure (including crane pedestals), such as scaffolding, fabrication, and erection aids, can create a localized stress rise (even after removal) and should be avoided where practicable. When these attachments are necessary, the following requirements apply.

— Temporary attachments shall not be removed by hammering or arc air gouging. Attachments to leg joint cans, brace joint cans, brace stub ends and joint stiffening rings shall be cut to 3 mm above parent metal and mechanically ground to a smooth flush finish with the parent metal.

— Attachments on all areas that are to be painted shall be removed as above, prior to any painting.

— Attachments to all other areas, not defined above, shall be removed by cutting just above the attachment weld (maximum 6 mm above weld). The remaining attachment steel shall be completely seal-welded.

— Attachments to aid in the splicing of legs, braces, sleeves, piling, conductors, etc. shall be removed to a smooth, flush finish.

— The parent steel shall be tested by magnetic particle inspection (MPI) following removal of temporary attachments.

11.2 Welding

Welding shall comply with the requirements of ISO 19902 with the following additional considerations.

a) Metal thicknesses encountered in topsides structures can exceed those in the associated support structures, particularly at support and lifting points. At such points post-weld heat treatment can be required and shall comply with the requirements of ISO 19902.

b) The sequence of welding can have a significant effect on residual stresses. This is of particular concern when large decks with several levels are fabricated deck by deck with a large number of splices in primary structural columns and braces. The potential for such build-up of residual stress shall be considered by both designer and fabricator and, where appropriate, measures shall be taken to reduce it to a minimum.

c) The use of automatic welding machines on large areas of deck plate or in the fabrication of girders or grillages can significantly increase heat-induced distortion which can result in unacceptable deflections in deck steelwork. Welding procedures shall be assessed for their potential to cause such distortion and modified if necessary.

11.3 Fabrication inspection

The requirements for quality control and fabrication inspection shall follow the applicable clauses of ISO 19902.

11.4 Quality control, quality assurance and documentation

The requirements for quality control, quality assurance and documentation, including drawings and specifications, given in ISO 19902 shall be followed for the topsides structure.

Drawings and specifications shall clearly and unambiguously show the intention of the design. Sufficient information shall be given to define the materials and any special construction methods, tolerances, inspection requirements and operational constraints.

All engineering information necessary for the safe use of the topsides structures shall be made readily available and transmitted to those personnel operating the platform. Such information shall include a topsides load plan defining the maximum carrying capacity of areas used for storage, access and maintenance and the total maximum topsides weight. Areas requiring periodic inspection to ensure the continued safe operation of the structures shall be identified.

11.5 Corrosion protection

11.5.1 Coatings

The application of coatings shall conform to the manufacturer's recommendations and to any suitable standard specified by the owner or by the designer.

11.5.2 Under deck areas

Splash zone protection, such as monel wrap, steel plate wrap, corrosion allowance, etc., shall be installed as specified and shall cover not less than the areas indicated on the drawings or in the specifications.

12 Corrosion control

12.1 General

Corrosion damage can affect structural integrity in various ways. The primary objective of corrosion control is to prevent loss of strength and changes in fatigue resistance. The presence of corrosion in fatigue-sensitive areas can result in increased stress concentrations and hence the initiation of fatigue damage.

12.2 Forms of corrosion, associated corrosion rates and corrosion damage

Corrosion damage to uncoated carbon steel is associated with oxygen attack and is typically more or less uniform. In typical conditions, the steady-state corrosion rate for carbon steel (i.e. as uniform attack) is around 0,1 mm/year or lower.

Aluminium alloys of the 5XXX and 6XXX series, as used for topsides structural components, are highly resistant to marine atmospheres and suffer only superficial staining or micropitting. However, galvanic coupling (i.e. metallic plus electrolytic coupling) to structural steel, stainless steels and copper alloys shall be avoided. Otherwise, severe galvanic corrosion of aluminium can result at the point of contact. Similarly, structural steel can suffer enhanced corrosion if in galvanic contact with stainless steel or with copper base alloys.

Very-high-strength steels (yield strength in excess of 1 200 MPa) and certain high-strength aluminium, nickel, and copper alloys are sensitive to stress corrosion cracking in marine atmospheres.

NOTE The term "stress corrosion cracking" refers to cracking that is caused by an interaction between static tensile stresses in a material and a specific corrosion medium.

12.3 Design of corrosion control

12.3.1 General

The main systems for corrosion control of topsides structures are

a) coatings, linings and wrappings,

b) corrosion-resistant materials, and

c) corrosion allowance.

12.3.2 Considerations in the design of corrosion control

The initial selection and subsequent detailed design of systems for corrosion control of topsides structures shall take the following factors into account:

a) regulatory requirements;

b) criticality of the overall system and the functional requirements to individual structural components to be protected;

c) type and severity of corrosion environment(s);

d) design service life (and likelihood of lifetime extension);

e) accessibility for inspection and maintenance, including overall maintenance philosophy;

f) suitability, reliability and economy of optional techniques for corrosion control.

12.3.3 Coatings, linings and wrappings

Coatings are defined as relatively thin (<1 mm) organic or metallic layers, single or multiple, that are applied by spraying, brushing or dipping. Linings and wrappings are defined as thicker (>1 mm) corrosion-protective layers applied with the objective of resisting mechanical wear, protecting against impacts, etc. Organic materials used for linings and wrappings are normally reinforced (e.g. by glass fibres or flakes). Metallic materials are typically copper-based.

Special precautions are required to prevent corrosion under coatings, linings and wrappings, including under fire-protective coatings. Metallic materials should be seal-welded to structural components.

Coating systems include various forms of organic (paint) coatings and certain metallic coatings. Of the latter, zinc layers are applied as hot dipping or thermal spraying. Thermally sprayed aluminium coatings have been used more recently, particularly for more demanding applications.

Coating and lining systems shall primarily be selected based on proven experience for a specific application and environment. Comprehensive field testing is required when practical experience is lacking. Maintainability is a major criterion. Resistance to damage is also required.

The design of all components to be paint-coated shall take into account relevant measures to ease both the initial application and later maintenance. This includes a preference for tubular shapes, rounding of sharp edges, requirements for securing scaffolding, etc. Structural components exposed to sea spray, rain or intermittent wetting, externally or internally, shall be designed to prevent accumulation of moisture, e.g. by using continuous welding and making provisions for drainage.

12.3.4 Corrosion-resistant materials

The selection of corrosion-resistant materials for structural components shall take into account their anticipated corrosion resistance for the intended application, their compatibility with other materials, their mechanical properties and their ease of fabrication.

Precautions shall be taken to prevent galvanic corrosion of less resistant materials; these can include coating components with the higher electrochemical potential or the use of electric insulation.

12.3.5 Corrosion allowance

A corrosion allowance, i.e. extra metal thickness to compensate for loss by corrosion, can be used alone or in combination with a coating. The thickness of any corrosion allowance shall be determined by taking expected corrosivity, design service life and maintenance plans into account.

12.4 Fabrication and installation of corrosion control

12.4.1 General

Fabrication procedures can affect the corrosion resistance of certain materials. All fabrication involving welding or brazing to structural components shall be performed in accordance with ISO 19902, regulatory requirements, applicable codes/standards and approved project-specific procedures and drawings.

12.4.2 Coatings and linings

Manufacturer's recommendations and any recommendations given in applicable standards and practices for surface preparation, materials, coating application, inspection and repairs should be followed.

Quality control during surface preparation, coating application and repairs ensures consistent performance of coatings and linings. All coating work, from surface preparation to completion, should be inspected by a certified coating inspector.

12.4.3 Corrosion-resistant materials

Work with corrosion-resistant materials shall be performed with due consideration of how the applicable techniques (welding, grinding, etc.) affect their corrosion resistance and mechanical properties. Improper fabrication methods can easily cause staining and incipient pitting of stainless steel and aluminium surfaces.

12.5 In-service inspection, monitoring and maintenance of corrosion control

12.5.1 General

Periodic inspections assess the physical condition and integrity of the corrosion control system(s), or the actual components to be protected, or both. Monitoring of corrosion control refers to regular recording of data associated with corrosion control.

Plans for inspection and monitoring of corrosion control shall take into account the following:

a) criticality of the overall system and of the individual components being protected;

b) type and severity of the corrosion environment(s);

c) potential forms of corrosion damage;

d) capability of inspection and monitoring tools, as well as accessibility for inspection;

e) results and consequences of previous inspections, or monitoring, or both.

NOTE See Clause 14 for in-service inspection and corrosion control requirements for structural integrity management.

12.5.2 Coatings and linings

Inspection of coatings and linings is primarily performed by visual inspection and has the objective to assess the need for maintenance (i.e. repairs). A visual examination will also disclose any areas where coating degradation has allowed corrosion to develop to a degree requiring repair or replacement of structural components.

12.5.3 Corrosion-resistant materials

Inspection of corrosion control based on corrosion-resistant materials can be integrated with visual inspection of the structural integrity of critical components associated with such materials.

13 Loadout, transportation and installation

The methodology for loadout, transportation and installation shall be agreed between the design, fabrication, transportation and installation contractors, taking into consideration any requirements of the owner.

The requirements given in ISO 19901-6 for loadout, transportation and installation shall be followed.

Any structural components required for loadout, transportation and installation shall be designed following the requirements of this part of ISO 19901 in conjunction with all factors from ISO 19901-6.

14 In-service inspection and structural integrity management

14.1 General

The requirements for in-service inspection and structural integrity management given in ISO 19902 shall apply to all topsides structures covered by this part of ISO 19901, noting the particular considerations applying to topsides structures given in 14.2 and the default inspection scopes given in 14.3.

14.2 Particular considerations applying to topsides structures

14.2.1 Corrosion protection systems

For many parts of topsides structures, corrosion, rather than fatigue or accidental damage, is likely to be the principal cause of deterioration. Topsides structures are generally protected by paint and coating systems to reduce the rate of corrosion (see Clause 12). Corrosion protection systems shall be suitably maintained to retain their effectiveness.

14.2.2 Access routes, floors and gratings

To safeguard personnel for both in-service and accident conditions, the safety criticality of these structures shall be considered and suitable inspection intervals and techniques devised.

14.2.3 Supports for safety-critical equipment, including communications, electrical and firewater systems

Equipment supports can be affected by extreme, abnormal and accidental actions, including consequent strong vibration. Inspection scopes and techniques shall be determined accordingly.

14.2.4 Control of hot work (e.g. welding and cutting)

Hot work in service to attach appurtenances, pipe supports, cable trays, etc., or to cut access holes, shall be carefully controlled to prevent damage to the integrity of safety-critical parts of the structure. Hot work shall be minimized as far as possible by considering possible future requirements at the design stage.

14.2.5 Accidental actions

Arrangements shall be made in drawing up the structural integrity management plan for the topsides structure to inspect, assess and implement any necessary remedial measures and evacuation procedures as quickly as possible after an incident.

14.2.6 Change control

Any changes, or the cumulative effect of changes, that can significantly affect the actions on and structural response of safety-critical structural components, or of the entire structural system, shall be assessed at the planning and design stages of the proposed alterations. As-built inspections shall be undertaken to assess the impact and extent of any potential modifications.

14.3 Topsides structure default inspection scopes

14.3.1 General

The default inspection scope for the baseline inspection and for the subsequent periodic inspections given in 14.3 shall apply to the topsides structure unless an in-service structural inspection strategy has been prepared and implemented in accordance with ISO 19902.

14.3.2 Baseline inspection

A baseline inspection, to provide a benchmark of the installed condition of the topsides structure, shall be undertaken as soon as possible after installation and commissioning of the topsides, and no later than one year after installation.

The objective of this inspection is to identify any defects which have the potential to impair the integrity of the topsides structure and equipment, so as to allow these defects to be assessed and repaired before having an effect on integrity.

The minimum scope of inspection shall include the following items.

a) A general visual inspection (GVI), without removal of paint and coatings, of all parts of the topsides structure, including equipment support structures, to check that

   1) all parts of the topsides structure are intact and undamaged,

   2) all fixings between structures and between structures and equipment, including gratings and handrails, are secure, and

   3) paintwork and protective coatings are not damaged.

b) A walk-down survey to assess the vulnerability of safety-critical equipment and supports to damage from impulsive actions and strong vibration induced by actions from extreme environmental events or accidental actions, unless this survey was undertaken at the fabrication site (see 6.9).

c) An examination to determine the integrity of any installed passive fire protection systems.

d) An assessment of any vibration caused by operating equipment or by local vortex-induced vibration.

14.3.3 Periodic inspection

The inspection intervals described in ISO 19902 shall apply to topsides structures. They may be simplified for topsides as described below.

a) In Level I inspection, the minimum scope shall consist of a visual survey to determine

   1) the continued effectiveness of coating and passive fire protection systems,

   2) any signs of excessive corrosion, and

   3) the existence of any bent, missing or damaged structural components.

   The survey shall identify indications of obvious overloading, design deficiencies and any operational usage that is inconsistent with the original design intent for the topsides structure. The survey shall include a GVI of all areas of topsides structure that have been identified as safety-critical. Should the Level I survey indicate that damage can have occurred, a Level II inspection shall be conducted as soon as conditions permit.

b) In Level II inspection, the minimum scope shall consist of

   1) a GVI without removal of paint and coatings of all parts of the topsides structure including equipment support structures (as described above for a Level I inspection),

2) a close visual inspection (CVI) of all structural components identified as safety-critical, and

3) a detailed non-destructive examination of a selection of safety-critical structural components and comprising not less than 10 % of all safety-critical structural components.

If damage is detected, 100 % non-destructive testing of the suspect area shall be used where visual inspection alone cannot fully determine the extent of the damage.

c) In Level III inspection, the minimum scope shall consist of

1) a GVI without removal of paint and coatings of all parts of the topsides structure including equipment support structures (as described above for Level I and Level II inspections),

2) a CVI of all structural components identified as safety-critical, and

3) a detailed non-destructive examination of all safety-critical structural components.

d) There is no requirement for a Level IV inspection of topsides structures.

14.3.4 Special inspections

Special inspections shall be performed where necessary, as follows:

— to assess the performance of repairs undertaken to ensure the continuing fitness-for-purpose of the topsides structure: the minimum requirement for such repairs is a GVI conducted approximately one year after completion of the repair;

— to monitor known defects, damage, local corrosion, or other conditions, which could potentially affect the fitness-for-purpose of the topsides structure, risers and J-tubes, conductors, and other safety-critical structures and equipment;

— if the topsides structure is planned for reuse.

See also ISO 19902.

14.3.5 Unscheduled inspections

An inspection shall be conducted as soon as practical after the occurrence of an environmental event (e.g. storm, earthquake or mudslide) exceeding that for which the platform was designed or assessed, or an accidental event (e.g. vessel impact, dropped object, fire or explosion). The minimum scope of inspection shall consist of a GVI of all safety-critical structures and supporting structures, including equipment and pipework supports, conductors, risers, and appurtenances,

— to check for any signs of damage, and

— to confirm the continuing effectiveness of corrosion protection systems.

Where signs of significant damage to the topsides structure or coatings are found, a CVI shall be performed. Detailed non-destructive examination shall be performed as appropriate, depending on the results of the CVI.

15 Assessment of existing topsides structures

The requirements for assessment of existing structures given in ISO 19902 shall apply to all topsides structures covered by this part of ISO 19901, following the assessment of actions and resistances detailed in this part of ISO 19901 where applicable.

16 Reuse of topsides structure

ISO 19902 gives requirements and guidance on fixed steel structure reuse. Topsides structures present particular problems in this respect as access for inspection is likely to be restricted by the plant and equipment and there is a significant likelihood that modifications to both the topsides structure and the equipment have been made during the platform's original service life.

In addition to survey work, all of the considerations applicable to a new design are likely to be relevant.

Annex A
(informative)

Additional information and guidance

NOTE The clauses in this annex provide additional information and guidance on the clauses in the body of this part of ISO 19901. The same numbering system and heading titles have been used for ease in identifying the subclause in the body of this part of ISO 19901 to which it relates.

A.1 Scope

No guidance is offered.

A.2 Normative references

No guidance is offered.

A.3 Terms and definitions

No guidance is offered.

A.4 Symbols and abbreviated terms

No guidance is offered.

A.5 Overall considerations

A.5.1 Design situations

No guidance is offered.

A.5.2 Codes and standards

General procedures for the design of fixed steel structures were originally developed and documented by the American Petroleum Institute in earlier versions of API RP 2A-WSD[1], which is only concerned with components formed from tubular sections. Topsides, however, are constructed from a range of structural sections so API RP 2A-WSD[1] provides little design guidance relevant to topsides. As a result, different countries adopted a variety of national codes to effect such designs, e.g. ANSI/AISC 360-05[2] in the USA, the so-called “Eurocodes” in Europe and CSA-S16-09[3] in Canada. These choices were natural, being the building codes for the countries concerned. Only one, however, has been formally adapted for offshore practice. This is ANSI/AISC 360-05[2], which was calibrated as AISC LRFD against API RP 2A-LRFD[4] with resistance factors derived for consistency with the reliability implicit in API. CSA-S16-09[3] was calibrated against land-based steel building practice and the resistance factors were determined for a given set of action factors.

Building code correspondence factors (K_c factors) for codes other than AISC are being developed by some national standards bodies (e.g. Canada) and should be available in the respective national standards version of this part of ISO 19901 when published.

A.5.3 Deck elevation and green water

For fixed platforms, for jackups and for some floating platforms, a safety margin or air gap is required between the crest of the design wave and the lowest point (structural component, equipment or fixing) of the lowest deck of the platform such that abnormal wave crests do not impinge on the deck or equipment. This is necessary since very large actions can occur if a wave hits the deck. If there is insufficient deck elevation, wave impact can reduce the reliability of the structure. The determination of the air gap should account for uncertainty in water depth, structure settlement, sea floor subsidence, sea level rise, storm surge and tide, and abnormal wave crest elevation.

Further guidance is given in ISO 19901-1, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1 and ISO 19906, as appropriate.

A.5.4 Exposure level

No guidance is offered.

A.5.5 Operational considerations

No guidance is offered.

A.5.6 Selecting the design environmental conditions

For most purposes a relatively simple wind model suffices (see ISO 19901-1).

The wind speed in a 3 s gust is appropriate for determining the maximum quasi-static local actions caused by wind on individual components of the structure; 5 s gusts are appropriate for maximum quasi-static local or global actions on structures whose maximum horizontal dimension is less than 50 m; and 15 s gusts are appropriate for the maximum quasi-static global actions on larger structures.

A.5.7 Assessment of existing topsides structures

No guidance is offered.

A.5.8 Reuse of topsides structure

No guidance is offered.

A.5.9 Modifications and refurbishment

No guidance is offered.

A.6 Design requirements

A.6.1 General

The design of all systems of a topsides should be undertaken by competent engineers with appropriate training, qualifications, design office and on-site experience, as necessary.

A.6.2 Materials selection

No guidance is offered.

A.6.3 Design conditions

No guidance is offered.

A.6.4 Structural interfaces

No guidance is offered.

A.6.5 Design for serviceability limit states (SLS)

A.6.5.1 General

No guidance is offered.

A.6.5.2 Vibrations

A.6.5.2.1 Sources of vibration

Vibration control is best achieved during design. The specification of equipment so as to minimize out-of-balance energy and the isolation of vibration at source by the use of AVMs at points of support should form a part of the design philosophy. Where AVMs are specified the flexibility requirements and potential for damage to services bridging between the equipment and topsides structure should be considered.

Because of the complex interaction between structural components and various actions within a topsides, the accurate calculation of the natural frequency of individual components is extremely difficult (particularly so for torsional modes in open sections). Where an analytical solution is sought, the supporting structure should be designed to have a natural frequency at least 2,5 times higher than or lower than the principal operating frequencies of the equipment concerned. Where this cannot be achieved, the amplitudes should be assessed and shown to be within acceptable limits.

The majority of resonant response will be avoided by compliance with the specified deflection limits. With the possible exception of the beams directly supporting large rotating equipment that is not isolated by AVMs, control or resonance can best be established by monitoring performance during commissioning and locally modifying stiffness or mass where any problems are identified.

A.6.5.2.2 Design limits

ISO 2631-1, ISO 2631-2 and ISO 6897[5] present methods of determining, and guidelines on, the effects of vibrations on humans. Vibration can contribute to

a) motion sickness,

b) discomfort,

c) noise,

d) health disorders, and

e) fatigue.

The acceptable accelerations depend on the duration of the exposure. With regard to comfort, accelerations of less than 0,315 m/s^2 are not likely to be uncomfortable and those in the range 0,315 m/s^2 to 0,63 m/s^2 can be a little uncomfortable. Guidance from ISO 2631 and ISO 6897[5] should be followed where expected accelerations are greater than 0,63 m/s^2.

A.6.5.2.3 Long-period vibrations

The natural period limit of 1 s should restrict movements from occasional resonant response to platform operations (drilling and crane operations). Where this cannot be achieved, the system should be analysed to demonstrate satisfactory performance in both serviceability and limit-state conditions. In areas of significant seismic activity, lower natural period limits would apply and an analysis should be performed unless previous design experience demonstrates that this is unnecessary.

A.6.5.2.4 Dynamic analysis and avoidance of resonance

No guidance is offered.

A.6.5.3 Deflections

The deflection limits in 6.5.3 are derived from BS EN 1993-1-1[6], but have been simplified. In BS EN 1993-1-1, the intention is to limit damage to finishes, but the same values are appropriate to limit stresses in pipework.

To avoid disruptions to communications, rotation at the top of telecommunications masts should not exceed 0,01 rad. Microwave communications require particularly well-aligned dishes.

A.6.6 Design for ultimate limit states (ULS)

No guidance is offered.

A.6.7 Design for fatigue limit states (FLS)

No guidance is offered.

A.6.8 Design for accidental limit states (ALS)

No guidance is offered.

A.6.9 Robustness

The robustness concept is closely related to resisting and mitigating the effects of accidental, abnormal and seismic events, consequences of human error, and failure of equipment. In accordance with ISO 19900, these situations are denoted hazardous circumstances, or simply hazards. Robustness is also important in the event of serious but unidentified fatigue damage.

Robustness is achieved by considering the ALS that represent the structural effects of hazards. Ideally, all such likely hazards should be identified and quantified by means of rational analyses. However, in many cases, it is possible, based on experience and engineering judgement, to identify and reasonably quantify the most important ALS. They will often be those from ship impact, dropped objects, explosions and fires.

The design should comply with ISO 19900, which uses the following approach:

— careful planning of all phases of development and operation;

— avoiding the structural effects of the hazards by either eliminating the source or by bypassing and overcoming these hazards;

— minimizing the consequences;

— designing for hazards.

When the hazard cannot reliably be avoided, the designer has a choice between minimizing the consequences (i.e. the consequences of losing a component due to a hazard), or designing for the hazard (i.e. making the component strong enough to resist the hazard). In the first case, the topsides structure should be designed in such a way that all primary structural components that can be exposed to hazards have redundancy such that the forces they carry can be redistributed within the topsides structure. In the second case, critical components that can be exposed to hazards are made strong enough to resist the hazards considered.

It should be emphasized that robustness requirements do not imply that all structures should be able to survive removal of any structural component if no hazards are likely to occur. The starting point is a hazard that is more unlikely to happen than the usual design situations, but not unlikely enough to be neglected. If there is no hazard, then there is no requirement in relation to robustness. Only one hazard at a time should be considered.

A walk-down is not just a physical inspection of a topsides, but encompasses all the steps necessary to demonstrate the adequacy of the components under assessment. These include the initial identification of SCE, whether a component and its anchorages appear able to withstand the applied actions, whether the component and its supports appear able to exhibit ductile behaviour under extreme actions, and whether there are likely to be any interactions with nearby equipment or structures. The review of the support structures and access platforms to equipment is based on knowledge from previous experience and consideration of possible load paths during different loading situations.

A.6.10 Corrosion control

No guidance is offered.

A.6.11 Design for fabrication and inspection

No guidance is offered.

A.6.12 Design considerations for structural integrity management

No guidance is offered.

A.6.13 Design for decommissioning, removal and disposal

A.6.13.1 General

A detailed design or analysis is not specifically required for platform decommissioning and removal, but a conceptual study should be carried out to ensure no major or unusual problems exist and to demonstrate that a cost-effective and environmentally sound method exists.

A.6.13.2 Structural releases

No guidance is offered.

A.6.13.3 Lifting appurtenances

Where lifting attachments are removed, particular consideration should be given to their reinstatement without undue constraint on the removal schedule (i.e. avoidance of marine spread waiting while attachments are reinstated). The method of lifting-attachment reinstatement should address possible load paths, with attention being given to shear connections and avoidance of lamellar tearing.

A.6.13.4 Heavy lift and set-down operations

The dynamic factors associated with placing a module on a transportation barge in open seas can be greater than those associated with their placement on a platform offshore or on the deck of the crane vessel (see also ISO 19901-6).

A.7 Actions

A.7.1 General

No guidance is offered.

A.7.2 In-place actions

During the design of a topsides the structure usually has to be analysed and dimensions of components defined before the design of the process plant and other equipment is completed. Usual practice is to use a weight database that is updated periodically during the design process. Contingency factors are applied to the values from the weight database to ensure that the structural design is sufficient for the weights at the conclusion of the topsides design. These contingency factors are progressively reduced as the design matures. The values of G_1, G_2, Q_1, and Q_2 should be increased to include these contingency factors until the conclusion of the design. See ISO 19901-5[7] for further guidance.

Potential shifts of centre of gravity can be addressed by considering a centre-of-gravity envelope and applying an addition factor based on the potential variation of reactions due to shifts of the centre of gravity within the envelope.

The assessment of existing structures can be required when the understanding of the weights is poor, possibly due to the loss of original design data. In such cases, efforts should be made to improve the quality of the weight database, for example by undertaking a weight audit, and contingency factors should be used to ensure weights are not underestimated.

A.7.3 Action factors

No guidance is offered.

A.7.4 Vortex-induced vibrations

The method detailed in API RP 2A-LRFD[4] provides a rational method for this evaluation.

DNV-RP-C205[9] and Reference[8] provide information for lattice structures and exposed pipework.

A.7.5 Deformations

In general, the ultimate strength of ductile structural systems is not sensitive to internal forces due to imposed deformations. For ductile failure modes, the resistance is not affected by the initial level of internal stressing or by deformation-controlled phenomena like uneven settlements. Internal forces due to deformations can become important when subsequent loading is cyclic and can cause repeated plastic deformation. In such cases, it should be shown that the structure, after the initial plastic deformation, establishes a stable condition after which the cycling takes place in the linear domain. This process is called shake-down. If shake-down is not achieved, a fatigue check against repeated yielding should be undertaken.

A.7.6 Wave and current actions

No guidance is offered.

A.7.7 Wind actions

No guidance is offered.

A.7.8 Seismic actions

A.7.8.1 General

For seismic zones 0 and 1 (see ISO 19901-2), the design of topsides for earthquake actions remains limited. In areas where the design horizontal spectral acceleration for the extreme level earthquake (ELE) does not exceed 0,10g, the design of fixed platforms for storm conditions generally produces support structures that are adequate to resist imposed seismic design conditions; module support frames, deck structure, and appurtenances can be exceptions to the foregoing. For fixed platforms in these seismic zones, the ductility requirements for topsides structure may be waived and the tubular joints designed only for the calculated joint forces (instead of structural component yield or buckling forces), provided the topsides structure meets the strength design requirements using ground motion characteristics established for the rare, abnormal level earthquake (ALE). However, even though the provisions do not require further earthquake analysis of the topsides structure, the designer should consider the seismic response in configuring the topsides structure by providing redundancy and recognizing the implications of abrupt changes in stiffness or strength, as well as by applying engineering judgement in the design of structures of unusual configuration.

Amplification of the vertical overall platform response has been found to be a problem when the natural periods of beams or cantilever trusses are close to a vertical mode of the overall platform. Coupled analysis can be necessary in these circumstances.

The ELE requirements are intended to provide a topsides that is adequately sized for strength and stiffness. This is to ensure that no significant structural damage is sustained. The ALE requirements are intended to ensure that the topsides has sufficient reserve capacity to prevent its collapse during rare, intense earthquake motions with an annual probability of exceedance of 10^{-4}. These rare earthquake motions may result in inelastic behaviour and structural damage as long as there is no progressive collapse.

Additional guidance on seismic design is given in ASCE/SEI 7-05[10] and BS EN 1998-1[11].

A.7.8.2 Minimum lateral acceleration

A minimum lateral deck acceleration of 0,20*g* for topsides on fixed platforms recognizes that the topsides are a particularly sensitive part of the platform in that they are not designed for wave actions and the requirement provides additional protection for inertial forces from accelerations due to minor vessel impacts.

A.7.8.3 Equipment and appurtenances

The method of deriving actions for the seismic design for equipment or a deck appurtenance depends upon its dynamic characteristics and the framing complexity. There are two analysis alternatives.

First, through proper anchorage and lateral restraint, most deck equipment and piping are sufficiently stiff that their support framing, lateral restraint framing, and anchorage can be designed using static actions derived from peak deck accelerations associated with the extreme level earthquake.

To provide assurance that the equipment or appurtenance is sufficiently stiff to meet this criterion, the lateral and vertical periods of the equipment or appurtenance should be very different from the main periods of vibration of the topsides structure. Additionally, the local framing of the deck that supports the equipment or appurtenance should also be rigid enough not to introduce dynamic amplification effects. In selecting design lateral acceleration values, consideration should be given to the increased response towards the corners of the deck caused by torsional response of the platform.

Second, in cases of more compliant equipment or appurtenances – such as drilling and well servicing structures, flare booms, cranes, deck cantilevers, tall free-standing vessels, unbaffled tanks with free fluid surfaces, long-spanning risers and flexible piping, escape capsules, and wellhead/manifold interaction – consideration should be given to accommodating the additional forces caused by dynamic amplification, or differential displacements, or both. These forces can be estimated through either coupled or uncoupled analyses.

Uncoupled analyses using deck floor spectra are likely to produce larger design actions on equipment than those derived using a more representative coupled analysis, particularly for more massive components and those with natural periods close to the significant natural periods of the overall platform. API RP 2A-LRFD[4] and References [13] and [14] describe coupled procedures, and uncoupled procedures that attempt to account for such interaction.

If coupled analyses are used on relatively rigid components that are modelled simplistically, the design accelerations derived from the modal combination procedure should not be less than the peak deck accelerations.

Walk-down inspection by experienced personnel of equipment and piping on existing platforms in seismic areas can identify equipment and pipework supports that should be improved (see 6.9). The addition or deletion of simple bracing, or supports, or both, can significantly improve the behaviour of equipment and pipework during an earthquake.

The use of higher partial action factors can be appropriate for designing deck supported structures, local deck framing, equipment supports, and lateral restraints under ELE actions. This higher partial action factor is intended to provide an additional margin of safety in place of performing an explicit ductility analysis. In areas where the ratio of rare, abnormal ground motion intensities to extreme level ground motion intensities is known to be greater than 2,0, an adjustment to the partial action factor should be considered. In addition, for certain equipment, piping, appurtenances or supporting structures, the degree of redundancy, consequences of failure and/or characteristics of the metallurgy can dictate the use of different (i.e. higher) partial action factors for the ELE or a full ductility analysis, depending on the component's anticipated performance under rare, abnormal earthquake ground motions.

For drilling and well servicing structures, ISO 13626[12] should be used to design for earthquake actions.

A.7.9 Actions during fabrication and installation

No guidance is offered.

A.7.10 Accidental situations

A.7.10.1 General

The maintenance of structural integrity can prevent or reduce escalation of accidental events causing significant damage. The design of the structure therefore plays an important role, along with other engineering disciplines, in developing inherent safety and in implementing a safety management system for the platform.

Extreme weather, seismic and accidental events that can have particularly significant effects on the topsides of certain types of offshore structures are dealt with in other International Standards in the ISO 19900- series. Requirements for FPSOs, for example, are stated in ISO 19904-1.

Accidental situations are low-probability events. Provided that the total risk from all causes is as low as reasonably practicable, actions having an annual probability of exceedance of less than 10^{-4} need not be considered.

In evaluating resistances, it is appropriate to use the ultimate strength of the individual components and global structure. Best estimates of actions and resistances should be used for the structural calculations. These may include removal of implicit factors of safety (e.g. actual yield strength, large deflection effects such as membrane and catenary effects, enhanced strength due to high strain rates and strain hardening, etc.). Where dynamic actions are involved, the energy absorption due to ductile deformation may be taken into account. In addition to direct accidental actions, the structure should have sufficient capability to resist projectiles resulting from explosions.

Certain locations of the deck, such as crane loading areas and areas near the drilling rig, are likely to be subject to dropped and swinging objects. The location of equipment and facilities below these areas should be considered so as to minimize potential damage from these causes.

When it is necessary to enclose parts of a topsides at locations where the potential for a gas explosion exists, the protective side panels or walls should include suitable blow-out panels to minimize confinement and reduce any resultant action on primary structural components. However, blow-out panels can have a limited effect in reducing overpressures in large, congested areas. Blow-out panels should not be considered an alternative to open boundaries unless the increase in the overall risk, e.g. by increasing probable cloud size, is shown to be acceptable. In cold climates their use can, however, be necessary.

Accidents or equipment failures can cause significant structural damage. Inspection of this damage in accordance with Clause 14 can provide the information for analytical work to determine the need for immediate or eventual repair. Such analysis will also identify under what conditions the platform should be shut in, or evacuated, or both. The probability of an accidental event coinciding with a design environmental event is considered to be too low for design.

The screening process shown in Figure 1 involves a preliminary assessment of accidental events for the options under study. This should be sufficiently accurate and comprehensive to ensure that:

a) realistic estimates of consequences are used in selecting the overall platform layout and risk level;

b) the levels of design actions determined in the detailed assessment are sufficiently close to those determined in Task 3 and used in setting out the basic design; any excess should be within the capabilities of practical and realistic mitigation measures.

An SCE is defined as an item of equipment whose failure in an accidental event can lead to unacceptable escalation, for example where failure of an SCE would cause a fire whose intensity and spread is such that the required fire endurance of other SCEs or the main structure cannot be achieved.

Where environmental protection is a particular consideration, the required fire endurance can be much longer than that required for personnel escape (life-safety consideration). In such instances, mitigation can involve measures to reduce the blow-down time of systems containing hazardous inventories.

Achieving an optimum design with respect to SCEs involves but is not limited to the following routes.

— Minimizing the number of SCEs by sectionalizing the hazardous components of the process system or sectionalizing the topsides layout to reduce the probability of escalation from one area to adjacent areas.

— Siting and orienting SCEs to minimize exposure to explosion wind and projectiles.

— Arranging and locating SCEs to minimize imposed accelerations and displacements of support structures, especially where SCE systems interconnect support structures that can move relative to each other.

— Where large displacements are imposed on piping systems, giving consideration to using all-welded pipe systems or increasing a specified fitting class beyond purely process requirements, so as to ensure that plastic deformation of pipe systems and nozzles can occur without prior failure of fittings.

— Reducing fire duration by reducing the blow-down times of critical systems, e.g. by draining the liquid phase at the same time as venting the gaseous phase of hazardous inventories. This can require the provision of liquid dump tanks in a suitably protected location (e.g. subsea).

— Ensuring that blow-down rates are consistent with the PFP provisions and fire intensities applied in likely escalation events, such as jet fires, so that the pressure decay always outstrips the decay in containment strength. In specifying PFP, allowance should be made for the possible impact of intermittent deluge on PFP endurance.

— Considering the interaction between the performance of SCEs and the structures that support them, or can impact them, in an explosion event. It can be necessary to classify secondary structures and equipment room envelopes as primary structure, where they are required to remain in place for the full duration of the hazardous event.

More guidance on protecting and designing SCEs can be found in Reference [21].

A.7.10.2 Evaluation of accidental situations

A.7.10.2.1 General

Task 3 in the assessment process shown in Figure 1 can require the quantification of any significant accidental actions identified, or reference to similar platform designs for which such actions have been evaluated in detail. When comparison to similar platforms is used, it can be necessary to also compare factors such as weather statistics, for example of winds that can influence gas cloud build-up and dispersion.

A.7.10.2.2 Probability of occurrence and severity of accidental events

It is usually necessary to consider a range of possible events of each accidental type; for example, a ship impact can include impacts from a small work boat, a supply boat undertaking routine operations, a contracted-in vessel undertaking specific work such as a diving support vessel, and passing ships unconnected with the installation. Each of these ships will have different masses, velocities and probabilities of impact. Similarly, for gas explosions, there can be many potential sources of gas leak and ignition, different detection equipment and different responses to detection (e.g. deluge on gas detection); all these can affect the location, magnitude and probability of an explosion.

A.7.10.2.3 Risk assessment

No guidance is offered.

A.7.10.3 Hydrocarbon incidents

Comprehensive guidance on the prevention, control and mitigation of fires and explosions is given in Reference [15]. Other guidance is provided in API RP 2FB[16]. Specific pieces of research and development work are undertaken by the Fire and Blast Information Group (FABIG) and published as technical notes; examples of these are References [17],[18],[19],[20] and [21].

All assessments should be performed in accordance with ISO 13702, which defines an overall framework for the fire and explosion assessment process. More detail can be found in References [22] and [23].

Typically, the topsides on an offshore structure are constructed as an open framework of structural shapes and tubular structural components which are relatively resistant to explosion. Decks and walls are subject to explosion actions when gas clouds are ignited; confined spaces and equipment congestion increase explosion actions.

A.7.10.4 Explosion

A.7.10.4.1 Explosion design situations and actions

A.7.10.4.1.1 General

The explosion scenario establishes the make-up and size distributions of potential vapour clouds and identifies potential ignition sources for the area being evaluated. Potential ignition sources include electrical equipment, instrumentation systems, hot surfaces and static electricity. Attention should be paid to minimize the possibilities of ignition, including earthing conductive equipment which is otherwise isolated electrically from the topsides structure. Reference should be made to ISO 13702. A general introductory guide on the subject of gas explosions can be found in Reference[18].

Further information, including possible default values for overpressures in certain areas on a platform, can be found in API RP 2FB[16]. Default values should be used only for the initial and conceptual design stage of a project by competent engineers. The effects of any likely interaction between explosion actions and the response of the structure should be considered. These can include the effects of deformation or other movement of the equipment and structural components when opening up vents, producing impact or shock loading, increasing local actions, load shedding, load redistribution and displacement and dynamic amplification, for example.

An explosion requires both a vapour cloud within the explosive limits of the mixture and an ignition source. The overpressures generated by an explosion depend on many factors, such as the type, volume and concentration of hydrocarbon released, the location of the ignition source, the degree of congestion within the compartment and the extent of confinement. In many cases, explosion actions can be reduced by active mitigation systems such as water spray and inert gas. Good natural ventilation reduces the probability of a major explosion.

A.7.10.4.1.2 Methods of analysis

The explosion overpressures on a platform can vary from near zero on a small, open platform to greater than 0,8 MPa (8 bar) on an enclosed or congested platform. Traditional simple hand calculation methods for determining explosion overpressures are not appropriate for offshore platforms. Equations that have been developed for applications on land do not always account for the significant amounts of turbulence generated as a flame front passes around equipment offshore. As a result, simple methods can significantly underpredict explosion pressures and should not be used. Techniques that can usefully be used include

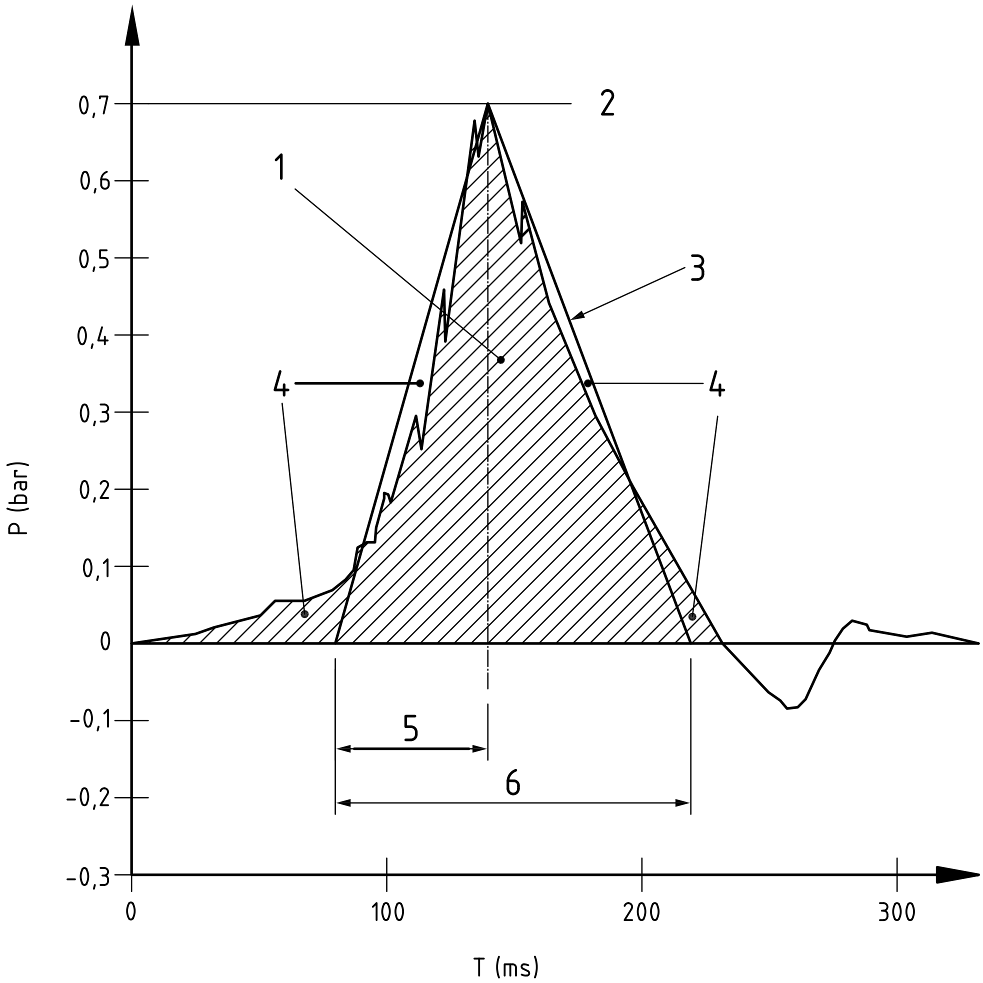
— computational fluid dynamics analysis (CFD), and

— phenomenological models.

Reference [15] presents various types of explosion models currently available for predicting explosion actions. Models should be able to predict both types of explosion actions: overpressure and drag actions. Both should be considered when designing topsides to resist explosions. The models chosen should be able to identify regions of particularly high overpressures. Presenting only one overall pressure level for an area is not likely to be sufficient. Reference [24] reports the results of full-scale test experiments and a test evaluation of a number of CFD codes and phenomenological models.

A.7.10.4.1.3 Overpressure

Explosion actions result from increases in pressure due to expanding combustion products. These actions are characterized by a pressure–time curve; an example is given in Figure A.1. Explosions actions can govern the design of many components such as explosion walls, floors and roofs. When idealizing the pressure–time curve, the important characteristics should be maintained. Such characteristics include the rate of pressure rise, peak overpressure and the area under the curve. For dynamic or quasi-static actions, it can be necessary to include the negative pressure portion of the curve.



Key

p	instantaneous overpressure, bar (1 bar = 0,1 MPa)
t	time from ignition, ms
1	impulse, $\int_0^{t_\mathrm{d}} p(t)\,\mathrm{d}t$
2	peak overpressure
3	idealized overpressure curves
4	idealized impulse (actual impulse)
5	idealized rise time
6	idealized impulse duration

Figure A.1 — Explosion overpressure versus time

A.7.10.4.1.4 Drag actions

Drag actions are caused by explosion-generated wind. The drag actions on small isolated obstacles (e.g. pipes up to 0,3 m in diameter) are a function of gas velocity squared, gas density, drag coefficient, and the cross-sectional area of the object being analysed. Critical piping, equipment and other items exposed to explosion wind should be designed to resist the predicted drag actions. Drag coefficients should be selected with due account for Reynolds number.

In the case of larger obstacles and grouped obstacles, drag actions can be increased by other effects such as inertial effects in an accelerating flow, turbulence, vortex-induced vibrations and flow stagnation at high Mach numbers (see Reference [25]). In such circumstances, actions should be calculated directly by computing the pressure differential between upstream and downstream sides or by using drag coefficients that suitably account for these factors (see References [21] and [26]).

In addition to directly applied explosion actions, concurrent actions such as self-weight and variable and operating actions should be applied to the structure. Environmental actions may be neglected in an explosion analysis. Any mass that is associated with in-place actions should be included in a dynamic analysis.

Congestion due to small structural components, piping and equipment can be a significant factor that should be considered. Explosion pressures and the nature of the explosion's behaviour cannot be accurately assessed without accounting for representative congestion in the geometry model used to analyse the topsides concept. All piping down to 25 mm in diameter and small structural components should be represented in such studies.

Simulations performed on offshore topsides projects where all piping is included have shown that explosion pressures can increase by one to two orders of magnitude compared to a bare equipment model. The effect of including piping of 50 mm in diameter and below has been seen to increase pressures significantly, sometimes by a factor of about two.

These factors should be taken into account when selecting the analytical tool and developing the geometry model.

Methodologies for generating and implementing anticipated congestion are defined in References [24] and [26].

Representative congestion can be developed from previous studies of similar platforms. The same techniques can also be adapted for re-analysis of existing platforms, when there is insufficient detailed information about small equipment.

Where a detailed equipment and structure layout is not established at the time of calculating explosion pressures, actions and consequences can be based upon assessments made using representative geometry models using anticipated congestion, or using similar geometry models, or previous studies of geometrically similar platforms.

It is therefore important when selecting a platform design to minimize congestion, i.e. the packing density of equipment. It can also be valuable to separate densely packed equipment zones by open un-congested zones, as practiced in onshore facilities. Studies have shown that 10 m open venting gaps are an effective means for reducing peak explosion pressures (see Reference [27]).

A.7.10.4.2 Structural resistance

A.7.10.4.2.1 General

The significance of risks from explosion events depends particularly on the structural barriers that are present between the area affected by the explosion and people and systems in adjoining areas. The purpose of A.7.10.4.2 is to give guidance on what should be considered when analysing a structure for explosion actions and what methods are appropriate. Connections should be assessed for their ability to allow structural components to develop their full plastic strength. Explosion actions can act in the opposite direction to gravity actions and the design of connections should take account of this. Dynamic actions cause high strain rates which, coupled with stress concentrations, can cause fracture.

In the analysis of accidental actions, all partial action factors, $\gamma_{f,G}$ and $\gamma_{f,Q}$, may be reduced to 1,0, and best estimates of yield strengths may be used. Strain rates and strain hardening effects should be included in determining yield strength and general material behaviour.

The main acceptance criteria are strength and deformation or strain limits (see A.7.10.4.2.2 to A.7.10.4.2.5).

A.7.10.4.2.2 Strength limit

Where strength governs the design, failure is defined to occur when the design value of the internal force or moment due to the design action exceeds the design resistance. The design action is determined by Formula (2) by setting the partial action factors to 1,0 and adding any accidental actions. The design resistance is determined by Formula (9) with the partial resistance factor set to 1,0. Strength design should satisfy the combination of these two equations as described in Formula (A.1).

$$F_\mathrm{d} = 1{,}0G + 1{,}0Q + 1{,}0A$$

$$S_\mathrm{d} \le R_\mathrm{D} = K_\mathrm{c} \times \frac{R_\mathrm{K,\,code}}{1{,}0} \qquad \text{(A.1)}$$

where

F_d	is the design value of the action;
G	is the permanent action;
Q	is the variable action;
A	is the action resulting from an accidental event;
S_d	is the design value of the internal force or moment due to F_d;
R_D	is the design value of the resistance;
K_c	is the building code correspondence factor (see 8.1);
$R_\mathrm{K,\,code}$	is the representative resistance of the structure or component.

A.7.10.4.2.3 Deformation limit

Permanent deformation can be acceptable following an accidental event. In such cases, the following should be demonstrated:

a) no part of the structure impinges on critical operational equipment;

b) the deformations do not cause collapse of any part of the structure that supports critical equipment, the safe area, evacuation routes or muster stations; a check should be performed to ensure that integrity is maintained if a subsequent fire occurs;

c) the deformations do not cause escalation of the event (e.g. by damaging riser integrity or emergency shut-down valve control).

Deformation limits can be based on a maximum allowable strain or an absolute displacement as discussed in A.7.10.4.2.4 and A.7.10.4.2.5. An absolute displacement can be dictated by the ductile bending and rotation capability of the structural components.

A.7.10.4.2.4 Strain limits

Generally, structural steels used offshore have sufficient toughness and are not significantly limited in strain capability at the high strain rates associated with explosion response. Reductions in strain limits can be required for cold weather applications or for steel that has low fracture toughness.

In general, tensile strain limits are related to elongation data from mill certificates or material specifications. There is no need to stipulate compression strain limits. Where resistance is limited by tensile strain, suitable limits should be placed on local and global strain. For out-of-plane bending in combination with in-plane stresses, higher surface strains can be acceptable. Where weldments or bolt holes occur in regions of high strain, the principal tensile strain should generally be limited to values that correspond to a midline tensile stress of not more than $0{,}9\sigma_\mathrm{u}$, where σ_u is the ultimate tensile strength.

When the weakening effect of bolt holes is more than 10 %, the limiting value of the principal tensile strain should be reduced. As the strain rate enhancement of σ_u is less than the enhancement at lesser strains, allowable strains are slightly lower under rapid load application. For typical structural steel grades, the principal tensile strains from a finite element analysis (FEA) should be limited to about 5 %.

The critical strain for plastic deformations of sections containing defects should be determined based on fracture mechanics methods. Welds normally contain defects and welded joints are likely to achieve lower toughness than the parent material. For these reasons structures that undergo large plastic deformations should be designed in such a way that the plastic straining takes place outside the weld. In ordinary full penetration welds, the overmatching of weld material strength relative to the parent material will ensure that minimal plastic straining occurs in the welded joints, even in cases with yielding of the gross cross-section of the structural component. In such situations, the critical strain occurs in the parent material and is dependent on

— stress gradients,

— dimensions of the cross-section,

— presence of strain concentrations,

— material yield to tensile strength ratio, and

— material ductility.

Simple plastic theory does not provide information on strains. Therefore, strain levels should be assessed by means of adequate analytical models of the strain distributions in the plastic zones or by nonlinear FEA with a sufficiently detailed mesh in the plastic zones. When structures are designed so that yielding takes place in the parent material, the value for the critical average strain, ε_{cr}, in axially loaded plate material given in Formula (A.2) can be used in conjunction with nonlinear FEA or simple plastic analysis.

For finite elements with $\frac{l}{\delta}$ W 5, the value of the critical average strain, ε_{cr}, is calculated as follows:

$$\varepsilon_{cr} = 0{,}02 + 0{,}65\frac{\delta}{l} \tag{A.2}$$

Where

l is the largest side length of a finite element model;

δ is the thickness of the finite element.

A.7.10.4.2.5 Absolute strain limits

Absolute strain limits are adopted where there is a risk of a deforming component striking another component, usually process or emergency equipment or key structural components.

It should be demonstrated that the structural component being considered can accept the deflections and deformations without failure due, for example, to local buckling or to rupture initiated at points of local stress concentration, e.g. at structural connections, welds and cut-outs. Membrane action in plating and stiffened plating can lead to increased compression in primary structural components and, by doing so, can cause buckling of these primary structural components. In floating structures, stresses in the decks due to unfavourable distributions of ballast or cargo can lead to reductions in ductile strength and out-of-plane deflection under explosion or fire actions.

Survival of deck-mounted SCE can dictate lower ductility limits for the structure in order to limit imposed deformations and acceleration of equipment supports. Similarly, the overall resistance of the topsides structure against explosions can be reduced when the wind actions associated with peak explosion overpressures reached at the ductile limit of the structure exceed the resistance of SCEs.

See References [18], [19] and [28] for more information on deformation limits.

A.7.10.4.3 Methods of analysis

A.7.10.4.3.1 General

The structural response to explosion actions can be determined by

— static and quasi-static analysis (see A.7.10.4.3.2),

— linear dynamic analysis (for non-redundant structures governed by strength limit criteria) (see A.7.10.4.3.3),

— simple calculation models based on SDOF analogies and elastoplastic methods of analysis (see A.7.10.4.4), or

— nonlinear dynamic FEA (see A.7.10.4.3.4).

Structures can be designed to respond elastically (i.e. in the elastic deflection range) or plastically, in response to explosion pressures. In the latter case, structures will be found to have resistance to higher levels of explosion. In design this can be accounted for by specifying two different explosion levels:

— a strength level explosion (SLB), being an explosion with a probability of exceedance of around 10^{-2} per year;

— a ductility level explosion (DLB), being an explosion with a probability of exceedance of around 10^{-4} per year.

These two cases are analogous to an extreme storm and an abnormal storm.

Design and analysis for SLB design situations are much easier and quicker to perform in a project time scale than an assessment to DLB, and design change and evolution can be handled more easily. A further advantage is that the code check aspect of an SLB assessment is an effective screening tool for all components of the structure, which will not necessarily be matched by nonlinear FEA. Therefore, an SLB assessment is a recommended step for all structure designs.

In design situations based on an SLB, the structure should not be permanently damaged by an explosion; however, the ultimate acceptance of the topsides structure should be based on the DLB.

The ultimate acceptance based on the DLB should demonstrate that

a) there is no sudden or progressive collapse of the overall topsides structure,

b) there is no excessive damage to SCE, e.g. by limiting deflections and acceleration of the structure (avoidance of escalation potential), and

c) there is no structural damage that significantly affects subsequent fire endurance.

For SLB design situations, SDOF methods are usually applied, coupled to a quasi-static analysis, using a linear FEA, controlled by code checks to ISO 19902 or the national or regional building standard.

For DLB design situations, SDOF methods can still be applied, providing that ductile deformation limits can be determined for the structural components and an overall characteristic load-deflection curve can be established for the topsides structure. For ductile deformation limits, literature references based on test data can be used, where available.

It can be difficult to determine the ductile deformation limits and overall characteristic load-deflection curve for complex structures, hence nonlinear FEA is often applied for a DLB assessment.

The type of structural analysis to be performed should be based on the nature of the explosion and the duration of the explosion pressure pulse relative to the natural period of the structure or component. Low overpressures can be satisfactorily considered with a linear-elastic analysis using factors to account for dynamic response. High overpressures can require more detailed analysis incorporating both material and geometric nonlinearities. The complexity of the structure being analysed will determine if a single- or a multiple-degree-of-freedom analysis is required.

If nonlinear dynamic FEA is used, all major effects described in A.7.10.4.3 and A.7.10.4.4 should either be implicitly covered by the modelling adopted or be subjected to special considerations, whenever relevant (e.g. local buckling, finite ductility, strength of connections, interaction with adjacent structure). The choice of FEA tool type, e.g. explicit/implicit program should be appropriate to the problem being studied.

A.7.10.4.3.2 Static analysis

Where actions due to explosion pressures are quasi-static (i.e. the duration of the action is long relative to the natural period of the structure or structural component being considered), static-elastic or static-plastic analysis methods can be used. The pressure derived from the assessment process in 7.10.2 and A.7.10.4.1 should be used to define the actions.

A.7.10.4.3.3 Dynamic analysis

Where the duration of an action due to explosion pressures is near the natural period of the structure, or structural component being considered, a linear or nonlinear dynamic analysis should be performed. Simplified methods using idealized pressure–time histories can be used to calculate DAFs by which static actions can be scaled to simulate the effects of inertia and rapidly applied actions. The pressure–time curve generated by a CFD analysis as part of the assessment process in 7.10.2 and A.7.10.4.1 can be applied to the structure or structural component to model more precisely the effects of the explosion.

In simple calculation models based on SDOF analysis, the structural component is transformed to a single mass-spring system exposed to an equivalent pressure pulse by means of suitable shape functions to determine the displacements in the elastic and elastoplastic range.

For any arbitrary pressure pulse, the maximum response for the SDOF model is generally obtained by numerical step-wise integration of the differential equation or by Duhamel integration. Provided that the temporal variation of the pressure can be assumed to be triangular, the maximum displacement of the component can be calculated from design charts for the SDOF system (see References [17] and [30]) as a function of pressure duration versus fundamental period of vibration and equivalent explosion pressure amplitude versus maximum resistance in the elastic range. The maximum displacement for both the primary and rebound response should comply with ductility and stability requirements for the structural component; for charts for rebound response, see Reference [18].

The response of a structural component can conveniently be classified into three categories according to the duration of the explosion pressure pulse, t_d, relative to the fundamental period of vibration of the component, T.

— In the impulsive domain, $t_d/T < 0,3$, the maximum displacement is governed by the explosion impulse:

$$I = \int_0^{t_d} p(t)\,dt \tag{A.3}$$

— In the dynamic domain, $0,3 < t_d/T < 3$, the response is solved from integration of the dynamic equilibrium equations.

— In the quasi-static domain, $3 < t_d/T$, the maximum displacement is governed by the peak pressure, p_{max}, and the rise time of the pressure relative to the fundamental period of vibration of the structure or structural component under consideration. If the rise time is large, i.e. if t_d/T is much greater than 3, the maximum deformation of the component can be solved from static equilibrium. If the rise time is small, i.e. if t_d/T is closer to 3, a dynamic magnification will be present.

In the near field the gas explosion pressure impulse has a finite rise time, typically 30 % to 70 % of the impulse duration, but in the far field the pressure rise is usually instantaneous.

Further guidance on structural design for explosion can be obtained from References [18] and [19]. Guidance on design of equipment for explosion actions is given in References [28] and [31].

A.7.10.4.3.4 Nonlinear finite element analysis

Where nonlinear FEA is used for dynamic analysis, the type of program selected (implicit or explicit) should be suitable for the type of structure being analysed and the potential local and global actions expected. Due to the practical limitations of modelling large complex structures in sufficient detail, the equivalent of a full code check, as used in linear-elastic analysis, is not normally carried out within the nonlinear FEA code. In many instances, it is necessary to perform additional code checks according to the recognized national or regional building standard, using forces and stresses generated from the nonlinear FEA code. Undertaking analysis of complex structures using nonlinear FEA requires a detailed understanding of the potential failure modes of the structure and the contribution of coexisting operating actions to component utilization. In nonlinear FEA, overall modelling accuracy can be checked by comparing this case with the results of the same case in the linear-elastic analysis.

The nonlinear FEA model used should contain initial imperfections of sufficient magnitude to trigger critical local and global failure modes. Initial displacements can be introduced by using distorted coordinates or induced by functional actions. Eigenmodes determined in linear buckling analysis do not always account for sufficient imperfections at all the required locations. In place of more accurate information, imperfections should be based on fabrication tolerances.

In conjunction with the modelling of imperfections, it should be ensured that the modelling of beams can allow torsional buckling behaviour.

When performing nonlinear FEA, a sufficient number of explosion load cases and sufficient simulation duration should be considered to ensure that the envelope of explosion scenarios is covered by the analyses undertaken.

The documentation of the nonlinear FEA work should include the results of code checks and a statement of the allowed permanent explosion damage (if any), so that structural input to fire response analysis can be consistent with output from the explosion analysis.

Further information on the use of nonlinear FEA techniques can be found in ISO 19902.

A.7.10.4.4 Simple calculation methods

A.7.10.4.4.1 Component resistance by simplified methods

Simplified methods can be used for the design of structural components as described below.

a) **Deck plating and stringers**

   1) Main deck girders rely on the deck plating and the secondary steelwork supporting the plating (stringers) for lateral and torsional restraint; the plating and stringers can assist in redistribution of loads and load paths in accidental situations. Deck plating and stringers should have higher design explosion pressure strength than the girders that support them; a wide spatial variation in explosion pressure in an area can be expected, so average pressures (applicable for the design of the deck girder) will be less than the local peak pressures (applicable to the deck plating and stringer design).

   2) Where deck plating and stringers are expected to fail prior to the girders that support them, the impact of the failure modes of the stringers should be considered when assessing the strength and ductile deflection limit of the girders.

   3) The combined effect of these factors is that a tradition has grown in some countries (e.g. Norway) for deck plating and stringer arrangements with at least two to three times the quasi-static explosion pressure resistance of the girders that support them. This leads to a deck design with enhanced reserve ductile deformation capability and improved performance in fire.

4) Elasto-plastic resistance can be fairly well determined from elastic and rigid-plastic methods. For plates continuously loaded over several spans, clamped boundary conditions can be assumed. It is always conservative to assume no restraint against inward displacements. If the beneficial effect of membrane forces is taken into account, the ability of the adjacent structure to anchor the membrane forces should be demonstrated. The flexibility of the adjacent structure can delay the build-up of membrane forces. A simplified method to quantify the effect of this flexibility on the basis of a plate strip analogy is given in Norsok N-004[32]. Finite ductility should be taken into account. In most cases, plate resistance is not the limiting factor, the stiffeners will collapse before the plating reaches its critical deformation.

5) For plate stringers, a beam-type idealization is often appropriate. It should, however, be demonstrated that the stringer does not undergo significant tripping undermining its bending resistance. Provided that the connections and the adjacent structure can anchor the generated forces, the beneficial effect of membrane forces in the large deflection range can be accounted for, so long as the reduction in bending strength resulting from the coexisting membrane stress in the stringer is also accounted for. Rupture due to excessive straining should also be considered.

b) **Beam or girder**

1) Resistance relationships of beams for the elastic, elastoplastic, and rigid-plastic domain, based on SDOF models can be found in Norsok N-004[32] and Reference [18]. Beams and girders with slender cross-sections should be checked for local failure in shear and bending. The tension field concept can be used to determine ultimate resistance. Although resistance in the post-ultimate region can be significant, information to allow the engineer to make use of this effect is limited. Shear deformations can have a significant impact on the response for beams and girders with small length/height ratios and clamped boundary conditions.

2) Deck girders often act with associated deck plates as a composite section, which causes an upward shift in neutral axis position. While this increases section modulus, it can also alter the section class and ductile bending resistance. Similarly, membrane forces in adjacent deck plating can cause axial compression force in girders. This can reduce bending moment resistance. Other effects, such as transverse deck stringers and cut-outs for transverse stiffeners can stabilize the compression flange against torsion and can affect section class and ductile bending resistance.

3) In hogging moment regions, lateral stabilization of the bottom flange is required at suitably frequent intervals to prevent lateral buckling prior to development of the full section moment resistance. In offshore topsides, deck girders can be subjected to lateral explosion wind actions.

4) Where girders act as pipe supports, lateral actions due to explosion wind on the supported pipes and cable racks can lead to significant additional lateral destabilizing actions on girders. If time-domain dynamic analysis of girders is performed without considering these additional actions, lateral instability modes can be missed, with consequent unconservatism in the analysis results.

5) Reference [18] gives guidance and worked examples on deck girders designed by manual methods. Reference [22] gives some limited guidance on the analysis of deck girders by nonlinear FEA.

A.7.10.4.4.2 Ductile deflection limits and local buckling

The maximum deformation the structural component can undergo is ultimately limited by local buckling on the compressive side or by fracture on the tensile side of cross-sections undergoing finite rotation. If the structural component is restrained against inward axial displacement, any local buckling occurs before the tensile strain due to membrane elongation overrides the effect of the compressive strain induced by rotation. If local buckling does not occur, further deflection can occur until fracture is assumed to occur, when the tensile strain due to the combined effect of rotation and membrane elongation exceeds a critical value. To ensure that structural components with small axial restraint maintain sufficient moment resistance during significant plastic rotation, cross-sections should be proportioned to class 1 requirements as defined in References [18] and [19]. Ductility limits for beams proportioned to class 1 are limited by the onset of local buckling (for further guidance, see Reference [33]). Simplified formulae extracted from this reference are contained in Reference [18].

The effective flange of the stiffened deck plating should be evaluated to allow calculation by SDOF methods. Norsok N-004[32] gives recommendations on effective deck plating, depending on whether the plate field is elastic (shear lag effect) or can undergo buckling (effective width concept for post-buckling resistance). Initiation of local buckling does not necessarily imply that the resistance with respect to energy dissipation is exhausted, particularly for class 1 and class 2 cross-sections. The degradation of the cross-sectional resistance in the post-buckling range can be taken into account where this information is available. Alternatively, for beams and plates with full or partial restraint, the limiting deformation at tensile fracture is given in Norsok N-004[32].

A.7.10.4.4.3 Support reactions, beam releases and non-fixed connections

In order to prevent structural component failure in shear at the supports preceding ductile bending failure, design support reactions for structural components should be enhanced by a minimum of 20 % compared to theoretical values to allow for the structural component resistance being higher than assumed in the response analysis.

Non-fixed joints at stringer connections to girders should not generally be specified where the stringers are otherwise continuous across the girder (i.e. there is no strength continuity in the bottom flange). However, such moment releases can be useful for enhancing the pattern of load distribution in deck structures, especially at overall deck deflections beyond the elastic limit. Deliberate releases can indicate lower tolerance for ductile rotation in regions of sagging moment.

A.7.10.4.4.4 Material properties for design

Strain rate affects yield strength and flow stress. Reference [21] gives the relationship between strain rate and strength enhancement for a range of carbon and stainless steels. Strain-rate-induced strength enhancement is beneficial in terms of increased strength, but can be detrimental in terms of section class and ductile deformation capability. It is important to use appropriate values of strain rate and enhancement. Reference [21] gives some typical straining rates for different structural situations

For design of new topsides, a minimum specified yield strength is normally used, but “probable” material strength can be used in place of specified minima, where such data exists. The strength values should be based on material tests for each component (e.g. mill certificates) or, where these are not available, the 90 % exceedance values from appropriate generic mill data should be used.

A.7.10.4.5 Explosion mitigation

Explosion effects can generally be minimized by

— making the vent area as large as possible,

— making sure the vent area is well distributed,

— concentrating on the layout, size and location of internal equipment, and

— using explosion barriers.

Active suppressant and mitigation systems can be used to minimize explosion effects in appropriate circumstances.

To minimize explosion pressures, vent areas should be located as close as possible to likely ignition sources. Consideration should also be given to the effect of vent area size and location on combustible cloud development. It is desirable to keep equipment, piping, cable trays, etc. away from vent areas to minimize the drag actions on these items and to fully use the vent area provided. Explosion relief panels and louvres can provide extra venting during an explosion, although these will not be sufficient where high overpressures are predicted. Relief panels should be designed to open rapidly at very low pressures to be effective in reducing the overpressures. Given that the use of relief panels is based on tests in medium scale and at low equipment congestion, an analysis of their effect on risk should be undertaken, particularly when they are used to close otherwise open walls. Although the pressures needed to open the relief panels are best kept low for relief of explosion pressures, they should not be so low as to allow wind to blow open the panels, e.g. 0,005 MPa (0,05 bar).

NOTE Wind pressures are at least an order of magnitude lower than explosion pressures.

Explosion walls and floors can be used to separate parts of a topsides, so an explosion within one area will not affect adjacent areas. This approach requires that the explosion walls and floors can withstand the design overpressures without being breached. Failure of these structures can generate primary projectiles and result in possible escalation. Explosion walls and floors generally double as fire walls and floors and should therefore maintain integrity after the explosion. Any passive fire protection attached to the wall or floor should function as intended after an explosion; alternatively, the loss of such fireproofing should be accounted for in the design.

In an explosion, pressure waves radiate out from the immediate area of the explosion becoming explosion waves that can affect persons and facilities in the far field. Such waves are typically of short duration and very dynamic with significant underpressure phases. Where explosion waves are reflected, pressures can be augmented increasing applied actions and mortality and injury rates for persons in such zones (see References [15] and [22]).

Where applicable, the actions due to explosion waves should be evaluated and used for the design of facilities and temporary refuges in the far field. Guidance can be obtained from References [15] and [34].

Further guidance on design of explosion mitigation systems, including explosion relief panels, can be found in Reference [17].

A.7.10.5 Fire

A.7.10.5.1 Fire design situations

The factors relevant to the assessment of the effects of a fire include

— the fire scenario, including duration,

— the heat flow characteristics from the fire to unprotected and protected steel structural components,

— the properties of the material at elevated temperatures, and

— the characteristics of any fire protection systems (active and passive).

The fire scenario establishes the fire type, location, geometry and intensity. The fire type will distinguish between a hydrocarbon pool fire, a hydrocarbon jet fire, or other, generally less significant, types of fire. The location and geometry of the fire determines the relative position of the heat source to the structure, while the intensity (thermal flux) determines the amount of heat emanating from the heat source. Structure and equipment engulfed by flames are subjected to a higher rate of thermal actions than those that are not engulfed. The fire scenarios can be identified during process hazard analyses.

The heat flow from the fire into structural components (by radiation, convection and conduction) is calculated to determine the temperature of each component as a function of time. The temperature of unprotected components engulfed in flames is dominated by convection and radiation effects, whereas the temperature of protected components engulfed in flames is dominated by the thermal conductivity of the insulating material. The amount of radiant heat arriving at the surface of a component is determined using a geometrical configuration or view factor. For engulfed components, a configuration factor of 1,0 is used.

The thermal and mechanical properties of the structural materials at elevated temperatures are required. The thermal properties (specific heat, density and thermal conductivity) are required for the calculation of the material temperature. The mechanical properties (expansion coefficient, yield strength and Young's modulus) are used to verify that the original design still meets the strength and serviceability requirements. Actions induced by thermal expansion can be significant for highly restrained components and should be considered.

A.7.10.5.2 Fire actions

Predictive techniques for the fire process are often classified as

— empirical models,

— zone (phenomenological) models,

— computational fluid dynamics (CFD), or

— field models (Reference [22]).

Empirical models can yield accurate and reliable predictions provided that conditions are similar to those in the underlying experiments. Examples of empirical models are the standard temperature–time curves for cellulosic fires and hydrocarbon fires. Zone models represent more of the governing phenomena, but the equations are limited to one dimension (the equations express the conditions in each zone and the fluxes present on the boundaries between the zones). Neither the empirical nor the zone models have the capability to model and predict the combustion process. CFD models analyse the problem in three dimensions, in either a steady-state or transiently, by applying basic principles such as conservation of mass, momentum and energy, supplemented by models for turbulence generation and dissipation, soot formation and the chemical reactions associated with the combustion. Suitable models for fire prediction are applicable for well-defined fuels or burning materials such as gas and oil, but less suitable for materials for which the combustion process is not well established (e.g. wood, building materials, etc.) The outcome of a CFD analysis is, in this context, radiative and convective heat flux to surrounding structures, and also smoke production and movement.

CFD analysis provides the most fundamental understanding of the processes involved and has the greatest potential, but is very challenging with respect to both demand for computer resources and mathematical modelling. Significant progress has been made in recent years and the scope of successful application expands. Simplified methods and FEA can be used where appropriate. Where PFP is applied, rigorous modelling of a numerical solution can become very difficult due to the thermal properties of the structural material and PFP differing by an order of magnitude. An equivalent heat transfer coefficient should be used, which has been derived from a value measured in tests.

Examples of the effects on the stress/strain characteristics of ASTM A36 and ASTM A633 Grade C and D steels at elevated temperatures are presented in Table A.1 and Figure A.2 (taken from Reference [31]) for temperatures in the range 20 °C to 900 °C.

The interpretation of these data to obtain representative values of temperature effects on yield strength and Young’s modulus should be performed at a strain level consistent with the design approach used:

— for a design approach that does not permit some permanent set in steelwork after the fire load case has been removed, a strain of 0,2 % should be assumed;

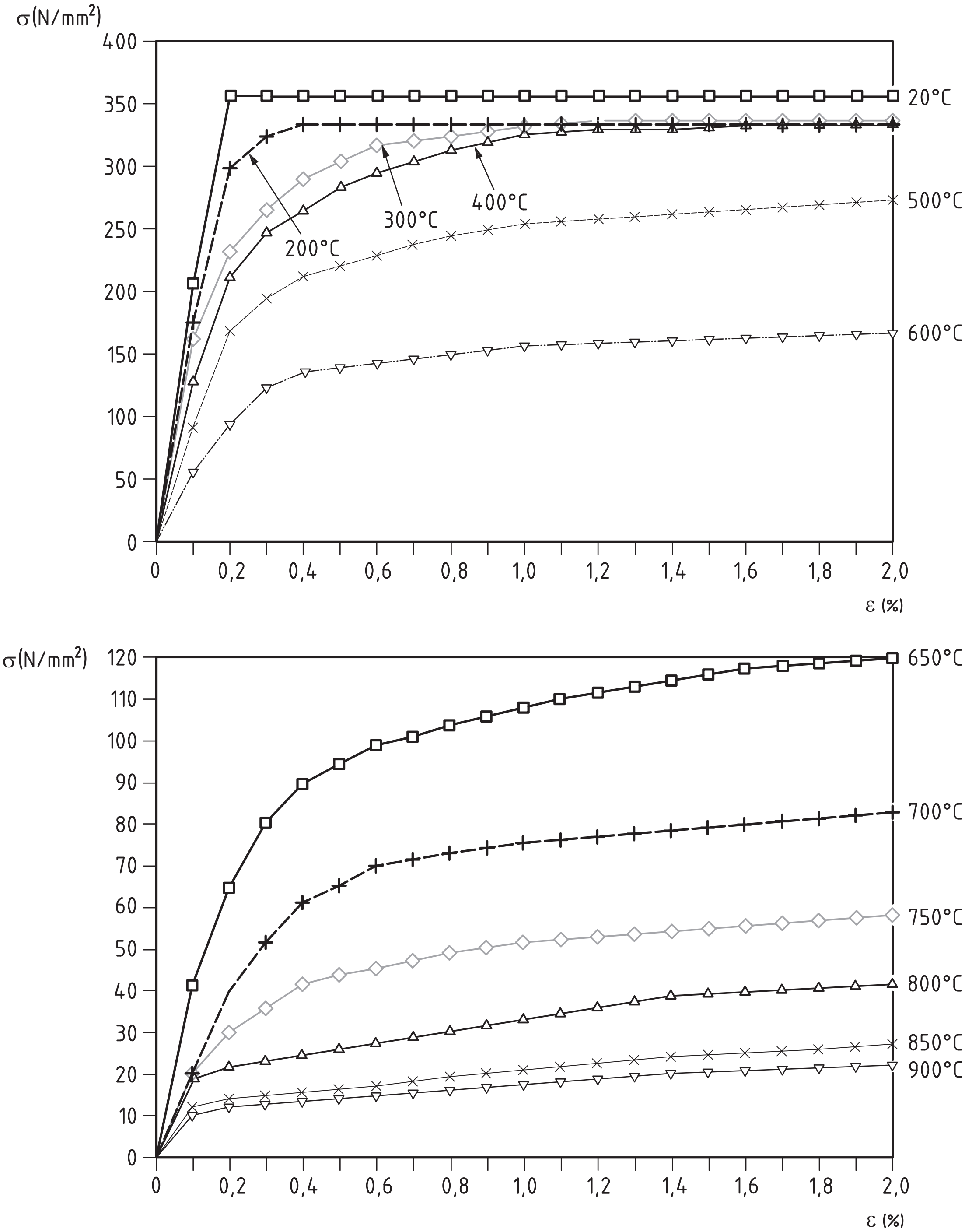
— for a design approach that allows some permanent set in steelwork after the fire load case has been removed, higher values of strain can be appropriate (0,5 % to 1,5 %).

At temperatures above 600 °C, the creep behaviour of steel can be significant and should be considered (see Reference [20]).

This part of ISO 19901 does not include any thermal data for other structural materials (see Clauses 10 and A.10).

Table A.1 — Yield strength reduction factors for steel at elevated temperatures (ASTM A36 and ASTM A633 Grades C and D)

Temperature °C	Strain 0,2 %	Strain 0,5 %	Strain 1,5 %	Strain 2,0 %
100	0,940	0,970	1,000	1,000
150	0,898	0,959	1,000	1,000
200	0,847	0,946	1,000	1,000
250	0,769	0,884	1,000	1,000
300	0,653	0,854	1,000	1,000
350	0,626	0,826	0,968	1,000
400	0,600	0,798	0,956	0,971
450	0,531	0,721	0,898	0,934
500	0,467	0,622	0,756	0,776
550	0,368	0,492	0,612	0,627
600	0,265	0,378	0,460	0,474



Key

ε strain, as a percentage

σ stress, in newtons per square millimetre

Figure A.2 — Stress versus strain relationships of typical structural steels at different temperatures

A.7.10.5.3 Design for fire

Fire load cases can be addressed using one of the following approaches.

a) **Zone method**

The zone method assumes that each component is fully utilized in normal situations and that, for structural steel, the combined total factor of safety results in a stress level of 60 % of yield strength for normal conditions (without a fire). The maximum temperature that can be tolerated is therefore taken as the temperature at which the yield strength reduces to 60 % of that at normal temperatures. This results in a maximum allowable temperature for structural steel of 400 °C; PFP should be applied to keep all structural steel below this temperature, the thickness of the PFP depending on the applied heat flux and the properties of the PFP.

The proportion of normal yield strength and the yield strength reduction is different for other materials and appropriate values should be determined and used.

Higher temperatures can be tolerated if higher strain rates are accepted (see Table A.2), but in these cases the reduction in steel stiffness should be taken into account by analysis, thereby detracting from the simplicity of this method.

Table A.2 — Maximum allowable steel temperature as a function of strain for use with the zone method

Strain %	**Maximum allowable temperature** °C
0,2	400
0,5	508
1,5	554
2,0	559

b) **Linear-elastic method**

For the linear-elastic method, a maximum allowable temperature in a steel component is determined from the stress level in the component prior to the fire, such that, as the temperature increases, the component's utilization ratio (UR) remains below 1,00, i.e. the component continues to behave elastically. For those components that do not suffer a buckling failure, the allowable stress should be such that the extreme fibres on the cross-section are at yield. The yield strength should correspond to the average core temperature of the component. For example, the maximum allowable temperature, $T_{\mathrm{C,max}}$, in a steel component as a function of UR is presented in Table A.3 for a 0,2 % strain limit.

Table A.3 — Maximum allowable steel temperature as a function of utilization ratio (UR)

Maximum component temperature $T_{\mathrm{C,max}}$ °C	**Yield strength reduction factor at** $T_{\mathrm{C,max}}$	**Structural component UR at** 20 °C **to give UR of** 1,00 **at** $T_{\mathrm{C,max}}$
400	0,60	1,00
450	0,53	0,88
500	0,47	0,78
550	0,37	0,62
600	0,27	0,45

At higher temperatures, the reduction in Young's modulus can be greater than the reduction in yield strength and care should be taken to ensure that failure modes (particularly those including any form of buckling) are not changed at these higher temperatures.

c) **Elasto-plastic method**, e.g. a progressive collapse analysis

For the elasto-plastic method, a maximum allowable temperature in a steel component is assigned based on the stress level in the component prior to the fire, such that, as the temperature increases, the component UR can go above 1,00, i.e. the component's behaviour is elasto-plastic. A nonlinear analysis should be performed to verify that the structure does not collapse and does still meet the serviceability criteria. Such an analysis should include the effects of temperature-dependent stress/strain behaviour and creep and be able to accommodate large deflections and large strains.

The linearization of the nonlinear stress/strain relationship of steel at elevated temperatures is necessary for an elasto-plastic analysis programme that does not permit temperature-dependent stress/strain curves to be input. Data for such relationships can be obtained from Reference [22].

A.7.10.5.4 Transient heat transfer analysis

The flow of heat from the fire into the structural component (by radiation, convection and conduction) is calculated in a transient heat transfer analysis. The analysis can be performed by

— simplified methods (see BS EN 1993-1-2[35]), or

— finite element analysis (FEA).

The thermal properties of the structural material, specific heat, density and thermal conductivity are required for the calculation of its temperature. Temperature-dependent properties or equivalent constant values can be used (see BS EN 1993-1-2[35]). Internal radiation from warm to cold surfaces should be considered for hollow sections and open sections with significant mutual-view factors. The effect of PFP should be included in the transient heat transfer analysis. Rigorous modelling of PFP is numerically difficult, due to very different thermal diffusivity of PFP and the structural material and the often complex physical behaviour of the PFP. Instead, the performance of PFP can be described by an equivalent heat transfer coefficient. The equivalent heat transfer coefficient can be derived from fire-proofing tests. It depends on the product used, is thickness-dependent and represents the average protection offered by the PFP (regardless of the physical processes involved) in the steady-state condition. The type of PFP and the thickness of application are specified for both the type and intensity or the fire and the duration for which the PFP has to remain effective; if the fire is still burning after this duration, the PFP should be assumed to be ineffective.

Large strains can be acceptable where permanent deformations can be allowed. For a component-based design approach, the effective yield strength can, for carbon steel, be taken as equal to the yield strength at 2,0 % strain. In nonlinear FEA-based design, in place of more accurate values, the yield strength should be assumed constant from 2 % strain up to the ultimate strain limit.

A.7.10.5.5 Creep

At temperatures above 600 °C, the creep behaviour of steel can be significant. The yield strength reduction factors implicitly take some degree of creep into account. Considering the relatively short fire duration, the explicit evaluation of creep may be omitted in most situations. However, if a vital compression component in a non-redundant structure is close to its critical temperature for a substantial time (significantly larger than 20 min), the effect of creep should be given explicit consideration.

Structural analysis can be performed on different structural components or systems including

— individual structural components,

— subassemblies, or

— an entire system.

The assessment of action effects and mechanical response due to fire should be based on either

a) simple calculation methods applied to individual structural components,

b) nonlinear FEA, or

c) a combination of simple and nonlinear methods.

Simple calculation methods can give overly conservative results. Nonlinear FEA allows simulation of the fundamental processes in a realistic manner. Assessment of individual structural components by means of simple calculation methods can, for example, be based upon the provisions given in BS EN 1993-1-2[35]. Assessment of ultimate strength of carbon steel is not needed if the steel maximum temperature does not exceed 400 °C.

A.7.10.5.6 Nonlinear finite element analysis

A.7.10.5.6.1 General

Structural analysis methods for nonlinear ultimate strength assessment can be classified as

— stress–strain-based methods, or

— stress-resultant-based (yield/plastic hinge) methods.

Stress–strain-based methods are methods where nonlinear material behaviour is accounted for on fibre level. Stress-resultant-based methods are methods where nonlinear material behaviour is based on closed-form solutions for interaction equations for cross-sectional forces and moments.

A.7.10.5.6.2 Material modelling of carbon steels

In stress–strain-based analysis of carbon steel structures, the temperature-dependent stress–strain relationships given in Figure A.2 may be used.

For stress-resultant-based design, the temperature reduction of the elastic modulus may be taken from BS EN 1993-1-2[35].The yield strength temperature reduction can be taken as equal to the yield strength at 2 % strain (see Table A.1).

A.7.10.5.6.3 Initial out-of-straightness

In nonlinear FEA, the model should contain an initial out-of-straightness of members of sufficient magnitude to trigger all relevant local and global failure modes that can become critical. Such initial out-of-straightness can be introduced by distorted coordinates or induced by functional actions. Eigenmodes determined in linear buckling analysis do not necessarily provide sufficient imperfections for all required locations. In place of more accurate information, the out-of-straightness should be taken as

— 1,0 times fabrication tolerance levels if cross-sectional temperature gradients are accurately simulated, or

— 2,5 times fabrication tolerance levels if cross-sectional temperature gradients are not accurately simulated.

The initial out-of-straightness should be applied on each physical structural component. If the component is modelled by several finite elements, the initial out-of-straightness should be applied as displaced nodes. The initial out-of-straightness should be applied in the same direction as the deformations caused by the temperature gradients.

A.7.10.5.6.4 Local cross-sectional buckling

If shell modelling is used, it should be verified that the software and the modelling is capable of predicting local buckling with sufficient accuracy. If necessary, local shell imperfections should be introduced in a similar manner to the approach adopted for lateral distortion of beams in A.7.10.5.6.3.

If beam modelling is used, local cross-sectional buckling should be given explicit consideration.

In place of more accurate analysis, cross-sections subjected to plastic deformations should satisfy compactness requirements:

— Class 1: locations with plastic hinges (approximately full plastic utilization);

— Class 2: locations with yield hinges (partial plastification).

If this criterion is not satisfied, the effects of plastic deformations should be explicitly considered. The strength will be reduced significantly after the onset of buckling, but can still be significant. A conservative approach is to remove the component from further analysis.

Compactness requirements for Class 1 and Class 2 cross-sections can be disregarded provided that the component develops a significant membrane tension as it undergoes finite displacements.

A.7.10.5.6.5 Strain limits

The ductility of beams and connections increases at elevated temperatures compared to normal conditions. Limited information exists. In place of more accurate analysis, the provisions given for structural components subjected to explosions should be followed in addition to the following.

a) **Tensile members**

In place of more accurate analysis, an average elongation of 3 % of member length for a reasonably uniform temperature may be assumed. Local temperature peaks can localize plastic strains. It is considered conservative to use critical strains for steel under normal temperatures.

b) **Connections**

In place of more accurate calculations, the strength of the connection at a temperature θ may be taken as:

$$R_\theta = k_{\mathrm{y},\theta} R_0 \tag{A.4}$$

where

R_θ is the strength of the connection at the maximum temperature, θ;

R_0 is the strength of the connection at normal temperature;

$k_{\mathrm{y},\theta}$ is the yield strength reduction factor for the maximum temperature, θ, in the connection (see A.7.10.5.2).

A.7.10.5.6.6 Robustness of calculation

In view of the uncertainties implicit in the fire process, the transient heat transfer and the mechanical response, the robustness of nonlinear FEA calculations should be checked by increasing the functional actions at the most critical time during the fire. If the structure remains intact for a 10 % increase of the functional actions, the structure may be considered to have a sufficient global resistance against the fire effects, in other cases a more rigorous analysis, such as elasto-plastic, should be undertaken.

NOTE Other criteria can be governing.

For the elasto-plastic method, a maximum allowable temperature in a steel component is determined based on the stress level in the component prior to the fire, such that, as the temperature increases, the component's UR can go above 1,00, i.e. the component's behaviour is elasto-plastic. A nonlinear analysis should be performed to verify that the structure does not collapse and still meets the serviceability criteria.

A.7.10.5.7 Fire mitigation

In the event of a fire, mitigation can be provided by active and passive fire protection systems to ensure that the maximum allowable component temperatures are not exceeded for a designated period. The active and passive fire protection systems can also inhibit escalation of a fire. The designated period of protection should be based on either the fire's expected duration or the required evacuation period, whichever is shorter, and is used to specify the materials and thicknesses of application.

PFP materials comprise various forms of fire-resistant insulation products that are either used to envelope individual structural components or are used to form fire walls that contain or exclude fire from compartments, escape routes and safe areas. Ratings for different types of fire protection are obtained from testing using a set time–temperature heating curve and are presented in Table A.4. These ratings are applicable to PFP materials subject to pool fires. Special consideration should be given to the application of PFP materials for jet fire service (see ISO 22899-1[36]). The minimum coat-back (protection given to components that can be exposed to the fire but that are not primary structure, measured from the connection with the primary component) should not be less than 150 mm. Particular attention should be given to ensuring adequate protection to beams supporting gratings and to the supports to safety-critical equipment.

Table A.4 — Summary of fire ratings and performance for fire walls

PFP rating[a]	Period required for stability and integrity performance to be maintained min	Period required for insulation performance to be maintained min
H120	120	120
H60	120	60
H0	120	0
A60	60	60
A30	60	30
A15	60	15
A0	60	0
B15[b]	30	15
B0[b]	30	0

[a] The classification system consists of two elements. The first element is a designation of the type of fire: "H" for a hydrocarbon fire; "A" and "B" for cellulosic fires; and (where used) "J" for a jet fire. The second element describes the required minimum protection time, in minutes. The intensity of the "H", "A" and "B" fires are described in Reference [15] and ISO 834-1[37] and that of "J" fires is described in ISO 22899-1[36].

[b] A "B" rating is not commonly used on offshore platforms, except on some occasions with accommodation units. "B"-rated fire barriers are not required to prevent the passage of smoke.

Active fire protection can be provided by water deluge, foam and, in some instances, by fire-suppressing gas that is delivered to the site of the fire by dedicated equipment pre-installed for that purpose.

Maintaining stability and integrity requires that the passage of smoke and flame be prevented and the temperature of load-bearing components not exceed 400 °C. Maintaining insulation performance requires that the average and maximum temperature rise of the unexposed face be limited to 140 °C and 180 °C, respectively, for the specified period.

A.7.10.6 Explosion and fire interaction

A.7.10.6.1 General

In many circumstances conflicts arise between fire and explosion engineering (see Reference [15]). For example, in order to resist a fire, the structure can be segregated into small zones using fire walls to contain the fire. However, this segregation can result in an increase of overpressure if an explosion occurred. To reduce explosion overpressures, the confinement should be reduced. This requires open modules with unobstructed access to the outside. This creates a direct conflict with the fire containment scheme. These conflicts should be considered when designing the topsides.

Fire and explosion assessments should be performed together and the effects of one on the other should be carefully considered. It is more likely for an explosion to occur first and be followed by a fire. However, it is possible that a fire could be initiated, which then causes an explosion. The iteration process required between the fire and explosion assessment is shown in Figure 2. Fire and explosion assessments should demonstrate that the escape routes and safe areas survive the fire and explosion scenarios.

A.7.10.6.2 to A.7.10.6.6 describe practical considerations for designing a structure to resist fire and explosion actions.

A.7.10.6.2 Deck plating

During fire and explosion actions, deck plating can impose lateral forces rather than restraint on deck structural components. Care should be taken in structural modelling of deck plates.

In general, the deck should be analysed as a series of beams. The effective width of deck plates can affect the calculation of deck natural period and should be included. Plated decks can generally be allowed to deform plastically in the out-of-plane direction provided that adequate performance of the primary support structure is demonstrated.

A.7.10.6.3 Explosion and fire walls

Designs should allow as large a displacement as possible at mid-span; however,

a) fire protection should be able to maintain integrity at the required strain,

b) member shortening under large lateral displacements can impose severe actions on top and bottom connections, and

c) the rotational capacity of the end connections should be sufficient, without prior rupture.

Piping, electrical or heating, ventilation and air conditioning (HVAC) penetrations should be located as near as possible to the top or bottom of the wall at locations of low predicted deformations (strains). However, for explosion pressures, reinforcement of any penetrations can be appropriate to ensure that wall strength and deflection capability are not compromised.

A.7.10.6.4 Beams and connections

Structural components acting primarily in bending can experience significant axial actions in fire and explosion situations. These axial actions can affect the strength and stiffness of the structural component. Any additional bending moment caused as a result of the axial action and lateral deflection should be considered in either elastic or plastic analyses.

Axial restraints can result in a significant axial force in the member caused by transverse actions being partially carried by membrane action. The effects of these actions on the surrounding structure should be taken into account.

Under the temperature effects of fires, beams can change from resisting actions through bending and shear to resisting the actions by displacing and developing tension. This effect can be exploited to provide significantly greater resistance by designing connections to withstand membrane behaviour. Connections should be designed to the ultimate plastic capacity of the beam under the fire and explosion loading scenario in bending as well as in axial compression or tension.

Both local and overall beam stability should be considered when designing for explosion actions. When considering lateral buckling, it is important that compression flanges be supported laterally. An upward action on a roof beam can put a normally unrestrained bottom flange into compression.

NOTE Explosion actions can act in reverse direction from the normal design actions.

A.7.10.6.5 Slender structural components

Slender members can be prone to premature buckling during fire actions. If used, suitable lateral and torsional restraint should be provided.

Deck plating during fire and explosion actions can cause lateral actions rather than restraint.

NOTE The classification of members and parts of members as “slender” is controlled by the slenderness ratio and by the ratio of yield strength to Young’s modulus.

A.7.10.6.6 Pipe and vessel supports

Pipe and vessel supports can attract large lateral actions due to explosion wind, or thermal effects, or both.

Vessel supports should remain integral at least until process blow-down is complete. The supports for vessels containing flammable liquids should remain integral for sufficient time to allow platform evacuation.

Stringers to which equipment is attached can have significantly different natural periods than the surrounding structure. Their dynamic response should be assessed separately. Further guidance can be obtained from References [19] and [28].

A.7.10.7 Vessel collision

No guidance is offered.

A.7.10.8 Dropped and swinging objects and projectiles

In general, design for dropped and swinging objects and for projectiles involves the following stages:

— determining scenarios for possible dropped and swinging objects and projectiles, including the dimensions, masses and velocities of objects;

— detecting the most likely progressive collapse mechanisms that can be caused by a swinging or falling object (e.g. global structural collapse, local impact on high-pressure pipework, etc.);

— checking whether the object has sufficient energy to trigger a collapse mechanism where no barrier structures exist;

— where there are barrier structures, checking whether the object is sufficiently arrested or retarded by the barrier to avoid triggering a collapse mechanism, and checking the load transfer mechanisms between the barrier structure and the topsides primary structure;

— checking if any damaged structure can resist the functional actions and the environmental actions with a return period reflecting the time to allow a repair to be effected: in place of other information and further analysis, a 10-year return period environmental event should be used.

A.7.10.9 Loss of buoyancy

No guidance is offered.

A.7.10.10 Strong vibration

Strong vibration is a shaking of the topsides structure of a platform. There can be a large overall movement of the topsides from side to side. The effects can be evident as shock damage. In the case of a gas explosion on a topsides module, for example, venting through one side of a module can cause a large out-of-balance action on the topsides structure leading to large horizontal deflections, together with accelerations of local structures, possibly including the accommodation and helideck. Even if there is no significant overall effect, considerable vibration of the topsides can occur in the higher modes.

During an earthquake, a fixed offshore structure can move vertically as well as horizontally. The structure is initially at rest, apart from movements due to waves and normal operations, until the movement of the ground begins to shake the base of the legs. Ground movement can continue for 20 s or more. Earthquakes have little effect on floating structures and can generally be discounted; however, vertical ground accelerations can affect TLPs and possibly FPSOs with taut leg moorings.

For topsides gas explosion or ship collision events, the acceleration or impact will be at a higher level in the structure, and the duration of the applied action will be shorter. For explosion actions, the pressure pulse is likely to last for less than 1 s, although a structure can continue to vibrate for some time after the initiating event.

Safety-critical systems can include auxiliary diesel generators, emergency fire pumps and firewater ring mains, electrical control panels and cabling. Vibration mountings for equipment have limited ability to resist strong vibration actions, which can result in large lateral displacements.

A.7.11 Other actions

No guidance is offered.

A.8 Strength and resistance of structural components

A.8.1 Use of local building standards

A simple and pragmatic method of determining the building code correspondence factor, K_c, comprises the steps below.

a) Select a typical size of tubular circular member for a primary component of a topsides structure, e.g. 1 000 mm external diameter, 30 mm wall thickness and 20 m length.

b) Assume end fixity conditions for the element.

c) Determine (by trial and error or other means) a set of member forces for the element that give approximately 90 % utilizations for the element for each of the following situations:

   1) pure tension;

   2) pure bending;

   3) pure compression;

   4) combined bending and compression.

d) Calculate the utilization of the element for each situation following the requirements of ISO 19902 (U_{19902}).

e) Calculate the utilization of the element for each situation following the requirements of the selected national or regional building standard (U_{code}).

f) Calculate K_c as the most onerous of the ratios of utilizations for equivalent situations for ISO 19902 and the selected national or regional building standard:

$$K_c = \min\left(\frac{U_{code}}{U_{19902}}\right) \tag{A.5}$$

An example calculation is given in Annex B.

NOTE Performing the calculation of the ratio of utilizations for a range of member dimensions and varying end fixity conditions increases the confidence in the value of K_c.

A.8.2 Cylindrical tubular member design

No guidance is offered.

A.8.3 Design of non-cylindrical sections

A.8.3.1 General

Suitable codes for the design of non-cylindrical shapes for offshore topsides structures include ANSI/AISC 360-05[2], CSA-S16-09[3], BS EN 1993-1-1[6] and NS 3472[38].

A.8.3.2 Plate girder design

Suitable codes for the design of plate girders for offshore topsides structures include ANSI/AISC 360-05[2], CSA-S16-09[3], BS EN 1993-1-1[6] and API Bulletin 2V[42].

A.8.3.3 Box girder sections

Suitable codes for the design of fabricated box girders of the size and type normally associated with offshore topsides structures include ANSI/AISC 360-05[2], CSA-S16-09[3], BS EN 1993-1-1[6] and API Bulletin 2V[42].

A.8.3.4 Stiffened plate structures

Suitable codes for the design of stiffened plating for offshore topsides structures include CSA-S16-09[3], BS EN 1993-1-1[6], API Bulletin 2V[42] and Reference [41].

Care should however be taken with the design of stiffened compression flanges since many design codes are written for flanges in uni-axial compression. Generally, flanges with significant biaxial stress can be designed to API Bulletin 2V[42].

A.8.3.5 Stressed skin structures

Suitable codes for the design of shear strength of stiffened plates for offshore topsides structures include BS EN 1993-1-1[6] and Reference [42].

A.8.4 Connections

A.8.4.1 General

No guidance is offered.

A.8.4.2 Restraint and shrinkage

Advice on restraint and shrinkage is given in Reference [41].

A.8.4.3 Bolted connections

Guidance on bolted connections can be found in Reference [43].

A.8.5 Castings

No guidance is offered.

A.9 Structural systems

A.9.1 Topsides design

No guidance is offered.

A.9.2 Topsides structure design models

No guidance is offered.

A.9.3 Support structure interface

No guidance is offered.

A.9.4 Flare towers, booms, vents and similar structures

Wind-sensitive structures can be evaluated by the methods in, or those similar to, BS EN 1993-3-1[44] and Reference [45]. A 3 s gust should be adopted for the design of individual structural components and equipment secured to components on open decks.

Guidance on vortex-induced vibrations can be found in Reference [46].

For individual tubular structural components in a wind-sensitive structure, the following guidelines may be used to avoid the necessity for a rigorous analysis:

a) member length-to-diameter ratios should not exceed 40;

b) member diameter-to-thickness ratios should be less than 33, i.e.

$$D / \delta < 33 \qquad \text{(A.6)}$$

where δ is the member thickness;

c) stress concentration factors at the end connections of members should not be greater than 5.

A.9.5 Helicopter landing facilities (helidecks)

A.9.5.1 General

Reference [47] gives overall requirements for all aspects of helideck design, construction and equipment applicable to certain jurisdictions. In other cases, the requirements are usually addressed in class rules for floating or mobile structures. This part of ISO 19901 addresses only structural considerations for helidecks.

The helicopter size used for design and assessments of helidecks should be the largest and heaviest type anticipated for both normal and emergency operations.

Aircraft operating conditions that influence the design of the helideck structure include the size and location of the obstacle-free sector required and the extent of the falling gradient required to be kept clear of structure and equipment. Reference [47] or other governing documents contain the detailed requirements.

The design should accommodate actions resulting from emergency landings as well as from normal landings, from personnel on the helideck, freight, refuelling equipment and other traffic, snow and ice, and rotor downwash, etc. The design should allow helicopters to be parked (for short periods) or stowed (for longer periods) anywhere on the helideck.

The selection of the platform layout should consider the effects of wind turbulence from items near the helideck, such as accommodation blocks, turbine exhausts, cranes and equipment. Thermal effects from hot and cold gases emitted by plant on the platform should also be considered. Design methods to model these effects can include

— wind tunnel testing (using small-scale physical models), or

— CFD.

These or other suitable methods should be used to determine any limitations of wind speed and direction on helicopter operational mass, or directions of take-off and landing, and any changes in the platform layout to mitigate against these limitations.

Maximum take-off masses, dimensions and other properties can be obtained from the helicopter manufacturer or operator. Data for some common types of helicopter in offshore use worldwide are given in Table A.5.

Table A.5 — Indicative helicopter masses and dimensions

Type	Dimensions		
	Maximum mass kg	Rotor diameter m	*D* value[a] m
Aerospatiale Allouette III			
Augusta A109	2 600	11,00	13,05
Bolkow Bo 105D	2 300	9,90	11,81
Bolkow 117	2 300	11,00	13,00
Bell 212	5 080	14,63	17,46
Bell 214ST	7 936	15,85	18,95
Boeing BV 234LR Chinook	21 315	18,29	30,18
Dauphin SA 365N2	4 250	11,93	13,68
EC225 Super Puma Mk II	11 000	16,20	19,50
EH101	14 290	18,60	22,80
Sikorsky S61N	9 298	18,90	22,20
Sikorsky S76B&C	5 307	13,40	16,00
Sikorsky S92	11 862	17,17	20.88
Super Puma AS 332L	8 599	15,00	18,70
Super Puma AS 332L2	9 250	16,20	19,50

[a] Maximum length from front of main/front rotor to back of rear/tail rotor.

A.9.5.2 Construction

No guidance is offered.

A.9.5.3 Design actions and resistances

Reference [48] contains an additional heavy landing situation intermediate between the emergency landing and helicopter-at-rest cases required by Reference [47]. This is unlikely to affect the structural design.

Failure of one engine of a helicopter in the air is the normally considered operating case amongst likely survivable emergencies that usually generates the highest rate of descent of a helicopter and, therefore, the governing dynamic actions on the helideck. This is the case considered in Reference [47] and in state aviation standards requirements such as that listed under Reference [48]. The dynamic actions on the helideck are calculated by considering the specified maximum impact action (normally 2,5 times the MTOW), suitably distributed over the deck.

A single-main-rotor helicopter is normally assumed to land simultaneously on its two main undercarriages or skids. A tandem-main-rotor helicopter can be assumed to land on the wheels of all main undercarriages simultaneously. Local actions should be used in locations derived from a consideration of the geometry of the landing gear.

For single-main-rotor helicopters, the total actions imposed on the structure are normally taken as concentrated actions at the undercarriage centres of the specified helicopter and divided equally between the two main undercarriages. For tandem-main-rotor helicopters, these actions are normally taken as concentrated actions at the undercarriage centres of the specified helicopter and distributed between the main undercarriages in the proportion in which they carry the maximum static action. These concentrated undercarriage actions are normally treated as point loads; alternatively, a tyre contact area can be assumed in accordance with the manufacturer's specification. The maximum take-off mass and undercarriage centres of the helicopter for which the helideck has been designed should be documented.

A similar distribution is taken for actions for emergency helicopter landing situations.

Web buckling should be checked from first principles. Closed-form solutions can be used to check for web crippling.

Stiffening components of the deck or supporting structure can be designed using plastic hinge theory. Cantilever components can be designed using a first-yield solution.

A.9.5.4 Reassessment of existing helidecks

No guidance is offered.

A.9.6 Crane support structure

When designing crane support pedestals, there are two common types of connection interface:

— where the crane incorporates a slewing ring;

— where the crane revolves around a king post, the base of which is bolted to the pedestal flange.

In both cases, the pedestal flange should be machined to a flatness and surface finish compatible with the type of slewing ring or king post base being used. Suitable tolerances should be obtained from the crane manufacturer along with the values of stiffness necessary to support the crane slewing ring or king post flange. The information given by the manufacturer should enable a suitable diameter of pedestal and thickness of top flange to be calculated and thus an outer diameter for the pedestal tube, determined after making clearance allowances for the mounting bolt tensioner or torque spanner to be used.

Any crane pedestal which has an unsupported height above the top deck attachment area of more than 10 times the outside diameter of the tube should be checked for its dynamic characteristics and, where necessary, these should be included in the fatigue life calculations for the pedestal structure.

The pedestal top flange should be hot-rolled and drop-forged, then machined to profile. The shaped flange should be attached to the pedestal wall by means of a full-penetration butt weld. The flange should be perpendicular to, and its axis concentric with, the pedestal axis. The flange material should be compatible with that of the pedestal and be provided with documentation from the flange supplier.

After fabrication of the pedestal, the top flange surface should be machine-skimmed to the tolerances and values specified by the crane supplier. Clearance holes for bolts should be drilled from a template supplied by the crane manufacturer. No further welding should be carried out around the flange area after machining, as any heavy welding work can jeopardize the integrity and flatness of the machined surface. When machining the top flanges of pedestals, the machined face of the flange should be kept as near as possible at 90° to the central axis of the pedestal tube. Values of angular tolerance should be determined for the diameter of pedestal being used and for the type of machine being used for the skimming process.

The minimum pedestal dimensions are dependent on the actions imparted by the crane.

The following three conditions should be considered in order to ensure pedestal integrity.

a) **Steady-state or static condition**

This consists of the overturning moment and direct action derived from static values only, with no dynamic factors or wind actions being taken into consideration. This condition should be used to calculate the fatigue life of the overhang of the pedestal flange. FEA can be used to calculate stress concentration levels at any changes of section of the flange profile and in the flange/weld interface region. Full rotation of 360° around the pedestal flange should be considered, with a complete reversal of action occurring on upper and lower faces of the flange during each rotation. The crane manufacturer should provide the spectrum factor for actions. This is generally dependent on the specification for the design of the crane. The number of expected crane rotations should be determined from the expected operational frequency and intended life of the crane. A minimum fatigue life of one million cycles under steady-state conditions should be considered, with an appropriate action spectrum factor.

b) **Dynamic condition**

This consists of determining static values of the overturning moment and axial action from the crane hook, multiplied by a dynamic factor which is determined from the crane stiffness, prevailing sea state, lifting speed and gravitational constant (see ISO 19901-6). This condition should include the effect of wind loading, acting in the most unfavourable orientation on the crane structure during crane operations. The effects of any mitigation device fitted to the crane should not be included as this could fail in service and thus have no effect on the dynamic actions imparted to the pedestal.

c) **Survival condition**

Two cases should be considered.

1) Crane failure collapse

In this case, the major structural components of the crane fail from gross actions applied, with none of the overload mitigation devices being activated. The crane design should be such that the last component to fail should be the lower machinery house with its attached bearing or kingpost. The pedestal should not fail or exhibit any local sidewall or flange damage or deformation under this extreme condition. Pedestal designs should allow for out-of-alignment deformations of the pedestal axis. Any permanent set that takes place should not inhibit normal crane operations, such as rotation.

2) Storm loading

The crane and consequently the pedestal are subject to extreme weather conditions, i.e. wind loading, such as that associated with hurricanes, typhoons or cyclones. In these environmental conditions, the crane will usually be shut down with the boom secured in the rest.

This part of ISO 19901 does not cover cranes that incorporate a pintle or house/hook rollers support structure: special design and loading conditions apply to these types of crane mountings.

Cranes that incorporate heavy counterweights can produce the maximum static pedestal moment in the unloaded condition. In such cases, the actions should be calculated only by the crane manufacturer or supplier.

Design guidance is given in BS EN 1993-1-1[6], BS EN 13852-1[49], BS EN 1993-1-9[51] and Reference [50] and data should be found from crane manufacturers' data sheets.

A.9.7 Derrick design

See API Spec 4F[39] and Reference [52].

A.9.8 Bridges

No guidance is offered.

A.9.9 Bridge bearings

No guidance is offered.

A.9.10 Anti-vibration mountings for modules and major equipment skids

No guidance is offered.

A.9.11 System interface assumptions

No guidance is offered.

A.9.12 Fire protection systems

Types of PFP commonly in use offshore include those applied directly to the structure to be protected and those attached as prefabricated composite fire walls, or combined fire-and-explosion protection panels, to the topsides structure. Inspection of structural components covered by PFP for signs of deterioration can have many of the same difficulties as for coated and painted structures.

A.9.13 Penetrations

No guidance is offered.

A.9.14 Difficult-to-inspect areas

Certain areas of the topsides can be difficult to inspect in service because of their function and location (e.g. flares, drilling derricks and areas hidden by plant and equipment or PFP). Construction details that are difficult to inspect and maintain in service should be avoided as far as possible. Where this is not possible, materials and configurations requiring little or no inspection and maintenance should be followed. Higher fatigue damage design factors are required in such areas (see 6.7).

The need for coating areas that cannot be inspected can be avoided by airtight welded sealing of such areas.

A.9.15 Drainage

Ponding can occur due to layout, deformation of plating and other causes. Where oil-fouled waste can be anticipated, the design should allow for contaminated water to be cleaned before discharge into the sea. Reference should be made to appropriate environmental standards.

A.9.16 Actions due to drilling operations

Jarring of the drilling derrick and equipment from drilling operations is a foreseeable cause of shock actions that should be considered during design.

A.9.17 Strength reduction due to heat

No guidance is offered.

A.9.18 Walkways, laydown areas and equipment maintenance

No guidance is offered.

A.9.19 Muster areas and lifeboat stations

No guidance is offered.

A.10 Materials

A.10.1 General

No guidance is offered.

A.10.2 Carbon steel

The range of applications for which steel is selected for use in topsides structures is considerable. Steel interfacing with process plant and pipework can be subjected to very low temperatures during certain operations, in particular system blow-down. Conversely, steel exposed to radiation from flaring during blow-down can be subjected to high temperatures.

Steel structures supporting risers can be subject to substantial impact and fatigue from slugging in the pipework and high levels of uncertainty with regard to the interaction with pipe stresses in lines, which can be orders of magnitude stiffer than the structure.

The above issues should be carefully considered in relation to material selection.

Selection of steel quality and requirements for inspection of welds should be based on a systematic classification of welded joints according to the structural significance and complexity of connections. The main criterion for decision is the significance, with respect to consequences of failure, of the connection. In addition, the stress predictability can influence the selection.

A.10.3 Stainless steel

Guidance on the structural use of stainless steel is given in Reference [53].

A.10.4 Aluminium alloys

Guidance on the structural use of aluminium is given in BS EN 1999-1-1[54].

A.10.5 Fibre-reinforced composites

Composites are strong, generally insulating, durable and corrosion-resistant and can be manufactured to a consistent and reliable standard providing the right materials and processes are used. Although the fibres in carbon-reinforced composites can be electrically conductive, any design using such fibres should not rely on either the conductivity of the fibres or the insulation of the composite material.

The majority of composites have lower stiffnesses than metals and this should be taken into account, particularly where composites and metals interface. High and ultra-high stiffness fibres are available and, with careful selection of the composite specification, the stiffness of steels can be matched. This is particularly useful for reinforcing steelwork (see Reference[55]).

Composites are usually brittle materials, and this can affect their robustness and ability to redistribute forces in a controlled manner. Long-term creep can occur and moisture absorption can influence the ultimate tensile strength.

The design of joints and connections requires detailed knowledge of the properties of the composite material.

Carbon fibre composites can theoretically cause galvanic corrosion problems when in direct contact with steel structures, and layers of non-conductive fibres have been used to ensure insulation between carbon fibres and adjacent steel structure.

A.10.6 Timber

Suitable standards for timber design include BS EN 1995-1-1[56] and BS EN 1995-1-2[57].

A.11 Fabrication, quality control, quality assurance and documentation

A.11.1 Assembly

The fabrication of topsides structures requires different skills, facilities and experience from those needed for their support structures. Fabricators selected for topsides structures should have appropriate experience and skills for the size and complexity of the topsides to be fabricated. Major topsides structures have a high level of multi-discipline interfaces and the early involvement of a fabricator in the planning and design process can yield significant advantages leading to the successful outcome of a project. Key issues to be considered include:

a) planning for timely delivery and installation of major items of equipment and mitigation of the effects of potential late delivery;

b) engineering joints and connections to suit the most efficient construction method;

c) scheduling to allow for the impact of design information that depends on the procurement cycle for equipment;

d) designing, fabricating and commissioning onshore the topsides to minimize the requirement for work offshore;

e) scheduling the application of paint and PFP coatings to minimize the impact on equipment installation and commissioning;

f) allowing for the potential interference of equipment and pipework with the temporary steel and equipment for moving and transporting the topsides, both during construction and on completion;

g) allowing for the reversal of normal load paths during loadout, transport and installation and for their impact on walls, piping and equipment.

ISO 20340[58] gives performance requirements for offshore painting systems.

A.11.2 Welding

Weld volumes have a significant impact on the cost of topsides structures and the heat input for oversized welds can increase distortion. Topsides structures can have a large number of small structural components that are sized for convenience or detail rather than stress. Careful consideration of the minimum acceptable size for welds can result in significant cost savings with no loss of safety or serviceability.

A.11.3 Fabrication inspection

The requirements of ISO 19902 do not give criteria for all the situations that can occur in a topsides structure. In particular, classifications of components in ISO 19902 do not include items of equipment support that can be critical to safety because of the potential for consequent fire or explosion. Designers of topsides structures should ensure that the inspection requirements specified are appropriate to component criticality.

A.11.4 Quality control, quality assurance and documentation

Drawings and specifications for topsides structures can cover a wider range and considerably higher levels of detail than those for support structures. The drawings define interfaces with details provided by other engineering disciplines. It is important that the responsibility for engineering interfaces be clearly defined and recorded to avoid the risk of details prepared by one discipline undermining the integrity of the engineering in another.

Issues that should be considered include

a) welded attachments affecting stress concentrations in primary structure,

b) penetrations in plates affecting structural assumptions of available support or load path,

c) penetrations in beam webs undermining bearing or shear strength,

d) the interface with major pipework affecting the load path and stress level in structure or pipes,

e) the temporary removal of key components to assist with installation of equipment, and

f) the use of minimum default weld sizes on drawings or in specifications (in the event that a large weld is not correctly defined, the undetected use of the default weld by a fabricator can result in connection failure).

A.11.5 Corrosion protection

No guidance is offered.

A.12 Corrosion control

Guidance for coatings is given in ISO 8501-1[59], ISO 8503[60], ISO 12944-5[61] and Norsok M-CR-501[62].

A.13 Loadout, transportation and installation

Many of the difficulties encountered in the movement and installation of topsides structures result from poor planning, poor communication and late decisions. They also arise from a lack of attention to detail. The following lists identify good practice and areas of detail that should be considered:

a) good practice:

  1) early involvement of all key contractors in planning and preliminary engineering;

  2) identification, recording and updating of all technical interfaces and those responsible for them;

  3) early agreement on methods and equipment to be used;

  4) early integration of the space required for all temporary equipment, topsides structure and equipment support structures in the design model;

b) common problems:

  1) spatial conflict between the topsides equipment and the equipment used for loadout and lift, including:

    i) under-deck platforms and piping clashing with loadout trailers;

    ii) external sea fastenings clashing with access platforms and walkways;

    iii) roof-mounted equipment clashing with lifting slings and sling lay-down areas;

  2) issues of detail, including:

    i) loose or poorly sea-fastened equipment or materials in modules causing damage (this can result from inadequate knowledge of the forces likely to be encountered);

    ii) eccentricities between temporary structural components not being considered and consequently failing;

    iii) the effect of incomplete work not being communicated or clearly understood;

    iv) inadequate consideration of load path reversals involved in transient phases.

An example of a possible consequence of load reversal is the buckling of non-structural walls that experience compressive stresses when trailer loadout is used.

Light structures can be lifted by a crane vessel or by the larger onshore cranes directly onto a transport barge or onto the deck of the crane vessel. In practice, use of crane vessels is often constrained by their draught.

A.14 In-service inspection and structural integrity management

A.14.1 General

No guidance is offered.

A.14.2 Particular considerations applying to topsides structures

A.14.2.1 Corrosion protection systems

No guidance is offered.

A.14.2.2 Access routes, floors and gratings

Good practice is to define main and secondary escape routes and to subject these areas to detailed inspection. This is normally done in close cooperation with the safety discipline. Main and secondary escape routes are often defined on specific drawings. These areas should be closely inspected to avoid any impediment to evacuation of the structure.

A.14.2.3 Supports for safety-critical equipment, including communications, electrical and fire-water systems

No guidance is offered.

A.14.2.4 Control of hot work (e.g. welding and cutting)

No guidance is offered.

A.14.2.5 Accidental actions

The structural integrity management plan for the topsides structure should consider emergency arrangements following an accidental event. These should include arrangements for the inspection of the damage, to assess and evaluate any effects on the integrity of the topsides structure, to recommend any necessary emergency evacuation, and monitoring or repairs.

A.14.2.6 Change control

No guidance is offered.

A.14.3 Topsides structure default inspection scopes

A.14.3.1 General

No guidance is offered

A.14.3.2 Baseline inspection

A walk-down is a systematic on-site inspection of the topsides structure and equipment that can complement the baseline structural inspection if not undertaken before installation (see A.6.9).

A.14.3.3 Periodic inspection

Where an owner decides not to develop a topsides structure-specific structural integrity management system, inspections should be performed at the frequencies stated in ISO 19902.

A.14.3.4 Special inspections

Special inspections are conducted to monitor repairs and other remedial work, any growth in the extent of known damage and defects, and any known or suspected areas of vulnerability, for example underdesign identified by later assessment. Special inspections can also be needed for topsides structure reuse (see Clause 16). Key features of special inspections include definition of the goals and objectives, selection of appropriate tools and techniques, scopes of work, and inspection intervals.

A.14.3.5 Unscheduled inspections

No guidance is offered.

A.15 Assessment of existing topsides structures

No guidance is offered.

A.16 Reuse of topsides structure

The following describes the minimum recommended inspection extent for a topsides, but this should be modified in the light of the structural assessment for the reuse condition and the previous in-service inspection history.

Ultrasonic testing (UT) or magnetic particle inspection (MPI) inspection should be carried out for:

— 10 % of each truss-bracing structural component;

— 10 % of each truss chord structural component;

— 10 % of each plate girder structural component;

— 25 % of each connection to a deck leg;

— 100 % of crane pedestal connections;

— 100 % of cantilever deck connections;

— 100 % of survival/safety equipment connections.

Unless the functional requirements of reuse are identical to those of the original design, engineering for the reuse of a topsides will be a multi-discipline exercise to assess the practicability of re-configuring the equipment within the space and strength of the existing topsides structures.

Unless the records of the existing design and any modifications are of a high standard, the re-engineering and modification of existing topsides can be more difficult and potentially more expensive than new construction. Extensive survey work can be necessary to identify the nature and condition of existing topsides structures. It can be necessary to use advanced forms of structural analysis and detailed reliability analysis to prove adequate strength, durability and safety for a reused topsides structure.

Annex B
(informative)

Example calculation of building code correspondence factor

B.1 General

This annex contains an example of the derivation of the building code correspondence factor for a commonly used code. Care should be taken to ensure that up-to-date versions of the standards are used for both the derivation of the correspondence factor and the member and joint checks.

B.2 Basic data

The data in Table B.1 are used for the example presented in this annex.

Table B.1 — Basic data

Data		Symbol	Value
Assumptions (circular tube)	Outside diameter	D	500 mm
	Thickness	δ	20 mm
	Length	L	15 m
	Yield strength	f_y	355 N/mm^2
	Effective length factor	K	1,0
	Young's modulus	E	205 000 N/mm^2
Derived properties (circular tube)	Inner diameter	d	460 mm
	D/δ	—	25,0
	Cross-sectional area	A	30 159 mm^2
	2nd moment of area	I	870 × 10^6 mm^4
	Elastic section modulus	Z_e	3,48 × 10^6 mm^3
	Radius of gyration	r	169,8 mm
Tension case	Factored axial tension	S_T	9 500 kN
Compression case	Factored axial compression	S_C	5 000 kN
Bending case	Factored bending moment	S_M	1 400 kNm
Combined case	Factored axial compression	$S_{C,bc}$	2 500 kN
	Factored bending moment	$S_{M,bc}$	700 kNm
	Bending amplification reduction factor	$C_m = C_{m,y} = C_{m,z}$	0,6 (uniform bending)

B.3 Design and utilizations to ISO 19902

B.3.1 Tension case

Table B.2 — Design and utilizations to ISO 19902:2007 Tension case

Parameter	Symbol	Method of calculation	Value
Axial tensile stress	σ_t	$\frac{S_T}{A}$	315 N/mm^2
Partial resistance factor for axial tensile strength	$\gamma_{R,t}$	From ISO 19902	1,05
Representative axial tensile strength	f_t	f_y	355 N/mm^2
Utilization	$U_{m,t}$	$\frac{\sigma_t}{f_t / \gamma_{R,t}}$	0,932

B.3.2 Compression case

Table B.3 — Design and utilizations to ISO 19902:2007 Compression case

Parameter	Symbol	Method of calculation	Value
Elastic critical buckling coefficient	C_x	From ISO 19902	0,30
Representative elastic local buckling strength	f_{xe}	$\frac{2 \times C_x \times E \times \delta}{D}$	4 920 N/mm^2
Ratio	f_y / f_{xe}	—	0,072
Representative local buckling strength	f_{yc}	f_y	355 N/mm^2
Column slenderness parameter	λ	$\frac{K \times L}{\pi \times r}\sqrt{\frac{f_{yc}}{E}}$	1,170
Representative axial compressive strength	f_c	$\left(1 - 0,278\lambda^2\right) f_{yc}$	220 N/mm^2
Partial resistance factor for axial compressive strength	$\gamma_{R,c}$	From ISO 19902	1,18
Axial compressive stress	σ_c	$\frac{S_C}{A}$	165,8 N/mm^2
Utilization	$U_{m,c}$	$\frac{\sigma_c}{f_c / \gamma_{R,c}}$	0,890

B.3.3 Bending case

Table B.4 — Design and utilizations to ISO 19902:2007 Bending case

Parameter	Symbol	Method of calculation	Value
Ratio	—	$\frac{f_y \times D}{E \times \delta}$	0,043 3
Hence use ISO 19902:2007, Formula (13.2–13)			
Plastic section modulus	Z_p	$\frac{1}{6}\left[D^3 - (D-2\delta)^3\right]$	$4{,}611 \times 10^6$ mm^3
Representative bending strength	f_b	$\left(\frac{Z_p}{Z_e}\right) f_y$	470 N/mm^2
Elastic yield moment	M_y	$Z_e \times f_y$	1 235 kNm
Bending stress	σ_b	$\frac{S_M}{Z_e}$	402 N/mm^2
Partial resistance factor for bending strength	$\gamma_{R,b}$	From ISO 19902	1,05
Utilization	$U_{m,b}$	$\frac{\sigma_b}{f_b / \gamma_{R,b}}$	0,898

B.3.4 Combined compression and bending

Table B.5 — Design and utilizations to ISO 19902:2007 Combined compression and bending

Parameter	Symbol	Method of calculation	Value
Euler buckling strength	$f_{e,y} = f_{e,z}$	$\frac{\pi^2 \times E}{(K \times L/r)^2}$	259,3 N/mm^2
Axial compressive stress	σ_c	$\frac{S_{C,bc}}{A}$	82,9 N/mm^2
Bending stress	$\sigma_{b,y}$ and $\sigma_{b,z}$	$\frac{S_{M,bc}}{Z_e}$	$\sigma_{b,y}$ = 201,0 N/mm^2 $\sigma_{b,z}$ = 0
Utilization 1	$U_{m,bc1}$	$\frac{\gamma_{R,c} \times \sigma_c}{f_c} + \frac{\gamma_{R,b}}{f_b}\left[\left(\frac{C_{m,y} \times \sigma_{b,y}}{1-\sigma_c/f_{e,y}}\right)^2 + \left(\frac{C_{m,z} \times \sigma_{b,z}}{1-\sigma_c/f_{e,z}}\right)^2\right]^{0,5}$	0,841
Utilization 2	$U_{m,bc2}$	$\frac{\gamma_{R,c} \times \sigma_c}{f_{yc}} + \frac{\gamma_{R,b}\sqrt{\sigma_{b,y}^2 + \sigma_{b,z}^2}}{f_b}$	0,725
Maximum utilization	$U_{m,bc}$	max ($U_{m,bc1}$, $U_{m,bc2}$)	0,841

B.4 Design and utilizations to ANSI/AISC 360-05[2]

B.4.1 Tension case

Table B.6 — Design and utilizations to ANSI/AISC 360–05[2] — Tension case

Parameter[a]	Symbol[a]	Method of calculation	Value
Axial tensile strength	P_n	$f_y \times A$	10 706 kN
Resistance factor for tension	ϕ_t	From ANSI/AISC 360–05	0,90
Design tensile strength	—	$\phi_t \times P_n$	9 636 kN
Utilization	$U_{m,t}$	$\dfrac{S_T}{\phi_t \times P_n}$	0,986
[a] ISO 19902 terminology and symbols are used in this table, where possible; P_n and ϕ_t are from ANSI/AISC 360–05.			

B.4.2 Compression case

Table B.7 — Design and utilizations to ANSI/AISC 360–05[2] — Compression case

Parameter[a]	Symbol[a]	Method of calculation	Value
Euler buckling strength	f_e	$\dfrac{\pi^2 \times E}{(K \times L/r)^2}$	259,3 N/mm^2
Slenderness ratio	—	$\dfrac{K \times L}{r}$	88,3
Ratio (from ANSI/AISC 360–05)	—	$4{,}71\sqrt{\dfrac{E}{f_y}}$	113,2
$\dfrac{K \times L}{r}$ u $4{,}71\sqrt{\dfrac{E}{f_y}}$ so critical stress [from ANSI/AISC 360–05, Formula (E3–2)]	F_{cr}	$0{,}658^{\frac{f_y}{f_e}} \times f_y$	200,1 N/mm^2
Compressive strength	P_n	$F_{cr} \times A$	6 038 kN
Resistance factor for compression	ϕ_c	From ANSI/AISC 360–05	0,90
Design compressive strength	—	$\phi_c \times P_n$	5 434 kN
Utilization	$U_{m,c}$	$\dfrac{S_c}{\phi_c \times P_n}$	0,920
[a] ISO 19902 terminology and symbols are used in this table, where possible; F_{cr}, P_n and ϕ_c are from ANSI/AISC 360–05.			

B.4.3 Bending case

Table B.8 — Design and utilizations to ANSI/AISC 360–05[2] — Bending case

Parameter[a]	Symbol[a]	Method of calculation	Value
Ratio (from ANSI/AISC 360–05)	—	$\frac{0{,}45 \times E}{f_y}$	259,9
Hence ANSI/AISC 360–05, Clause F.8, is applicable			
Plastic section modulus	Z	$\frac{1}{6}\left[D^3 - (D-2\delta)^3\right]$	$4{,}611 \times 10^6$ mm^3
Nominal flexural strength for yielding	$M_n = M_p$	$f_y \times Z$	1 637 kNm
Check for compactness	D/δ	—	25
Limit for compact section (from ANSI/AISC 360–05, Table B4–1)	—	$0{,}07 \times \frac{E}{f_y}$	40,4
Section qualifies as compact, so local buckling inapplicable			
Resistance factor for flexure	ϕ_b	From ANSI/AISC 360–05	0,90
Design bending strength	—	$\phi_b \times M_n$	1 473 kNm
Utilization	$U_{m,b}$	$\frac{S_M}{\phi_b \times M_n}$	0,950

a ISO 19902 terminology and symbols are used in this table, where possible; M_n and ϕ_b are from ANSI/AISC 360–05.

B.4.5 Combined compression and bending

Table B.9 — Design and utilizations to ANSI/AISC 360–05[2] — Combined compression and bending

Parameter[a]	Symbol[a]	Method of calculation	Value
Required axial compressive strength	P_r	$S_{C,bc}$	2 500 kN
Available axial compressive strength	P_c	$\phi_c \times P_n$	5 434 kN
Required flexural strength	M_{rx} and M_{ry}	$S_{M,bc}$	M_{rx} = 700 kNm M_{ry} = 0
Available flexural strength	M_{cx} and M_{cy}	$\phi_b \times M_n$	1 473 kNm
Ratio	—	$\frac{P_r}{P_c} = \frac{\text{Required axial compressive strength}}{\text{Available axial compressive strength}}$	0,460
Hence use ANSI/AISC 360–05, Equation (H1–1a)			
Utilization	$U_{m,bc}$	$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right)$	0,883

a ISO 19902 terminology and symbols are used in this table, where possible; P_c, P_n, P_r, M_n, M_{cx}, M_{cy}, M_{rx}, M_{ry}, ϕ_b and ϕ_c are from ANSI/AISC 360–05.

B.4.6 Derivation of building code correspondence factor

Table B.10 — Derivation of building code correspondence factor

Check	**Utilization from** ISO 19902 U_{19902}	**Utilization from ANSI/AISC 360–05** U_{360-05}	$\frac{U_{360-05}}{U_{19902}}$
Tension	0,932	0,986	1,058
Compression	0,890	0,920	1,034
Bending	0,898	0,950	1,058
Compression and bending	0,841	0,883	1,050
		Minimum ratio	1,034

Hence the building code correspondence factor, K_c, for ANSI/AISC 360-05 is 1,034.

The above correspondence factor, while based on cylindrical tubular sections, is applicable to non-cylindrical sections (including design checks not explicitly covered, e.g. web shear checks). As can be seen from the range of the utilization ratios between the two standards, there is scope for more sophisticated analysis for different cases. Any results should be shared with other users of the same national or regional building code.

Annex C
(informative)

Regional information

The values in Table C.1 may be used for the building code correspondence factor, K_c, for certain national or regional building standards.

Table C.1 — Values of building code correspondence factor, K_c

Building standard	Building code correspondence factor K_c
ANSI/AISC 360–05	1,034
CSA S16–09	1,058

The correspondence factors given in Table C.1 are dependent upon the input parameters in Table B.1 and therefore the designer should only use such a value of correspondence factor if it is representative of the specific design situation being considered.

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