International Standard

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Third edition
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Oil and gas industries including lower carbon energy — Specific requirements for offshore structures —

Part 3: Topsides structure

Industries du pétrole et du gaz, y compris les énergies à faible teneur en carbone — Exigences spécifiques relatives aux structures en mer —

Partie 3: Structures Top Sides

Reference number
ISO 19901-3:2024(en)

Contents

Foreword

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The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular, the different approval criteria needed for the different types of ISO document should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

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This document was prepared by Technical Committee ISO/TC 67, *Oil and gas industries including lower carbon energy*, Subcommittee SC 7, *Offshore structures*, in collaboration with the European Committee for Standardization (CEN) Technical Committee CEN/TC 12, *Oil and gas industries including lower carbon energy*, in accordance with the Agreement on technical cooperation between ISO and CEN (Vienna Agreement).

This third edition cancels and replaces the second edition (ISO 19901-3:2014), which has been technically revised.

The main changes are as follows:

— alignment of terminology with that of ISO 19900;

— a rational re-arrangement of the clauses content and numbering;

— adoption with modifications of IOGP supplementary requirements (S-631-04);

— ‘national or regional codes’ and ‘national or regional building codes’ have been replaced by ‘national building standards’ throughout the whole document;

— ‘supporting structure’ has been replaced by ‘substructure’ and definition of ‘substructure’ has been added to Clause 3;

— ‘wave, wind and current’ has been replaced by ‘metocean’;

— ‘design assessment/situations’ has replaced ‘design situations’ according to ISO 19900;

— 5.2.1 has been updated distinguishing between ASD (Allowable strength design) associated to ANSI/AISC 360-22 and WSD (Working stress design) associated to AISC 335-89 and API RP 2A-WSD. Further guidance is provided for floating structures where the hull is typically designed using the WSD method. In 5.2.2 guidance on the application of K_c is given in case of WSD method.

— subclause 5.7 on critical structures has been added;

— in 6.5.2.4 the frequency range to avoid structural resonance has been changed according to NORSOK N-004:2022, F-2-9-6;

— Table 2 has been updated with the introduction of ‘restricted access for inspection, maintenance and repair’ partial damage factors and reduction in case of full accessibility (with reference to ISO 19904-1, NORSOK N-004,[32] Reference [30] and DNV-OS-C101[31]). Guidance in case of dissimilar materials has been added;

— subclause 6.8.2 on ductility has been introduced, adapted from NORSOK N-004:2022, 7.2;

— addition of Table A.1 with typical minimum values for local, primary and global design of operational actions (Q);

— subclause 7.3 has been re-ordered and updated;

— subclause 7.5 has been renamed ‘Indirect actions and resulting forces’ and updated according to the modifications and assumptions in 10.1 and 10.2;

— wind actions, 7.6.2 and A.7.6.2, introduction of national building standards for the evaluation of the representative wind actions; alignment with ISO 19900 and ISO 19901-1 and addition of more guidance;

— alignment of minimum lateral acceleration for seismic (7.7.2 and A.7.7.2) with ANSI/API RP 2TOP[82].

— all sources of topsides accelerations collected (7.9.9 and A.7.9.9) and aligned;

— technical review of the accidental events (7.9 and A.7.9), with introduction of risk-informed and reliability-based approaches for fire and explosion in addition to the default semi-probabilistic approach;

— K_c correspondence factor (8.1 and A.8.1) defined according to an equivalent reliability procedure for ANSI/AISC 360-22,[12] CSA-S16:19[14] and EN 1993-1-1[13];

— bolted connection (8.4.3 and A.8.4.3) have been modified according to IOGP supplementary specification S-631-04;

— 8.5 has been renamed as ‘Castings and forgings’, adding references to forgings;

— addition of 8.6 and A.8.6 on design for structural stability in alignment with ANSI/API RP 2TOP[82] and based on ANSI/AISC 360-22[12] and EN 1993-1-1[13] criteria;

— addition of Clause 9 dedicated to the description of the limit state verification approaches including risk-informed and reliability-based approaches for fire and explosion (9.2, 9.3, A.9.2 and A.9.3) in addition to the default semi-probabilistic approach;

— in 10.2.1, an alternative method (method b) for the analysis of the topsides structures has been introduced with further guidance in A.10.2.1. The associated 6.4, 7.5, 7.8 and 10.1 and A.6.4, A.7.5, A.7.8 and A.10.1 have been updated accordingly;

— helicopter landing facilities (10.5) updated according to CAP 437[21] for emergency landing and addition of design load combinations (Table 7) adapted from NORSOK N-004:2022, Table F.5.[32] Deletion of the previous Table A.5;

— crane support structure clauses, 10.6 and A.10.6 have been reviewed. Crane support structure is to be designed according to API Spec 2C or EN 13852-1 and additional provisions reported. The simplified fatigue method has been aligned with ANSI/API RP 2TOP[82];

— Table 9 adapted with modifications from NORSOK N-004:2022, Table F.1[32] and addition of some example figures for DC;

— former 12.1 to 12.3.5 have been deleted and moved to ISO 19902:2020, Clause 18.

— in 12.2 Welding requirements have been reviewed;

— in 12.5 provisions for dissimilar materials have been added, adapted from NORSOK N-004:2022, F.4.4;

- the previous Clause 12 and A.12 “Corrosion control” in ISO 19901-3:2014 has been removed because it is now included in ISO 19902:2020; in Clause 14, reference to ISO 19901-9 has been added and the previous Clause 14 in ISO 19901-3:2104 on “Topsides structure default inspection scope” has been removed, being now covered by ISO 19901-9; in Clause 14 and A.14, the subclauses 14.2.2 and A.14.2.2 on “Critical structures” have been added;

- in Annex B, updated example of K_c calculations by utilization ratio for ISO 19902 and ANSI/AISC 360-22[12].

- removal of former Annex C. K_c is now reported as normative value in Table 4 for ANSI/AISC 360-22[12], CSA-S16:19[14] and EN 1993-1-1[13].

A list of all parts in the ISO 19901 series can be found on the ISO website.

Any feedback or questions on this document should be directed to the user’s national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

The International Standards on offshore structures prepared by TC 67 (i.e. ISO 19900, the ISO 19901 series, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906) constitute a common basis covering those aspects that address design requirements and assessments of all offshore structures used by the petroleum and natural gas industries including lower carbon energy 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 International Standards on offshore structures prepared by TC 67 are 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 document has been prepared for those structural components of offshore platforms which are above the wave zone and are not part of the substructure or of the hull.

Historically, the design of structural components in topsides has been performed to national building standards for onshore structures, modified in accordance with experience within the offshore industry, or to relevant parts of classification society rules. While this document permits use of national building standards, 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 substructures 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 document.

Annex B provides an example of the use of national building standards for onshore structures in conjunction with this document.

Oil and gas industries including lower carbon energy — Specific requirements for offshore structures —

Part 3: Topsides structure

1 Scope

This document provides requirements, guidance and information for the design and fabrication of topsides structure for offshore structures, including in-service, pre-service and post-service conditions.

The actions on topsides structure and the action effects in structural components are derived from this document, where necessary in combination with other International Standards in the ISO 19901 series (e.g. ISO 19901-1 for wind actions - see 7.6.2, ISO 19901-2 for seismic actions - see 7.7) and ISO 19902 for fatigue design (see 6.7).

This document is applicable to the following:

— topsides of fixed offshore structures;

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

— topsides of arctic offshore structures, excluding winterization (see ISO 19906).

If any part of the topsides structure forms part of the primary structure of the overall structural system which resists global platform actions, the requirements of this document are supplemented with applicable requirements in ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906.

For those parts of floating offshore structures and mobile offshore units that are chosen to be governed by the rules of a recognized classification society, the corresponding class rules supersede the associated requirements of this document.

This document also addresses prevention, control and assessment of fire, explosions and other accidental events.

The fire and explosion provisions of this document can be applied to those parts of the hulls of floating structures and mobile offshore units that contain hydrocarbon processing, piping or storage.

NOTE Requirements for structural integrity management are presented in ISO 19901-9.

This document 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.

This document provides requirements for selecting and using a national building standard with a correspondence factor for determining the resistance of rolled and welded non-circular prismatic components and their connections.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

API Spec 2C, *Offshore Pedestal-mounted Cranes*

API Spec 2SC, *Manufacture of Structural Steel Castings for Primary Offshore Applications*

API Spec 2SF, *Manufacturer of Structural Steel Forgings for Primary Offshore Applications, 1 edition, August 2013, reaffirmed 2020*

ASTM F2329/F2329M, *Standard Specification for Zinc Coating, Hot-Dip, Requirements for Application to Carbon and Alloy Steel Bolts, Screws, Washers, Nuts, and Special Threaded Fasteners*

ASTM F3125/F3125M, *Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 Mpa and 1 040 Mpa Minimum Tensile Strength*

EEMUA PUB NO 176, *Specification for structural castings for use offshore*

EN 13852-1, *Cranes — Offshore cranes — Part 1: General-purpose offshore cranes*

EN 1993-1-8, *Eurocode 3: Design of steel structures – Part 1-8: Design of joints*

ISO 898-1, *Mechanical properties of fasteners made of carbon steel and alloy steel — Part 1: Bolts, screws and studs with specified property classes — Coarse thread and fine pitch thread*

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 10684, *Fasteners — Hot dip galvanized coatings*

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

ISO 17776, *Petroleum and natural gas industries — Offshore production installations — Major accident hazard management during the design of new installations*

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 19901-9, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 9: Structural integrity management*

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

ISO 19903, *Petroleum and natural gas industries — Concrete offshore structures*

ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Ship-shaped, semi-submersible, spar and shallow-draught cylindrical structures*

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

ISO 19905-3, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 3: Floating units*

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

ISO 20088-1, *Determination of the resistance to cryogenic spillage of insulation materials — Part 1: Liquid phase*

ISO 20088-2, *Determination of the resistance to cryogenic spill of insulation materials — Part 2: Vapour exposure*

ISO 20088-3, *Determination of the resistance to cryogenic spillage of insulation materials — Part 3: Jet release*

ISO 22899-1, *Determination of the resistance to jet fires of passive fire protection materials — Part 1: General requirements*

ISO 834-1, *Fire-resistance tests — Elements of building construction — Part 1: General requirements*

NORSOK M-122, *Cast structural steel, rev. 2, October 2012*

NORSOK M-123, *Forged structural steel, rev. 2, October 2012*

3 Terms and definitions

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

ISO and IEC maintain terminology databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

3.1
active fire protection
equipment, systems, and methods which, following initiation, can be used to control, mitigate, and extinguish fires

[SOURCE: ISO 13702:2015, 3.1.3]

3.2
caisson
appurtenance used for abstracting water from the sea or as a drain

3.3
critical structure
structural components, forming parts of the topsides structure, that provide support to safety and environmental critical elements (SECE), loss of which can cause specific life-safety, environmental or business consequences

Note 1 to entry: Subclause 5.7 provides details and examples.

Note 2 to entry: Critical structure also includes structures which support safety and environmental critical elements (SECE) previously termed safety-critical elements (SCE), see ISO 19901-9:2019, 9.4 and A.9.4.

Note 3 to entry: SECE includes all relevant equipment and systems.

3.4
endurance period
time estimated for evacuation as defined by the Emergency Evacuation Rescue Analysis (EERA)

Note 1 to entry: The endurance period is specified in the basis of design.

3.5
major accident
MA
hazardous event that results in

— multiple fatalities or severe injuries; or

— extensive damage to structure, installation or plant; or

— large-scale impact on the environment (e.g. persistent and severe environmental damage that can lead to loss of commercial or recreational use, loss of natural resources over a wide area or severe environmental damage that will require extensive measures to restore beneficial uses of the environment)

Note 1 to entry: In this document, a major accident is the realization of a major accident hazard.

[SOURCE: ISO 17776:2016, 3.1.12, modified — Note 2 to entry deleted.]

3.6
passive fire protection
PFP
coating or cladding arrangement or free-standing system which, in the event of fire, provides thermal protection to restrict the rate at which heat is transmitted to the object or area being protected

[SOURCE: ISO 13702:2015, 3.1.36]

3.7
risk curve
probability of consequences exceeding a defined limit during a reference period

3.8
substructure
structure supporting the topsides

Note 1 to entry: The substructure can take many forms including fixed steel (see ISO 19902), concrete (see ISO 19903), floating (see ISO 19904-1 and ISO 19905-3), jack-up (see ISO 19905-1), or the various forms of arctic structures (see ISO 19906).

3.9
ideal hinge
pinned connection
idealisation by which no moments are transferred

Note 1 to entry: Plastic strain in an ideal hinge is typically a fraction of a percent.

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	environmental action
E_e	extreme quasi-static environmental action due to metocean and ice
E_o	environmental action due to metocean and ice for the operator defined operating conditions
F_d	design action
F_r	representative action
g	acceleration due to gravity
G	permanent action
I	explosion impulse
l	span or length
K_c	correspondence factor
p	instantaneous explosion overpressure
$p(t)$	variation of overpressure with time
P	probability
Q	operational action
R	resistance
R_d	design value of resistance
R_k	characteristic value of resistance, or value based on characteristic values of material properties
S_d	total design action effect
t	time from ignition of an explosion
t_d	duration of positive 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

ALARP as low as reasonably practicable

ALE abnormal level earthquake

ALS abnormal/accidental limit state

APoE annual probability of exceedance

ASD allowable stress design

AVM anti-vibration mounting

CFD computational fluid dynamics

CoG centre of gravity

CTOD crack tip opening displacement

DL damage limitation limit state

DLB ductility level blast

DES derrick equipment set

ELE extreme level earthquake

EER escape, evacuation and rescue

FEA finite element analysis

FEED front end engineering design

FPSO floating production storage and off-loading structure

FRP fiber reinforced polymer

IRPA Individual risk per annum

ISD inherently safer design

LRFD load and resistance factor design

MA major accident

MPI	magnetic particle inspection
MTOM	maximum take-off mass
MTOW	maximum take-off weight
NC	near collapse limit state
NTE	not-to-exceed
PFP	passive fire protection
SDOF	single degree of freedom
SECE	safety and environmental critical elements
SLB	strength level blast
SRPA	societal risk per annum
SWL	safe working load
TR	temporary refuge
U	utilization
UTS	ultimate tensile strength
VIV	vortex induced vibration
WSD	working stress design

5 Overall requirements

5.1 Conceptual design

Conceptual design shall include the determination of functional requirements and design criteria to obtain a workable and economical topsides layout and structure to perform its mission for a specified length of time (design service life).

Conceptual design shall include the development of structural and equipment layouts with due regard to congestion and confinement, which are important factors influencing the extent and magnitude of possible fire and explosion events and actions. Configuration of the facilities should be based on Inherently Safer Design (ISD) principles to eliminate or minimize the risks from hazards.

5.2 Codes and standards

5.2.1 Limit states and allowable stress philosophies

In general, the International Standards on offshore structures follow the approach of ISO 2394[1] and as such are intended to be limit state design standards.

NOTE 1 Limit state methods are also known as load and resistance factor design (LRFD) methods.

The intent of this document is that partial factor methods in accordance with ISO 19900, see 9.1, should be used where possible, but, where the substructure is designed using the WSD (Working Stress Design) method, such as for floating structures, the WSD method may also be used for the design of the topsides structure (e.g. AISC 335-89[72])). ASD may also be used where WSD is acceptable.

Note 2 In the more recent versions of AISC, i.e. AISC 360, ASD (Allowable Strength Design) has replaced WSD. ASD is a hybrid design approach using factored action combinations along with a safety factor applied to the resistance.

Where WSD methodology is used for the hull design in a floating facility and where partial factor design is used for the topsides, the appropriate partial action factors should be applied to the individual unfactored dead, imposed/live, and metocean actions applied to the hull.

Any dynamic response should be calculated using non-factored actions, and the resulting action effects should then be subjected to the appropriate partial factors.

The specific requirements and guidance in this document apply to the use of limit state (or LRFD) codes. Where an WSD or ASD code or standard is used, the requirements for allowable stresses and other factors shall be in accordance with the selected national building standard.

5.2.2 Use of national building standards

Topsides structures shall be designed and fabricated in accordance with this document and supplemented by the specific requirements from a national building standard as specified in this document. The national building standard shall be selected and agreed with the operator and regulator.

If the substructure is designed using the WSD method, as is often the case with hulls in floating facilities, and the topsides or any modules or assemblies are similarly designed using the WSD method, then no correspondence factor shall be applied.

5.3 Deck elevation

Air gap requirements are addressed in ISO 19901-1, ISO 19902, ISO 19904-1 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 substructure. These actions should be communicated to the substructure designer as early as possible.

For floating structures, and for ship-shaped 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 structure shall be designed to withstand these actions according to ISO 19904-1.

Deck elevation for platforms in arctic and cold regions with sea ice and iceberg hazards is addressed in ISO 19906.

5.4 Exposure level

The exposure level for topsides structure shall be determined in accordance with the criteria given in ISO 19900. Additional guidance is provided in ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 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 structures are not expected to be manned during governing design conditions and have medium consequences of failure; and L3 structures are normally unmanned and have low consequences of failure. These are more fully described in ISO 19900.

5.5 Operational requirements

5.5.1 Functional requirements

The functional requirements of the platform including drilling, process, access, safety and auxiliary systems, shall be established and documented, and communicated to design and operating personnel. In particular, the integrity of the structure shall conform 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 provided that collects and stores liquid spillages and overflows for subsequent handling (see 10.15).

Accumulation of combustible liquids in containment areas and on plated decks shall be mitigated through drainage systems and applied in fire risk studies.

5.6 Design physical environmental conditions

The topsides design physical environmental conditions (metocean, ice and seismic, see ISO 19900) for use in the relevant design/assessment situations shall correspond to those of the substructure. Specific short-duration design/assessment situations may adopt less onerous physical environmental conditions.

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

5.7 Critical structure

The following structural components shall be identified and recorded in design documentation as critical structure:

a) structural barriers, such as blast walls, that are intended to prevent a hazardous event from causing a major accident, or used to provide mitigation of the consequences.

b) structural components that support the weight of the TR (Temporary Refuge), including anti-vibration mounting (AVM), structural components that comprise the external surface of the TR, and structural components that comprise and support permanently occupied buildings;

c) structural components that support equipment required for EER (Escape, Evacuation and Rescue) including escape from the location of the hazardous event to the TR or muster location, and evacuation from the TR and muster location during the endurance period following a hazardous event;

NOTE 1 Endurance period for environmental risk in a fire event can be much longer than that required for personnel evacuation (life-safety risk). In such instances, mitigation can involve measures to reduce the blow-down time of systems containing hazardous inventories.

NOTE 2 Equipment required for EER typically includes:

— primary escape routes;

— lifeboats, life rafts, and means of escape to sea;

— fire, gas, and smoke detection in the TR;

— active fire protection (pumps and pipes);

— emergency generators.

d) structural components that prevent escalation (including supports for hydrocarbon-containing equipment and process safety critical piping systems, blast walls preventing escalation to the TR and fire walls preventing escalation to the TR, dropped object protection structure, riser impact protection structure, etc).

NOTE 3 Hydrocarbon-containing equipment typically includes:

— emergency shut-down valves/equipment (ESDVs);

— blowdown (venting and flare) equipment and piping;

— well isolation equipment;

— derrick equipment set (DES);

— hydrocarbon vessels and equipment (including all hydrocarbon-containing pipework and equipment such as generators that are fed gas);

— crane pedestals.

Other examples of structural components critical to topside structural integrity and/or business consequence are:

— topside deck post and brace framing into posts;

— drilling rig foundations;

— riser top tensioner foundation;

— flare boom foundation;

— helideck foundation.

NOTE 4 Further examples of critical structure are provided in ISO 19901-9:2019, A.9.4.

5.8 Assessment of existing topsides structure

Any assessment of existing topsides structure and structural components shall be performed in accordance with ISO 19900 and ISO 19901-9.

5.9 Reuse of topsides structure

Existing topsides structure or part of the structure can be removed and relocated for use at a new location. In such cases, the structure shall be assessed in accordance with ISO 19900 and ISO 19901-9, and for use (accounting for exposure level) and conditions applicable at the new location.

NOTE 1 Topsides structures present particular challenges as access for inspection is likely to be restricted by the plant and equipment and modifications can have been made to both the topsides structure and the equipment during the platform's original service life.

NOTE 2 In addition to survey work, all of the provisions applicable to a new design are likely to be relevant.

5.10 Repairs, modifications and refurbishment

Where repairs, modifications or refurbishment of an existing topsides structure are planned, the structure shall be assessed for the revised configuration in accordance with the basis of design and the assessment requirements of ISO 19901-9.

6 Design requirements

6.1 General

This clause presents the overall requirements for design of topsides structure.

The topsides design shall satisfy the fundamental requirements set out in ISO 19900.

6.2 Design/assessment situations

Design/assessment situations shall be established as specified in ISO 19900. They shall include all operational requirements, temporary conditions, physical environmental conditions, accidental conditions and post-damage conditions which could affect the design or assessment.

Each phase of the life of the structure and each mode of operation of the platform, such as drilling, production, work-over, or anticipated combinations thereof, shall be explicitly evaluated in design/assessment situations.

6.3 Material selection

Materials shall be selected that have a performance compatible with the design standard applicable for the structural components. ISO 19902 gives specific requirements and guidance for the selection of carbon steels that are applicable to topsides structure. Clause 11 addresses further requirements and alternative materials, including FRP, in 11.5.

6.4 Structural interfaces

Particular attention shall be paid to the following:

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

— distortions and displacements during fabrication, loadout, transportation, installation and in-service conditions;

— effects of relative displacements of different structures supported on one or more separate structures;

— consistency of forces (action effects) on both sides of interface.

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

NOTE Small displacements can be tolerated by adopting ductile design (see 10.2.2).

If a jack-up is to be located in close proximity to a topsides structure, its design shall ensure that under design metocean and seismic conditions there is no clash with primary steelwork and potentially manned areas (e.g. LQ, muster areas, emergency escape routes).

6.5 Design for serviceability

6.5.1 Serviceability limits

Design for serviceability shall include establishing design/assessment situations as specified in ISO 19900.

The serviceability of the topside structures can be affected by excessive relative displacement or vibration (vertical or horizontal). Serviceability limit states shall be established with respect to:

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,

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

Vibration limits are specified in 6.5.2 and 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 identified 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) transient fluid flow in piping systems, in particular slugging;

c) wind-induced vibrations (VIV) on slender elements (see 7.4);

d) global motions due to environmental actions on the facility;

e) earthquake and accidental events.

6.5.2.2 Design limits on vibrations

Design limits for vibration shall be established from operational limits set by equipment suppliers and by the requirements for 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 operator 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, should be proportioned to have a natural period of less than 1 s (see Reference [110]).

6.5.2.4 Dynamic analysis and avoidance of resonance

Dynamic analysis shall be used to assess the response of various parts of the topsides to ensure resonance is avoided. Such analysis shall include unfactored static and imposed actions.

To avoid resonance, the applicable frequency range is typically from 20 % below to 20 % above the excitation frequency range. For each frequency, the dynamic stiffness may approximately be taken as the force amplitude divided by the displacement amplitude in the direction of the force.

NOTE 20 % is considered sufficient to avoid resonances based on:

— theoretical margin to the resonance frequency to limit amplification;

— normal accuracy of actual excitation frequency compared to nominal;

— normal accuracy of predicted eigenfrequencies compared to actual eigenfrequencies;

— rotating machinery codes typically use a 15 % margin for rotor dynamic design, however, the 3 DOF rotor dynamic system is a less complex system and some more margin is needed.

Where heavy rotating machinery is installed (such as variable speed pump skids, compressors, etc.), three-dimensional vibration analysis should be performed. Simplified two-dimensional vibration analysis may be used for stand-alone components such as cantilevers.

6.5.3 Deflections

6.5.3.1 Vertical deflections

The final maximum defelction, Δ_{max}, of any element, structural component or structure comprises three parts as given in Formula (1):

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

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 serviceability limits or maximum values for vertical deflections are given in Table 1.

Table 1 — Serviceability limits for 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	—	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 operator 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. Cambering deck plating in areas susceptible to icing of any surface water can be used to avoid ponding.

Tolerances required for telecommunications equipment should be verified and maintained.

NOTE The alignment of telecommunications equipment can be critical for their reliable operation.

6.5.3.2 Horizontal deflections

Horizontal deflections shall be limited to 0,3 % of the height between floors. For multi-floor modules or structures, the total horizontal deflection shall not exceed 0,2 % of the total height of the modules or 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 strength

Design for strength shall include establishing operational, extreme, abnormal, accidental, and short duration design/assessment situations as specified in ISO 19900.

Methods of limit state verification are described in Clause 9.

6.7 Design for fatigue

Topsides fatigue design shall be in accordance with ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906, whichever is relevant, supplemented by the additional provisions given in Clause 10, particularly with respect to actions.

NOTE Pre-service fatigue damage during transportation can be significant depending on the length and severity of the environment of the transportation route (see A.6.7 for further guidance).

Pre-service (e.g. transportation) and in-service fatigue damage shall be combined to evaluate the total fatigue damage.

Where the substructure standard does not provide adequate fatigue guidance for a topsides structure, other suitable standards may be applied (see A.6.7).

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

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

Classification of components based on damage consequence	Not accessible for inspection or not repairable	Restricted access for inspection, maintenance and repair	Fully accessible for inspection, maintenance and repair
Substantial consequences	10	3	2
Without substantial consequences	5	2	1

NOTE 1 The γ_{FD} factors assume that inspections and maintenance intervals (see 6.13) are planned for all accessible areas, even if restricted, however the interval between inspections can be larger if the factor is higher. Inspection intervals and inspection methods are typically considered for the design for welds that are likely to crack at the weld roots.

NOTE 2 Restricted access can be for example flare structures, joints under removable passive fire protection, areas requiring rope access and areas with restricted access during normal operations.

NOTE 3 The γ_{FD} are associated to appropriate inspection intervals (see A.6.7).

When combining different materials such as steel and aluminium, the cyclic thermal effect shall not cause the fatigue limit state to be exceeded.

NOTE 4 The different materials will expand differently due to thermal effects.

NOTE 5 Fatigue damage for solar heating when using same material is generally negligible.

6.8 Robustness

6.8.1 General

Qualitative assessments to evaluate measures to improve the robustness of critical structure (see 5.7) shall be performed before detailed analysis.

6.8.2 Ductility

All failure modes shall be ductile.

Ductile failure modes ensure the structural behaviour will be in accordance with the anticipated model used for determination of the responses. In general, all design procedures, regardless of analysis method, will not capture the true structural behaviour. Ductile failure modes will allow the structure to redistribute forces in accordance with the presupposed static model. Brittle failure modes shall therefore be avoided or shall be verified to have excess resistance compared to ductile modes, and in this way protect the structure from brittle failure.

The following sources for brittle structural behaviour shall be considered for a steel structure:

a) unstable fracture caused by a combination of the following factors:

   1) brittle material,

   2) a design resulting in high local stresses,

   3) the possibilities for weld defects, and

   4) presence of fatigue cracks.

b) structural details where ultimate resistance is reached with plastic deformations only in limited areas, making the global behaviour brittle, e.g. partial butt weld loaded transverse to the weld with failure in the weld,

c) shell buckling, and

d) buckling where interaction between local and global buckling modes occur.

In general, a steel structure will be of adequate ductility if the following is satisfied:

— material toughness requirements are met, and the design avoids a combination of high local stresses with possibilities of undetected weld defects;

— details are designed to develop a certain plastic deflection e.g. partial butt welds subjected to stresses transverse to the weld is designed with excess resistance compared with adjoining plates;

— member geometry is selected such that the resistance does not show a sudden drop in capacity when the member is subjected to deformation beyond maximum resistance;

NOTE An unstiffened shell in cross section class 4, is an example of a member that can show such an unfavourable resistance deformation relationship (for definition of cross section class see EN 1993-1-1[13]).

— local and global buckling interaction effects are avoided; and

— adequate fatigue strength is ascertained to keep the probability of cracks sufficiently low.

6.9 Confirmation of execution of design requirements

For a new topsides, a walk-down survey shall be carried out in the fabrication yard prior to loadout. Remaining work which is missing from the walk-down survey and is performed post loadout shall be surveyed after installation (see baseline inspection in ISO 19901-9), but before production starts, in order to confirm that specific design requirements have been executed in the detailed fabrication and construction.

NOTE Walk-downs are methodical, on-site, visual evaluations of existing structures and equipment as installed.

6.10 Corrosion control

Corrosion control of topsides structure and structural components shall be performed in accordance with ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906, whichever is relevant.

In addition, 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 and operating criteria for the topsides structure. These criteria include:

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

b) the design service 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 ISO 19901-9).

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

NOTE Coating breakdown typically occurs a few years after installation on painted steel deck and corrosion thickness allowance can be considered as mitigation.

6.11 Design for fabrication and inspection

The designers shall be familiar with or anticipate likely methods of fabrication, welding and erection to execute the design and 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 service 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 operator.

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 document for materials (see Clause 11), quality control, quality assurance and documentation, welding and fabrication inspection (see Clause 12).

6.12 Design for loadout, transportation and installation

The objective of these design requirements is to ensure that a topsides structure begins its in-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 substructure and installing them to form the completed platform. Clause 13 contains further details on installation procedures.

Specific requirements, recommendations and guidance for marine operations, including loadout, transportation and installation are given in ISO 19902 and ISO 19901-6.

6.13 Design for structural integrity management

Structural integrity management shall be in conformance with ISO 19901-9.

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 the structures should provide access to the structural components to facilitate inspection and maintenance.

6.14 Design for decommissioning, removal and disposal

6.14.1 General

Decommissioning and removal shall be in accordance with ISO 19901-6 and ISO 19901-9, with the additional requirements given in this document.

Decommissioning and removal requirements shall be addressed during the topsides structure design phase, particularly for fixed platforms and floating platforms.

6.14.2 Structural releases

If practical, secondary structures between modules and elsewhere should be designed to be 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 take into account access for future disconnection.

6.14.3 Lifting appurtenances

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

The design should allow for periodic access for inspection.

6.14.4 Heavy lift and set-down operations

Dynamic impact during the operation of set-down onto a transportation vessel or barge shall be assessed in the design for decommissioning.

7 Actions and analysis methods

7.1 General

Topsides structure design/assessment situations (see 6.2) shall include:

— maximum (and minimum) operating conditions including drilling, variable inventory, and other operational actions;

— extreme and abnormal metocean and ice conditions, including the survival draft for floating structures;

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

— earthquakes;

— post-damage conditions, e.g. after abnormal or accidental events;

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

— fabrication;

— loadout;

— transportation;

— installation;

— decommissioning;

— removal and disposal.

Each design/assessment situation comprises several types of actions such as permanent, operational and environmental actions, deformations, temperature effects and accidental events, each with appropriate partial action factors.

These actions shall include both actions directly applied to the topsides and also the effects of actions on the supporting structure (such as metocean, ice and earthquakes). In addition, actions due to the motions of the supporting structure shall be assessed; these are particularly significant for floating structures.

Classification of topsides equipment and structure weights shall be in accordance with ISO 19900 supplemented by the provisions of this document. Guidance on weight management can be found in ISO 19901-5. Permanent actions (G) and operational actions (Q) shall be quantified for relevant weight items. In the early stages of the design process, such as in conceptual design and the early stages of FEED, weights are typically quantified by using top-down estimates and uniform area loads (UALs), with weight reserve. Uncertainty in these estimates is reduced during FEED as weights and associated CoG envelopes are more firmly established from weight take-offs, combined with weight allowance and estimate-to-complete.

Detailed engineering shall use not-to-exceed-weights (NTE weights) and associated CoG envelopes.

NOTE 1 Detailed engineering refers to project activities after FEED when the design is typically developed and refined to allow commencement of construction activities.

NOTE 2 Any UALs used in detailed engineering are typically calculated to represent the NTE weight and the CoG envelope.

NOTE 3 NTE weights are described in ISO 19901-5.

A description of operational actions (Q) is provided in ISO 19900 and is further described for Q_1 and Q_2 in 7.2. Specific requirements for operational loads are provided in each relevant subclause.

Specific structural response analysis methods are provided in this clause for certain categories of accidental actions.

7.2 In-service actions

Action types are described in ISO 19900, with more detail provided below.

Each topside structural component shall be assessed for total design action effect, $S_{d,}$ resulting from the design action, F_d. The design action shall be derived from action types for which principal actions are combined with appropriate companion actions as generally described in ISO 19900 and supplemented by the additional details of this Clause 7.

For operational design/assessment situations, design actions shall be derived for the following combinations of actions (see 7.3.1 and 7.3.2):

a) maximum permanent and operational actions, G_1, G_2, Q_1, and $Q_{2,}$ with and without accompanying environmental actions E_o and D_o;

b) minimum permanent and operational actions, G_1, G_2, Q_1, and Q_2, with and without accompanying environmental actions E_o and $D_{o.}$

For extreme design/assessment situations, design actions shall be derived for the following combinations of actions (see 7.3.3):

a) extreme environmental actions, E_e and D_e, combined with maximum permanent and operational actions G_1, G_2, Q_1;

b) extreme environmental actions E_e and D_e, combined with minimum permanent and operational actions G_1, G_2, Q_1,

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; in addition, any actions due to the misalignment of structures, such as between the topsides structure and the substructure, 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;

Q_1 is the operational 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; in addition, any actions due to the movement of substructures 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 substructure due to such effects, are part of Q_1;

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

E_e is the extreme quasi-static environmental action on the topsides structure and any environmental action effects transmitted through the substructure (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 substructures due to extreme environmental effects, such as roll of an FPSO, including any consequent flexure of the substructure 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 substructure due to operator-defined operating environmental conditions; for particular operations these can be the limiting environmental conditions for the particular operation (see 7.3.2); in addition, any actions due to the movement of substructures due to the operating environmental effects, such as roll of an FPSO, including any consequent flexures of the substructure 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.

NOTE Operational actions (Q) include all variable actions that are neither environmental (including earthquakes) nor accidental.

For all design/assessment situations, any action resulting from associated substructure movement shall be included.

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 shall 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 evaluated, particularly with respect to design for loadout, transportation and installation of the topsides.

General guidance on operational actions (Q) on deck areas is provided in A.7.2.

7.3 Action factors

7.3.1 Design actions for operational design/assessment situations in still water

Design action, F_d, for operational design/assessment situations in still water, with maximum permanent and operational actions only, shall be calculated using Formula (2).

$$F_d = \gamma_{f,G1} \cdot G_1 + \gamma_{f,G2} \cdot G_2 + \gamma_{f,Q1} \cdot Q_1 + \gamma_{f,Q2} \cdot Q_2 \quad (2)$$

The partial action factors, γ_f, shall be the same as those used for the substructure design or assessment, in accordance with ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 or ISO 19906, as appropriate.

For components where relevant, design actions shall also be calculated using maximum and minimum values of G and Q positioned spatially in loading patterns to cause the most onerous action effects in the component.

7.3.2 Design actions for operational design/assessment situations with operating environmental actions

Each operation shall be assessed in an operational design/assessment situation using Formulae (3) and (4), in which E_o and D_o represent the operating environmental actions defined by the operator either generally or as the limit for that particular operation.

Design actions, F_d, for operational design/assessment situations on fixed structures with maximum values of G_2, Q_1 and Q_2 shall be calculated using Formula (3).

$$F_d = \gamma_{f,G1} \cdot G_1 + \gamma_{f,G2} \cdot G_2 + \gamma_{f,Q1} \cdot Q_1 + \gamma_{f,Q2} \cdot Q_2 + \gamma_{f,Eo} \cdot (E_o + \gamma_{f,D} \cdot D_o) \tag{3}$$

If the internal forces due to minimum G_2 and Q_1 (Q_2 = 0) permanent and operational actions oppose those due to environmental actions, design actions, F_d, shall be calculated in accordance with Formula (4) using reduced partial action factors for the permanent and operational actions.

$$F_d = 0{,}9 \cdot G_1 + 0{,}9 \cdot G_2 + 0{,}8 \cdot Q_1 + \gamma_{f,Eo} \cdot (E_o + \gamma_{f,D} \cdot D_o) \tag{4}$$

For this check in Formula (4), G_2 and Q_1 shall exclude any actions associated with the design/assessment situations being assessed that cannot be ensured of being present during the specified operating environmental conditions.

Design actions for floating structures shall be calculated using the same formulation but including rotation of the topsides and inertial actions due to the vessel motions.

7.3.3 Design actions for extreme design/assessment situations

Design actions, F_d, for the extreme design/assessment situations for topsides on fixed structures with maximum values of G_2 and Q_1 shall be calculated using Formula (5).

$$F_d = \gamma_{f,G1} \cdot G_1 + \gamma_{f,G2} \cdot G_2 + \gamma_{f,Q1} \cdot Q_1 + \gamma_{f,Ee} \cdot (E_e + \gamma_{f,D} \cdot D_e) \tag{5}$$

Design actions for floating structures shall be calculated using the same formulation but including rotation of the topsides and inertial actions due to the vessel motions.

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

If the internal forces due to minimum G_2 and Q_1 permanent and operational actions oppose those due to environmental actions, design actions, F_d, shall be calculated in accordance with Formula (5) using reduced partial action factors for the permanent and operational actions as used for the substructure design or assessment (see ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, or ISO 19906, as appropriate).

For the check involving opposing actions, the minimum G_2 and Q_1 shall exclude any actions that cannot be ensured of being present during the extreme environmental conditions in that design/assessment situation.

NOTE The appropriate partial action factors for the extreme 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.

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.

Wind induced fatigue analysis and vortex shedding analysis shall be performed on lattice structures (e.g. flare booms and drilling derricks) and exposed pipework.

7.5 Indirect actions and resulting forces (action effects)

Internal forces due to imposed deformations of fabrication tolerance and foundation settlement may be neglected in the verification of limit states (excluding fatigue) when fabrication requirements (see 12.1) are satisfied. Internal forces due to differential temperature under normal conditions such as solar heating may also be neglected.

NOTE 1 Internal forces can be divided into two groups, those that react the external actions, and those in internal equilibrium that redistribute and dissipate as the structural system reaches its limit state for strength.

Subclause 10.2.1 gives allowable procedures and simplifications that may be applied when determining the internal forces without altering the global equilibrium.

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 substructure and the topsides structure should be analysed together for the verification of the relevant limit states. If not, consistent forces on both side of the interface shall be applied.

For a floating structure the differences between essentially static behaviour due to ballast and cargo loading, and dynamic behaviour due to environmental effects shall be assessed.

NOTE 2 The hull of a monohull is usually much stiffer than the topsides structure and, as it sags and hogs, significant deformations can be introduced at the topsides structure level.

In case of temperature effects larger than solar expansion, the capability to expand shall be assessed to ensure either ability to expand or to ensure ductility of the structure.

NOTE 3 Sliding bearings can be an effective solution for high-temperature pressure vessels.

7.6 Metocean and ice actions

7.6.1 Wave, current and ice actions

7.6.1.1 General

All wave, current, and ice actions on the substructure, appurtenances (e.g. conductors, risers, caissons, etc.) and the topsides shall be included in the calculation of the metocean and ice effects on the topsides.

NOTE 1 Although wave, current and ice actions mainly affect the substructure directly, there are frequently indirect actions on the topsides due to the displacements and deformations of the substructure.

NOTE 2 For the purposes of this subclause, ice means ice on the sea which includes sea ice and icebergs, as described in ISO 19906 for arctic and cold regions. Such ice can move both horizontally and vertically.

Where insufficient air gap exists to prevent wave or ice run-up or ice ride up against large diameter legs or columns striking the deck, or when water inundates the deck of a floating facility (such as green water effects), all actions resulting from the water or ice 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.

7.6.1.2 Fixed platform

Primary topsides structure on fixed platforms shall be designed for:

a) reactions on the topsides due to metocean and ice actions on conductors and appurtenances;

b) framing actions due to horizontal actions on the substructure;

   NOTE Wave actions on a multi-leg concrete platforms can act in opposite directions on the legs, resulting in forces and moments applied to the topsides that can be a maximum in conditions that are less than the extreme metocean conditions.

c) inertial actions resulting from acceleration due to dynamic response of the platform and deck during extreme or abnormal metocean or ice conditions.

Supports for safety and environmental critical elements (SECE) shall be designed using consistent forces for the substructure and the SECE supports.

Cellar deck bottom of steel elevation should be set to avoid wave-in-deck or ice-in-deck actions at the abnormal metocean or ice action probability or return period relevant for the exposure level of the structure. Where this is not possible, topsides primary structure, supports for critical equipment, and the substructure shall be designed for wave-in-deck or ice-in-deck actions.

Bottom of steel elevations of sub-cellar deck structures set below the abnormal wave crest shall be designed for wave-in-deck or ice-in-deck actions.

7.6.1.3 Floating facilities

Primary topsides structure and structural supports for SECE on floating facilities shall be designed for:

a) inertial actions resulting from motions (sway, surge, heave, roll, pitch and yaw);

b) rotation of the topsides due to roll and pitch with the consequent effects on the directions of actions;

c) the actions imposed due to deformation of the substructure.

7.6.2 Wind actions

The physical environmental data for offshore wind speeds and profiles shall be in accordance with ISO 19901-1. Where available, site specific wind speed data shall be used.

The return period of the wind speed shall be consistent with design of the substructure, with partial action factors in accordance with the relevant standard used for the substructure (i.e. ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906).

Representative wind actions shall be calculated in accordance with a national building standard selected and agreed with the operator and regulator. This standard should be consistent with the national building standard selected and agreed in 5.2.2.

The calculation of the representative wind actions, including wind drag coefficients, shielding effects, upward, sideward, windward and leeward factors and any dynamic effects and spatial and temporal coherence (by gust factors or gust duration) on the topsides shall be in accordance with the methodology in the national building standard selected for wind actions.

The time averaged wind speed for the basic wind speeds, which is the reference wind speed that the spatial and coherence factors are applied to, shall be based on the national building standard selected for wind actions.

NOTE The time averaged wind speed for the basic wind speed varies depending on the national building standard selected. As an example, for EN 1991-1-4[2], the reference is the characteristic 10-minute mean wind velocity for terrain category II, whereas for AISC/ASCE 7-22[3] it is the 3-second gust speed for exposure category C.

In the absence of more accurate wind tunnel testing or CFD, the same calculation of wind actions used for the topsides should be applied in the design/assessment situations for the substructure as the principal action, to ensure that relevant parts of the substructure are designed to act as an adequate foundation for the topsides.

Wind actions shall be calculated for sufficient directions, including diagonal or quartering directions, to ensure all governing local or global failure modes have been investigated. The calculations for each direction shall take account of the projected area and shape coefficients appropriate for each direction.

Wind actions on items in laydown or storage areas shall also be included in the wind action calculation for local and global effects.

7.6.3 Cold regions effects

7.6.3.1 Snow accumulation

Snow accumulation upon the topsides shall be accounted for in accordance with ISO 19906.

7.6.3.2 Ice accumulation

In areas where icing or icing accumulation can occur, the effects of both the weight of ice accretion and the increase in effective dimensions of components due to the ice shall be included in accordance with ISO 19906.

NOTE Increased wind actions on topsides structure and equipment due to icing can be significant.

7.6.3.3 Effects of cold weather

The effects of cold weather on ballast water, firewater etc. which can freeze shall be taken into account in accordance with ISO 19906.

7.7 Seismic actions

7.7.1 General

The topsides structure shall be analysed and designed for earthquake conditions as part of the overall platform consisting of the substructure with its foundation, where applicable, and the topsides in accordance with ISO 19901-2. Additional guidelines for components of the topsides are given in 7.7.2 and 7.7.3.

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.

Restraints for SECE and for other piping and equipment whose failure could result in personnel injury, hazardous material spillage, damage to the environment, or hindrance to emergency response shall be designed appropriately.

Design acceleration levels shall include the effects of overall platform dynamic response, and local dynamic response of the deck and any appurtenances as appropriate.

NOTE Due to the platform's dynamic response, the design acceleration levels can be much greater than the ground motions and hence greater than those commonly associated with the seismic design of similar onshore processing facilities.

7.7.2 Minimum lateral acceleration

Topsides primary structure and critical structure shall be designed for inertial actions resulting from acceleration during a seismic event.

A minimum lateral acceleration of 0,05 g shall be adopted as the extreme level earthquake (ELE) to apply to topsides, including equipment and supporting framework, for all fixed structures including those in seismic zone 0.

7.7.3 Equipment and appurtenances

Connections of equipment to the main topsides structure shall be designed for inertial actions resulting from acceleration during a seismic event.

Where the dynamic response of the equipment is independent of the dynamic response of the deck supporting structure, inertial actions may be calculated by the product of the deck acceleration with the mass of the equipment.

The inertial actions shall be calculated by a coupled dynamic analysis when both of the following apply:

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

b) the natural frequency of the equipment or appurtenance is greater than 50 % of the main topsides lateral natural frequency.

NOTE 1 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.

NOTE 2 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 conformance with API Spec 4F[22] (see A.7.7.3) and shall be tied down or always restrained except when being moved.

7.8 Actions during fabrication, loadout, transportation, and installation

Design/assessment situations for each temporary condition shall be established and relevant structure shall be verified to satisfy relevant limit states. See also 6.12. In 7.5 and 10.2.1 allowable simplifications are provided.

NOTE Temporary conditions include jacking and weighing conditions.

Individual support reactions during fabrication, loadout, transportation, and installation depend on the stiffness of the topsides structure and of the supporting foundation or supports.

Topsides structure and structural components shall be designed for inertial actions resulting from acceleration during loadout and transportation.

7.9 Actions arising from accidental events

7.9.1 General

7.9.1.1 Accidental hazards

The following accidental hazards, including major accident (MA) hazards, shall be identified, and characterized:

a) hydrocarbon fire or explosion due to an ignited release from topsides piping or equipment (including piping on bridges);

b) hydrocarbon fire or explosion due to an ignited release from a well;

c) hydrocarbon fire or explosion due to an ignited release from a riser;

d) hydrocarbon fire or explosion due to an ignited release from a subsea pipeline within 500 m of the installation;

e) non-process fire at any location on the installation;

f) non-process hydrocarbon or flammable material event;

g) collision (supply vessels, passing vessels and helicopter emergency landing or crash landing);

h) impact (dropped or swinging mass) resulting in hydrocarbon release escalating to fire and explosion or flooding of a compartment on a floating substructure;

i) impact from collapse or displacement of adjacent structures;

j) flooding in a floating substructure resulting in loss of stability.

NOTE 1 Adjacent structures include jack-up platforms and bridge-linked platforms and structural components such as derricks, flare towers and cranes.

NOTE 2 Adjacent structures can collapse onto and significantly damage the Temporary Refuge (TR) or its systems, or obstruct escape and evacuation routes.

When characterising hazardous events and accidental actions, close communication should occur between the safety engineers, the hazard expert engineers and the project structural engineers.

The probabilities of exceedance (or return periods) for determining representative values of accidental actions for fire and explosion for each design/assessment situation and limit state shall be determined in accordance with ISO 19900.

7.9.1.2 Design for accidental events

Topsides structure shall be designed for hazardous events arising from accidental hazards as identified in 7.9.1.1.

Interaction between the topsides structure and the substructure shall be included if forces in the topsides structure are affected by the deformation of the substructure (see 10.1 and 10.2).

Topsides structure shall be designed using ISD in conformance with ISO 17776 to eliminate credible fire and explosion MAs or to reduce their potential consequences by design measures that are inherent in the design, being permanent and inseparable features of the facility.

Topsides structure shall be designed for control and mitigation of fires and explosions in conformance with ISO 13702.

For fire and explosion hazards, the verification to limit states and determination of design controls may use a risk-informed approach or a deterministic (consequence-based) approach.

Tolerable-if-ALARP risk criteria shall be established where a risk-informed approach is used (see example Figure 5).

7.9.1.3 Intensity of hazardous event

The annual probability of exceedance of a hazardous event with given intensity should be determined from the hazard curve (see Example in Figure 1, which is not intended for use in actual design).

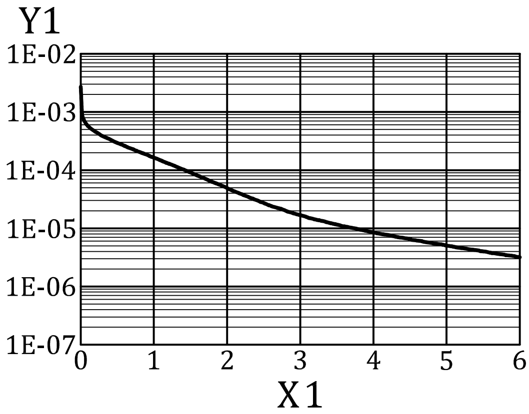
NOTE The hazardous events intensity is also known as the DAL (dimensioning accidental load).

For fire and explosion:

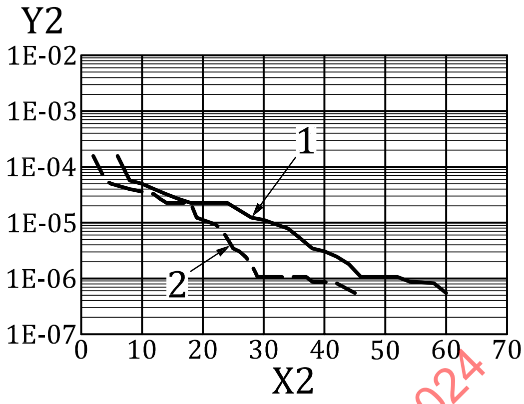
a) a risk-informed approach may be used rather than using representative actions in a semi-probabilistic approach;

b) a deterministic (consequence-based) approach, based on the worst credible action, may be used to determine the representative action.

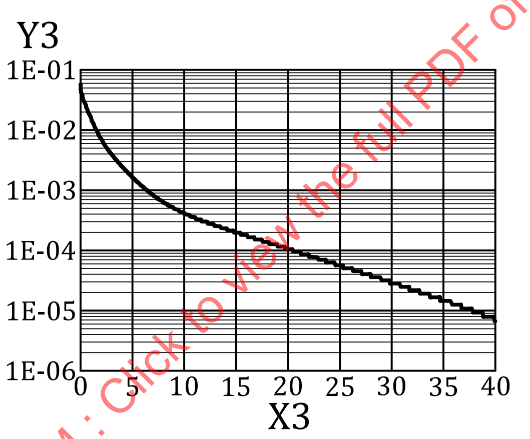
NOTE Option b) above typically applies where the risk resulting from a hazardous event is likely to be tolerable and ALARP; approximate and conservative estimates of the intensity of the hazardous event can be based on worst credible consequence modelling.



a) explosion hazard curve
X_1=overpressure (MPa)
Y_1=annual probability of exceedance of overpressure



b) fire hazard curve
X_2=flame length (m)
Y_2=annual probability of exceedance of flame length
1 t = 0 min
2 t = 2 mins



c) collision hazard curve – platform/supply vessel
X_3=collision energy (MJ)
Y_3=annual probability of exceedance of collision energy

These example hazard curves are not to be used for design.

Figure 1 — Example hazard curves

7.9.2 Structural design for fire hazard

Design of topsides critical structure shall be based on fire scenarios and hazard curves listed in the Fire Hazard Analysis (FHA), including jet fires and pool fires.

NOTE 1 Magnitude of the fire hazard is described in terms of a hazard curve relating fire sizes to the exceedance probability.

Where FHA is not available, approximate heat flux data for various fire types may be taken from tables 3 to 6 of Reference [4].

Data for steel yield strength and Young's modulus (as a function of temperature) may be taken from tables 8 and 9 in Reference [4] and from References [6] and [7].

PFP or fire walls may be used to achieve tolerable-if-ALARP risks.

A coat-back analysis may be performed to optimise the extent and length of the PFP. Further details are provided in A.7.9.2.6. In any case the minimum coat-back length shall be 150 mm.

NOTE 2 The additional cost of removal and replacement of PFP, to allow inspection and maintenance of topsides critical structure over the service life of the facility, can reduce the perceived cost-benefit of PFP.

NOTE 3 Hydrocarbon fires generate peak temperatures that can exceed 1 400 °C over a large area. At these temperatures unprotected steelwork can collapse in a short time.

7.9.3 Structural design for explosion hazard

7.9.3.1 General

Design of topsides critical structure:

a) should be based on explosion scenarios and hazard curves (when available in the Explosion Hazard Analysis, EHA);

b) may be based on industry generic overpressure values[8];

c) may be based on worst credible overpressure values (where the design of the topsides structure is not sensitive to such overpressure).

NOTE Representative congestion method (RCM) and, more specifically, the anticipated congestion method (ACM) reduces the potential for overpressures determined from CFD models to increase during development of the topsides design (as the congestion increases with greater definition of the design), see Reference [104].

7.9.3.2 Explosion actions

Explosion actions shall be applied:

a) as a time-varying overpressure to large flat surfaces (e.g. blast walls and decks);

b) as a time-varying overpressure difference loading on vessels, pipework, steel structures, and other obstructions of 1,0 m diameter or greater.

NOTE 1 Explosion actions on vessels ≥1,0 m diameter due to the ‘out of balance loads’ and ‘peak blast overpressure’ are typically determined from the CFD analysis and specified in the EHA.

c) as a time-varying blast drag load on vessels, pipework, steel structures, and other obstructions of less than 1,0 m diameter.

Blast drag actions on the structure, vessels, or pipework less than 1,0 m diameter shall be determined from Formula (6):

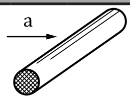
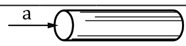
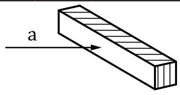
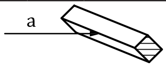
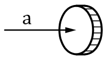
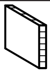
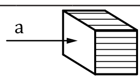
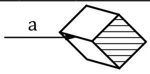
$$F_{\mathrm{d}} = C_{\mathrm{d}} \times q_{\mathrm{d}} \times A \qquad (6)$$

where

F_{d}	blast drag force applied to element (kN);
C_{d}	drag coefficient (from Table 3);
q_d	blast drag wind pressure (kN/m^2);
A	projected area of element (m^2) including fire protection.

NOTE 2 In lieu of specific data from CFD analyses, the blast drag wind pressure can be taken as 1/3 of the blast overpressure at the same location. This approximation is based on review of many CFD analyses.

Table 3 — Drag coefficients for component shapes

shape	sketch	C_d
cylinder (side-on)		1,2 to 2,0[b]
cylinder (end-on)		0,82
rectangular prism (face-on)		2,05
rectangular prism (edge-on)		1,55
disk (face-on)	 or 	1,17
cube (face-on)		1,05
cube (edge-on)		0,80

[a] Flow.

[b] C_d approximately 2,0 can occur for pipes in a strong explosion with Mach-numbers approaching 1,0. [9]

Blast drag pressure on grating shall be applied to the total grated area (not the area of the grating bars).

NOTE 3 Loading data for grated deck areas is typically determined from the CFD analysis (taking into account porosity) and specified in the EHA. FRP grating has a lower porosity than steel grating.

7.9.3.3 Overpressure time-history

Overpressure time-history for structural response shall use a simplified form as shown in Figure 2 where the rise time is:

a) $t_d/2$ (when within the combustion zone);

b) zero (when outside the combustion zone for explosion events that transition to a shock wave).

Where CFD is performed, the overpressure for structural design should be averaged from panels in the CFD model over an area of approximately 3 m^2.

NOTE 1 Point pressure from the CFD model is not suitable for structural design use as it incorporates very short duration spikes that the structure does not respond to.

For large areas including large blast walls, different blast loads recognizing time delays may be considered.

Explosion hazard curves, that plot the annual probability of exceedance of overpressure and the annual probability of exceedance of impulse on critical structure, should be used to select the peak overpressures (rather than use of deterministic worst credible values).

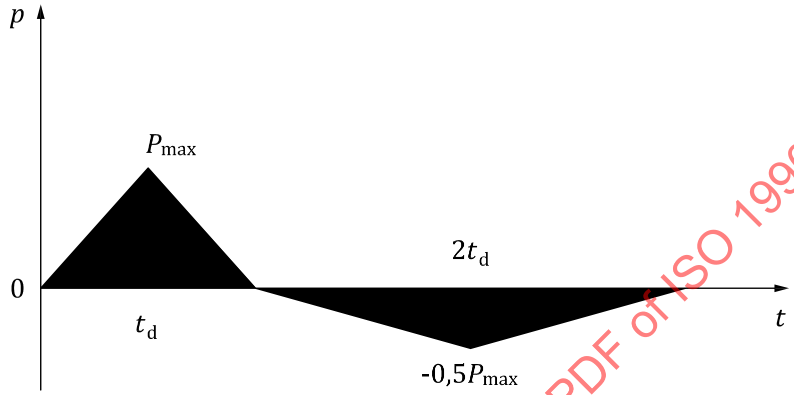
NOTE 2 Impulse exceedance curves can also be useful in selecting the appropriate time-histories for use to determine the structural response. Impulse exceedance is included in the worked example in A.9.3.3.

Evacuation hazardous events for explosion (also known as ductility level blast, DLB, events) shall have a minimum positive overpressure duration of 40 ms.

Controllable hazardous events for explosion (also known as strength level blast, SLB, events) shall have a minimum positive overpressure duration of 120 ms.

NOTE 3 Peak pressure in the positive phase is typically specified by the EHA. In addition, a range of durations of the positive phase are typically specified in the EHA. Peak negative pressure is typically not specified in the EHA and CFD results are not reliable for negative pressure levels. Peak negative pressure is typically between 20 % and 50 % of the peak positive pressure.

NOTE 4 Rebound of members from passing of blast wave can be a critical design condition due to unsupported compression flanges. Rebound can sometimes produce complex load interactions as the blast duration time can be different to the natural recovery period of the structure, resulting in a severe load combination, e.g. equipment-load and self-weight can be additive to rebound load, vertical walls can develop membrane compression and deck beams can have stress reversals.



Key

t time duration

p overpressure

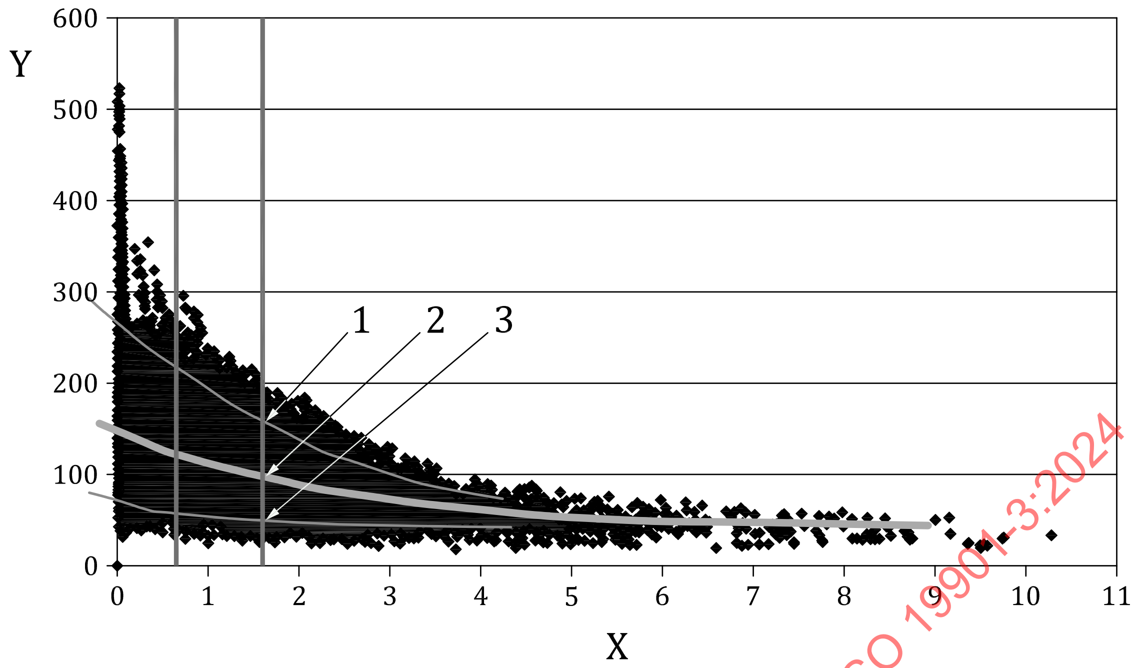
P_{max} peak overpressure

t_d time duration of the positive overpressure

Figure 2 — Overpressure time history

Structural design should account for probability of response severity due to probability of overpressure duration.

In lieu of overpressure duration data provided in the EHA, Figure 3 may be used to estimate the upper, lower, and most probable overpressure duration.



Key

X overpressure (10^5 Pa)

Y pulse duration (ms)

1 approx upper bound t_d value for P_{max} = 1,6 × 10^5 Pa

2 probable t_d value for P_{max} = 1,6 × 10^5 Pa

3 approx lower bound t_d value for P_{max} = 1,6 × 10^5 Pa

Figure 3 — Variability in pulse duration versus peak overpressure

NOTE 5 Selection of the peak pressure, pulse duration, time delay and area over which the pressure is approximately constant, depends on whether the blast wall acts as a single or two-way spanning.

7.9.3.4 Structural response analysis

The structural response shall be determined by non-linear time-history analysis using either:

a) SDOF approximation with material non-linearity;

b) non-linear FE dynamic analysis with material and geometric non-linearity.

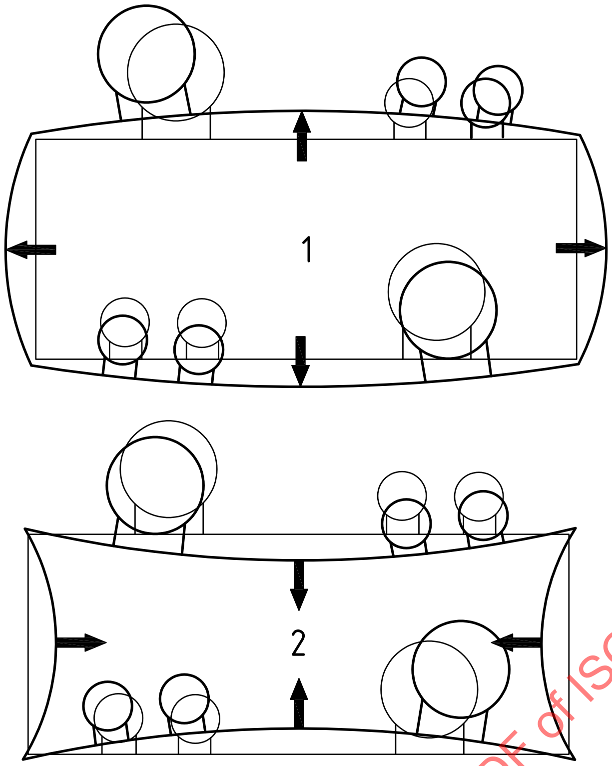
NOTE 1 SDOF for response to blast load is typically used for initial design of primary structure and for detailed design of secondary structure.

Data for steel yield strength, ultimate tensile strength (UTS) and fracture strain may be taken from tables 16 and 17 of Reference [4] and from Reference [10] in clause 5.5, Part 3.

Structural model shall include the roof, adjacent walls and floors of the module or deck if structural interaction is expected and wall reactions shall be included in design of roof and floor beams.

Interaction between components shall be accounted for in the structural analysis.

NOTE 2 Figure 4 illustrates interaction between components where overpressure acting on the roof and/or floor causes membrane tension in the wall that is additive to the bending and membrane tension in the wall due to the direct application of the blast pressure.



Key

1 blast

2 rebound

Figure 4 — Structural interaction between roof, wall, and floor

7.9.3.5 Simplified design methods

Simplified methods, such as equivalent static analysis or response charts, may be used for the design of structural components as described in A.7.9.3.5.

NOTE Simplified methods do not represent the non-linear response and load redistribution effects that can be important for efficient design against large blast loads.

7.9.3.6 Explosion mitigation

Explosion effects should be mitigated as far as reasonably practical.

Barriers such as explosion walls and floors should maintain integrity after the explosion.

7.9.4 Explosion and fire interaction

Topsides design of critical structure shall include the consequences of explosion and fire scenarios with either event occurring first.

NOTE 1 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 follows a fire (as there is little or no unburnt gas cloud in a fire event) unless additional inventory is released due to the fire.

Following an explosion event, the relevant critical structure risks shall be tolerable for the endurance period in a subsequent fire event (to allow a controlled evacuation from the TR).

NOTE 2 An example critical structure risk is the risk of a fire wall collapsing during an explosion or during the following endurance period (resulting in escalation if a fire follows the initial explosion).

NOTE 3 ISO 23693[11] provides test requirements for PFP to simulate the mechanical loads that could be imparted to PFP materials and systems by explosions resulting from releases of flammable gas, pressurised liquefied gas or flashing liquid fuels perhaps precede a fire.

7.9.5 Cryogenic spill

Structure subjected to cryogenic spill or vapor jet shall be protected using a cryogenic spill protection (CSP) system that is in accordance with ISO 20088-1, ISO 20088-2, ISO 20088-3.

NOTE Cryogenic liquids such as LNG or refrigerants can be present at facilities with liquefaction modules. Cryogenic liquid spill and vapor jets can rapidly cool structural steel members resulting in brittle fracture. Design against exposure to cryogenic temperatures can be performed in a similar way with fires with the difference of adjusting material properties for low temperatures. Cryogenic spills are usually followed by fires and typically dual-purpose systems are specified that can function as CSP and PFP to mitigate both types of risk.

7.9.6 Actions due to vessel collision

The annual probability of exceedance of a collision hazardous event of given intensity should be determined from the hazard curve in conformance with 7.9.1.3.

Where the risk resulting from a hazard is likely to be negligible or broadly acceptable, approximate and conservative estimates of the intensity of the hazardous event may be used in lieu of a hazard curve.

Topsides critical structure, as defined in 5.7, shall be designed for the inertial actions resulting from the collision (see 7.9.9 and A.7.9.9).

7.9.7 Actions due to dropped and swinging objects and projectiles

The annual probability of exceedance of a dropped object hazardous event of given intensity should be determined from the hazard curve in conformance with 7.9.1.3.

Where the risk resulting from a hazard is likely to be negligible or broadly acceptable, approximate and conservative estimates of the intensity of the hazardous event may be used in lieu of a hazard curve (e.g. using the crane operating conditions and following any restrictions on heights of lifts above decks).

Topsides critical structure, as defined in 5.7, shall be designed for the actions resulting from the dropped object.

Risk treatment shall be used to mitigate the risk of damage to topsides critical structure and subsequent life-safety risk or operational risk.

Operational procedures should limit the exposure of personnel to overhead material transfer.

7.9.8 Actions due to loss of buoyancy

For floating structures, the minimum criteria for compartment damage (see ISO 19904-1) shall be achieved.

Topsides critical structure, as defined in 5.7, should be designed for the actions resulting from loss of buoyancy (i.e. actions resulting from heel and trim or flooding of hull).

See A.7.9.8 for guidance.

7.9.9 Actions due to topsides acceleration

Topsides primary structure, secondary structure, and structural supports for SECE shall be designed for topsides acceleration due to:

a) wave actions, including breaking waves (see 7.6);

b) ice actions, if applicable (see 7.6);

c) seismic events (see 7.7);

d) pre-service conditions, i.e. fabrication, load-out, transportation and installation (see 7.8);

e) gas explosions (see 7.9.3), ship collision (see 7.9.6), helicopter emergency landing or crash (see 10.5.3.3), sudden failure of loaded cables;

f) mechanical equipment.

To account for accidental actions transmitted to the topsides that are not specifically accounted for otherwise, a minimum lateral acceleration of 0,05 g shall be applied to the topsides, including SECE (safety critical equipment and systems, piping, and supporting framework) in order to ensure a minimum level of anchorage and connectivity for such components. This applies to topsides on both fixed and floating facilities.

Failure or excessive deformation of individual components that could lead to failure of SECE shall be avoided.

See A.7.9.9 for guidance.

7.10 Other actions

7.10.1 Drilling

Platforms with drilling operations contribute a variety of major actions to the topsides. These include actions caused by weights, drilling operations generally, and environmental conditions.

Drilling equipment and materials include the derrick equipment set (DES), see 10.7, which moves from well to well, and the normally stationary drilling-related weights installed on the deck, separate from the DES, usually on the drilling deck level.

The CoG of the DES depends on its position over each well slot. The variable weights installed on the deck separate from the DES are typically stationary, but some can also change location depending on the DES position.

The designer should consult and coordinate with project/asset drilling engineers to identify the appropriate moveable and stationary permanent and variable actions and load combinations for each design situation on a given project. ISO 19901-5 provides guidance on typical concurrent weight items for drilling. In particular, ISO 19901-5:2021, Annex D provides guidance on typical variable weights.

Drilling weights can be classed as follows:

— weights of components where weight is constant but CoG varies, such as the drilling derrick itself, typically classed as permanent actions type G_2 (see 7.2);

— variable weights from drilling operations, such as weight of drill pipe and casing, weight of well cuttings, weight of consumables including drilling fluids, cement, and powders, typically classed as operational actions, either Q_1 or Q_2 (see 7.2).

Principal and accompanying actions shall be identified and included in design/assessment situations for the following conditions:

a) drilling actions with variable weights and CoG for a range of derrick positions and storage dispositions for casing and drill pipe to ensure that all maximum action effects in the supporting structures are identified;

NOTE 1 A typical maximum/minimum condition is one where the DES is positioned so that the drilling derrick is over a corner slot farthest from the wind (and wave) direction.

b) increased (maximum) action from the drilling derrick when it is pulling, including yanking and jarring, on a stuck drill string or stuck casing. The pulling action on stuck casing is typically concurrent with drill string stored in setback;

NOTE 2 Yanking and jarring occur when the drawworks are used to impose repetitive vertical impulse actions on the drill string or casing. (i.e. repetitive shock actions such as when the drill crew attempts to free a casing string or drill string / drill bit that has become stuck downhole).

NOTE 3 The maximum action is usually based on the design rated capacity of the derrick.

c) minimum actions, with minimum variable weights and without drilling operations occurring (for checking stability and uplift under reverse environmental actions);

NOTE 4 A setback full of 90 ft high drill string increases the effect of the wind overturning actions, thereby further increasing the potential for uplift on the upstream side.

NOTE 5 Setback is the storage area or areas for drill string stacked vertically within the derrick.

d) actions from skidding the derrick substructure over the skid base and from skidding the skid base on the supporting topsides deck beams/girders, for sufficient locations to identify:

— maximum actions resulting from all possible relative positions of the structures to be evaluated;

— horizontal frictional forces and bearing stresses, including racking actions, that can result from a stuck jack.

e) actions caused by the temporary support of the casing or drill string from the drill floor rotary table.

Drilling action combinations in operational design/assessment situations shall be with and without accompanying operating environmental actions.

Drilling operational actions accompanying environmental actions in extreme and abnormal design/assessment situations may be reduced but only if drilling operations are suspended under extreme forecast or imminent environmental conditions. Typical reductions in such situations include the elimination of any hook load where the drill string would be stored in setback or supported by the rotary table.

NOTE 6 Drilling actions can be reduced if the relevant operating procedures are documented in the project operations manual or equivalent. For example, drilling-related actions are typically reduced for design situations where the operations manual specifies that drilling be suspended whenever a storm exceeding prescribed criteria is predicted.

Operational actions accompanying accidental actions such as due to dropped objects shall be determined for accidental design/assessment situations.

7.10.2 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 considered.

Where drilling is performed from a derrick cantilevered from a jack-up through a platform conductor, design/assessment situations shall include forced displacements and any consequent actions on the conductor and its supports due to the relative movement between the structure and a representative drilling jack-up.

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.

7.10.3 Risers

In addition to normal actions due to weight, supports for risers shall consider 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.

7.10.4 Caissons

In addition to normal actions due to weight, supports for caissons shall consider 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.

7.10.5 Maintenance, mechanical handling and lifting aids

Design of the structure shall take into account the actions resulting from equipment maintenance, operational 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 combinations that do not satisfy design criteria. 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 evaluated. These effects include the potential negative influence on local stability of webs and flanges caused, for instance, by differential displacements of beam supports.

7.10.6 Bridge supports

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

The following potential actions shall be evaluated and considered as applicable:

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

b) actions resulting from any bridge-imposed constraints on differential displacement of linked platforms in all degrees of freedom, taking into account in-phase and out-of-phase wave actions at different wave periods including increased friction over time due to degradation of bearing coatings;

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

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

When active or passive motion compensated temporary bridges or gangways are used for transfer of personnel or goods from floating support vessels to a topsides structure the designer shall use data from the vendor of a representative bridge or gangway to determine the representative values of actions on the topsides during installation, operation, and removal of the bridge or gangway.

8 Strength and resistance of structural components

8.1 Correspondence factor K_c

A correspondence factor, K_c, shall be applied to the partial resistance factors in the national building standard selected for the limit state verification, in order to realize a similar reliability to that implicit in ISO 19902, consistent with 8.2 and 8.3.

Partial resistance factors (γ_R) to be used in the strength and stability equations of the national building standard shall be in accordance with Formula (7):

$$\gamma_R = \frac{\gamma_{R,code}}{K_c} \tag{7}$$

where

$\gamma_{R,code}$ is the partial resistance factor from the selected national building standard;

K_c is the correspondence factor for the selected national building standard.

K_c values in Table 4 shall be used for all code checks of structural components, (e.g. tension, compression, shear, bending, and combined compression and bending) to the national building standards listed.

Table 4 — Values of correspondence factor K_c

National building standard correspondence factor (K_c)			
ISO 19902	ANSI/AISC 360-22[12]	EN 1993-1-1[13]	CSA S16:19[14]
1,00	1,05	0,95[a,b]	1,00
[a] The value of K_c is based on using the values of partial resistance factors γ_{Mi} recommended in EN 1993-2[15], in accordance with the recommendation in EN 1993-1-1[13] for structures not covered by the code, such as offshore platforms;			
[b] The value of K_c listed for EN 1993-1-1 is based on the second-generation Eurocode, i.e. EN 1993-1-1:2022. The local buckling criteria for welded box sections in the first generation code, i.e. EN 1993-1-1:2005 are unconservative, see Reference [16].			

National building standards other than listed in Table 4 shall apply a K_c based on a similar reliability to that implicit in ISO 19902.

A.8.1 provides guidance on K_c calculation if required.

8.2 Design of cylindrical tubular sections

The design of cylindrical tubular sections:

a) should conform to ISO 19902;

b) may conform to the selected national building standard with K_c where applicable.

NOTE For conical transitions and tubular joints, see 8.4.1.

8.3 Design of non-cylindrical sections

8.3.1 Rolled and welded non-circular prismatic members

Design for the strength and stability of rolled and welded non-circular prismatic members shall conform to the selected national building standard with its partial resistance factors modified by the correspondence factor K_c in accordance with 8.1.

8.3.2 Plate girder

Design of plate girders shall conform to a suitable standard or code of practice (see A.8.3.2).

Plate girders shall have web thicknesses of not less than 1,25 % of the web depth or 6 mm, whichever is greater, unless the design is justified otherwise based on the procedures and guidelines in the applicable national building standard.

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.

Stiffened plate girders, designed based on developing tension fields in the web, shall not be used where the following apply:

a) web penetrations exist (unless designed by non-linear FE analysis);

b) the necessary anchorages at the ends of plate girders do not exist (e.g. due to lack of plating continuity through legs).

Plate girders shall be designed for fatigue and fracture at locations where stress concentrations exist, such as abrupt changes in section, penetrations, jacking slots, etc.

Stiffening the web penetrations (coaming) with pipe sleeves or plate should be used where necessary to achieve reduction in stress concentrations and/or sufficient girder strength.

8.3.3 Box girder

Design of box girders shall conform to a suitable standard or 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 components and stressed skin structures

Design of the webs of longitudinally stiffened plate girders and box girders shall conform to an appropriate standard (see A.8.3.4).

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 10.2.1 and A.8.3.4). If the stressed skin structure is exposed to cyclic actions, the possible detrimental effects shall be evaluated.

8.4 Connections

8.4.1 General

Connections shall be designed in accordance with the selected national building standard using the same correspondence factor K_c, except for cylindrical to cylindrical tubular joints and conical transitions, where the provisions of ISO 19902 shall apply.

Connections in the primary structure should be designed to transfer the full strength of the adjoining members unless structural releases are part of the design. Detailing of the joint should avoid brittle failure modes, see ductility requirements in 6.8.2.

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 to minimize, insofar as practicable, stresses due to the contraction of the weld metal and adjacent base metal upon cooling. 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

8.4.3.1 Design and installation requirements

Bolted connections shall be designed in accordance with the national building standard (see A.8.4.3.1) with partial resistance factors modified by the correspondence factor K_c and the additional provisions reported in this clause.

8.4.3.2 Connections classes

Structural connections shall be assessed against consequence of failure in conformance with Table 5.

Table 5 — Connection class - CC

Connection class	Consequence of failure	Examples
CC1	**Catastrophic:** Failure can cause fatalities, environmental damage, or loss of production.	— Primary steel trusses or steelwork. — Primary steel girders and cantilevers. — Lift points. — Escape routes and assembly and/or embarkation areas. — Support steelwork for equipment on the Master Equipment List. — Pre-installed sub-cellar deck assemblies. — Riser supports on the seaward side of the Emergency ShutDown Valve (ESDV).
CC2	**Critical:** Failure can cause major injury, environmental damage, or asset damage.	— Secondary steelwork (stairs and walkways). — Support steelwork for minor equipment support steelwork. — Access platforms. — Towers. — Structural designed supports for hydrocarbon and safety critical piping (e.g., fire water). — Structural designed major supports for services (i.e., piping greater than 12” NB and/or cable trays greater than 300 mm (12 in) nominal width. — Cantilevered walkways (except escape routes). — Retrofitted sub-cellar deck assemblies.
CC3	**Marginal:** Failure can cause injury, environmental damage, or asset damage.	— Supports for services (piping up to 12” NB and/or cable trays up to 300 mm (12 in) nominal width.
CC4	**Minor:** Failure is not serious enough to cause injury, environmental damage, or asset damage, but could result in unscheduled maintenance or repair.	— Temporary steelwork (e.g., temporary bracing that is permanently removed after installation of the topsides by a controlled disassembly operation). — Purpose designed demountable structures that have controlled disassembly and/or assembly operations (e.g., access platforms disassembled to permit maintenance). — Supports for services for field routed, small diameter, non-hydrocarbon, and non-safety critical piping and/or cable trays 150 mm (6 in) nominal width or less. — Non-load bearing structural steelwork.

Connection Class 1 (CC1) - connections in Greenfield or Brownfield projects should be welded. When a bolting is selected, a rigorous procedure shall be followed (e.g. see NORSOK M-101:2011/A1:2022, Annex H[94]).

NOTE 1 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.

NOTE 2 Bolted connections can be extremely simple and cost-effective for the modification of existing topsides. Bolted connections can be preferable to welded connections on production platforms if designed to the national building standard and with pre-load application and corrosion protection in conformance with the provisions below.

SIM for bolted connections CC1 and CC2 shall be prepared.

Connection Class 2 (CC2) - connections in Greenfield projects should be welded.

Connection Class 2 (CC2) - connections in Brownfield projects may be bolted provided connections are assessed for reliability and lifecycle costs on an individual basis.

Connection Class 3 (CC3) - connections in Greenfield or Brownfield projects may be bolted provided connections are assessed for reliability and lifecycle costs on an individual basis

Connection Class 4 (CC4) - connections in Greenfield or Brownfield projects may be bolted.

Bolted connections for CC1 to CC4 shall be high-strength friction grip bolted connections, i.e. EN 1993-1-8 category C or E, or RCSC joint type SC – slip critical or PT – pretensioned, for connections exposed to dynamic or cyclic loading.

The shape and size of the bolt holes shall be in conformance with the following Table 6.

Table 6 — Nominal clearance for bolts and pins (allowable additional hole diameter)

<table>
<tr><th rowspan="2">Type of hole</th><th colspan="9">Nominal bolt or pin diameter d (mm)</th><th colspan="3">Connection class</th></tr>
<tr><th>12</th><th>14</th><th>16</th><th>18</th><th>20</th><th>22</th><th>24</th><th>27 to 36</th><th>39 to 72</th><th>CC2</th><th>CC3</th><th>CC4</th></tr>
<tr><td>Normal round holes [a]</td><td colspan="2">1 [b,c]</td><td colspan="5">2</td><td colspan="2">3</td><td>Allowed</td><td>Allowed</td><td>Allowed</td></tr>
<tr><td>Oversize round holes</td><td colspan="2">3</td><td colspan="4">4</td><td>6</td><td>8</td><td>8</td><td>Not allowed</td><td>Allowed</td><td>Allowed</td></tr>
<tr><td>Short slotted holes [d, e]</td><td colspan="2">4</td><td colspan="4">6</td><td>8</td><td>10</td><td>-</td><td>Not allowed</td><td>Allowed</td><td>Allowed</td></tr>
<tr><td>Long slotted holes [d,e]</td><td colspan="8">1,5d</td><td>-</td><td>Not allowed</td><td>Not allowed</td><td>Allowed</td></tr>
</table>

[a] For applications such as towers and trusses, the nominal clearance for normal round holes shall be reduced by 0,5 mm unless otherwise specified.

[b] For coated fasteners, 1 mm nominal clearance may be increased by the coating thickness of the fasteners.

[c] Bolts with nominal diameters of 12 mm and 14 mm, or countersunk bolts, may also be used in 2 mm clearance holes.

[d] For bolts in slotted holes, the nominal clearances across the width shall be the same as the clearances on diameter specified for normal round holes.

[e] The resistance of bolted connections with oversize round holes or with slotted holes should be reduced as given in EN 1993-1-8:2005, 3.6.

High-strength friction grip bolted connections shall be used whenever the connection is expected to experience significant and sustained fluctuating stresses. All surfaces of high-strength friction grip bolted connections that are in contact after assembly should be surface treated and corrosion protected in conformance with EN 1090-2 or AISC 325.

Bolts in conformance with ISO 898-1 Gr 4.6 or ASTM A307 shall not be used.

Bolt designations with strengths higher than ISO 898-1 Gr. 10.9 or ASTM F3125/F3125M A490 Type 1 shall not be used.

NOTE 3 Bolting materials with actual yield strength exceeding 950 MPa or with hardness exceeding 34 HRC/325 Hv10 can be increasingly subject to hydrogen embrittlement.

NOTE 4 Bolt failures due to hydrogen embrittlement can occur when exposed to corrosion following coating breakdown. Coating breakdown can be as low as 5 years for Hot Dipped Spun Galvanising (HDSG) depending on regional areas, joint configuration (e.g. water ponding), and quality of coating application.

Bolts on floating structures shall be preloaded (see also A.8.4.3.1).

NOTE 5 The intent is to avoid lateral movement and the associated effects of reversal of shear forces on the bolt to minimize fatigue of the bolt and to prevent the nut from loosening. This is particularly applicable to floating facilities where oscillating movements commonly occur. This can decrease the maintenance cost and increase the fatigue life. A.8.4.3.3 gives further guidance.

When bolted connections are used, the following provisions apply:

a) Preloaded bolts should have a minimum effective length of length/diameter > 5 or collars should be added for CC1 and 2;

   NOTE 6 Maximising the effective length of preloaded bolts (between the underside of the head and the nut) will minimize the loss of tension in the bolt due to embedment. Also, the dynamic axial stress is lower in a long bolt than in a short bolt, resulting in a longer fatigue life.

b) a reduction in pretension of 20 % over the life of the bolted connection (20 to 50 years) shall be assumed for bolts due to creep of the protective coating applied to the bolt and bolted surfaces if not taken into account by the national building standard;

c) for repair clamps with a grout annulus or an elastomer liner, additional losses due to long-term creep shall be accounted for in deriving the residual bolt tension;

d) strength check of net section and fatigue check shall be performed on members where bolt holes are applied;

e) structural bolts should be tensioned using one of the following:

   1) turn of nut method pretensioning (see ANSI/AISC 360-22)/combined method (see EN 1090-2);

   2) calibrated wrench method pretensioning (see ANSI/AISC 360-22)/Torque method (see EN 1090-2);

   3) hydraulic tensioning method.

f) bolts shall be re-tightened at a minimum of 40 mins after the initial tightening (to recover loss in pre-tension due to embedment);

   NOTE 7 Direct tension indicating devices/direct tension indicator method (see EN 1993-1-8).

g) the nut should be prevented from loosening due to vibration due to high frequency vibrating equipment or adjacent impact actions (e.g. conductors impacting guides barrels). Methods may include chemical bonding of the nut to the bolt thread, wedge-locking, or other proven methods;

h) washers shall be used beneath the bolt head and the nut to minimize coating damage;

i) bolt hole edges shall be rounded as required for coating application. The inside of bolt holes shall be coated the same as for adjacent surfaces.

Preloading to develop slip-critical connections is an effective way of precluding the effects of reversal of shear forces on the bolt, which can arise from vessel movement. For bolted connections on a floating structure where the inertia from the ship is insignificant compared with the static variable actions, a non-preloaded solution may be adopted.

NOTE 8 An example is bolting of handrail supports where bolts are not influenced by the ship inertia but dominated by the live load on the handrail. It is emphasized that the fatigue life on a non-preloaded connection is less than for a preloaded connection.

Nuts shall be tightened in a non-preloaded connection as given in national building codes, e.g. see EN 1090-2:2018, 8.3[95].

8.4.3.3 Corrosion protection

Corrosion protection of bolted connections shall be assured by the adoption of corrosion resistant alloy or high durability coating or metallizing.

Bolt grades with strengths up to ISO 898-1 Gr 8.8 or ASTM F3125/F3125M A325 type 1, shall be hot dip spun galvanised (HDSG) in conformance to ISO 10684 or ASTM F2329/F2329M.

Bolt grades with strengths greater than ISO 898-1 Gr 8.8 or ASTM F3125/F3125M A325 type 1 and up to ISO 898-1 Gr 10.9 and ASTM F3125/F3125M A490 type 1:

a) should be hot dip spun galvanised (HDSG) in conformance to ASTM F2329/F2329M or ISO 10684, and shall minimise the time to remove surface oxides via acid pickling (see Reference [112]);

b) may be coated with a liquid applied Zn/Al based coating system in conformance with ASTM F1136/F1136M Gr3, ASTM F2833 Gr1 or ISO 10683. After final assembly, accessible parts of the bolts shall be overcoated with an organic coating system to match the surrounding structure.

NOTE 1 Liquid coating typically results in shorter time to coating breakdown than HDSG.

Cadmium-plated bolts shall not be used as they can emit a lethal toxic fume when heated.

SS316 or Super Duplex Stainless Steel (SDSS) bolts may be used for topsides applications but should be solution annealed for external applications.

NOTE 2 Solution annealing provides the optimum resistance to Stress Corrosion Cracking (SCC) for corrosion-resistant alloys (CRAs) in a marine environment. Structural applications typically result in ambient temperature conditions that are typically below solution annealed SCC threshold temperatures. Strain hardened material conditions can lower the threshold temperature for the onset of SCC.

Regular inspection of bolted connections should be specified over the service life.

Structural details where steel is connected to aluminium should be made according to provisions given in 10.5.2.

8.5 Castings and forgings

Castings may be used in place of otherwise complex fabricated components (e.g. padeyes, 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 or forging shall be compatible with the adjacent materials to ensure weldability and avoidance of corrosion. Weldability tests shall be undertaken prior to the casting or forging being manufactured.

The manufacture and testing of castings, including qualification of welding shall be in accordance with EEMUA PUB NO 176, API Spec 2SC, or NORSOK M-122, unless operator requirements indicate otherwise.

The manufacture and testing of forgings, including qualification of welding shall be in accordance with API Spec 2SF, or NORSOK M-123, unless operator requirements indicate otherwise.

8.6 Design for structural stability

Design for structural stability of the components of topsides structure, including the interface with the substructure, shall be in conformance with the national building standard selected for design of the topsides structure (see 5.2.2).

NOTE 1 National building standards typically allow use of alternative methods for design for stability.

The method chosen from the selected national building standard for stability design of the topsides structure:

a) shall be a rigorous 2nd order structural analysis with elastic or inelastic material model and explicitly modelled imperfections when $\alpha_{cr} < 3$ or $B_2 > 1{,}5$, e.g. ANSI/AISC 360-22: Appendix 1, 1.2 (elastic) or 1.3 (inelastic)[12] Direct Method with Member Imperfections (DMMI) or EN 1993-1-1:2022, 7.2.2(7) b)[13] method M4 or EN 1993-1-1:2022, 7.2.2(8) [13] method M5;

b) should be a rigorous 2nd order elastic structural analysis when $3 \le \alpha_{cr} < 10$ or $1{,}1 < B_2 \le 1{,}5$, e.g. ANSI/AISC 360-22, C2[12] Direct Method (DM) (based on a rigorous 2nd order structural analysis); stability design to 8.6 a) or 8.6 c) may be used as an alternative;

c) should be a 1st order elastic structural analysis with 2nd order effects included by amplification of forces and moments when $\alpha_{cr} \ge 10$ or $B_2 < 1{,}1$, e.g. ANSI/AISC 360-22: Appendix 7, 7.2 Effective Length Method (ELM), or EN 1993-1-1:2022, 7.2.2(9)[13] Equivalent Member method (EM). ANSI/AISC 360-22: Appendix 7, 7.3[12] First Order Method (FOM), can also be used and can give results comparable to EM and DM but it is based on a calibration for B_2=1,5 only. Stability design to 8.6 a) or 8.6 b) may be used as an alternative.

where

α_{cr} is the smallest eigenvalue obtained from an eigen bifurcation buckling analysis of the structure when factored actions have been applied (i.e. $\gamma_G G + \gamma_Q Q + \gamma_E E$);

B_2 = Δ_2 / Δ_1 where Δ_2 and Δ_1 are the second order and first order bay deflections (inter-story drift) of the frame respectively.

NOTE 2 Guidance, including values of α_{cr}, is provided in A.8.6 for example offshore structures. Typical offshore structures with braced frames have $\alpha_{cr} \ge 10$ or $B_2 > 1{,}1$ so that 8.6 c) is generally sufficient for the stability design of topsides structure, i.e. amplifying the forces and moments from a 1st order structural analysis (e.g. ANSI/AISC 360-22[12] ELM or EN 1993-1-1:2022 [13] EM). Where the deck or top of jacket has unbraced frames, 8.6 a) or 8.6 b) can apply depending on the magnitude of α_{cr} (or B_2).

Initial imperfections, for system out-of-plumb and member out-of-straightness, shall not be applied in 8.6 b) and c) for offshore structures with lateral actions E.

NOTE 3 If 8.6 a) is used, the eigenvector corresponding to the smallest eigenvalue from a bifurcation buckling analysis can be used to apply the initial imperfections for system out-of-plumb and member out-of-straightness.

Guidance is given in A.8.6.

9 Limit state verification

9.1 Limit state verification approach

The default approach for limit state verification of topsides structure and structural components shall be the semi-probabilistic approach using the partial-factor method in accordance with ISO 19900, as stated by Formula (8):

$$S_d \le R_d \tag{8}$$

where

S_d is the total design action effect (typically the internal stresses) calculated from action combinations using the representative values of the actions, G, Q, E and A, see 7.2, each factored by the appropriate partial action factor (γ_f) from the relevant standard (i.e. ISO 19900, the ISO 19901 series, ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906);

R_d is the design value of the resistance.

R_d is calculated from Formula (9):

$$R_d = \frac{R_r}{\gamma_R} \tag{9}$$

where

R_r is the representative value of resistance;

γ_R is the partial resistance factor from the relevant building standard modified if necessary, by the correspondence factor, see 8.1, or from ISO 19902, see 8.2. Correspondence factors are 1,0 for limit state verification arising from accidental hazards.

9.2 Limit state verification for fire and explosion events

Verification of critical structure exposed to fire or explosion events shall be performed for the following two limit states:

a) the Damage Limitation (DL) limit state, in which the action effects do not exceed the corresponding resistance of structural components;

b) the Near Collapse (NC) limit state, in which structure does not exceed specified deformation limits beyond which it can be considered as collapsed.

NOTE 1 These two limit states are related to severity of loss as follows:

1. DL limit state is the state where one or more components have sustained little or no damage. This can be conservatively approximated as the resistance at which the component strength is based on the representative resistance.

2. NC limit state is strain-based where the critical structure can sustain local damage provided structural collapse causing loss of life and/or major environmental damage does not occur.

Representative values and return periods of accidental actions due to fire and explosion associated to the two limits states DL and NC are given in 9.5.3.

NOTE 2 For design of critical structure to resist explosion actions, the DL limit state typically occurs at the Strength Level Blast (SLB), and the NC limit state typically occurs at the Ductility Level Blast (DLB). Reference [17] provides further information.

NOTE 3 The DL and NC limit states are similar to the ELE and ALE performance levels in ISO 19901-2. These limit states are one and the same for a structure with zero redundancy.

9.3 Approaches for limit state verification for fire and explosion events

The approach to be used for limit state verification:

a) should be the semi-probabilistic approach for design/assessment situations where uncertainty and consequence are within the range typically used to determine the exceedance probability or return period of the accidental actions;

NOTE 1 The Semi-Probabilistic approach for fire and explosion hazards implicitly accounts for uncertainty and consequence. 9.5 provides further detail.

b) may be the Reliability-Based approach for design/assessment situations where the target reliability is determined explicitly;

NOTE 2 The Reliability-Based approach for fire and explosion hazardous events implicitly accounts for consequence and explicitly accounts for uncertainty. A.9.3.2 provides further detail.

c) may be the Risk-Informed approach for design/assessment situations where the risk is determined explicitly.

NOTE 3 The Risk-Informed approach for fire and explosion hazardous events explicitly accounts for uncertainty and consequence. A.9.3.3 provides further detail.

NOTE 4 The three approaches above conform to ISO 2394[1] and ISO 10252.[18] Reliability-Based and Risk-Informed approaches can offer more economical design.

9.4 Risk and risk targets

Topsides critical structure exposed to any of the accidental hazard groups listed in 7.9.1.1 shall be designed to achieve individual and societal risks (IRPA and SRPA) that are tolerable if ALARP.

NOTE 1 Reference [4] provides guidance on the philosophy for managing the risk due to fire and explosion hazards.

NOTE 2 Risk is the product of a given degree of loss with the probability of exceedance of that given degree of loss. The degree of loss can be expressed by repair cost, time to remediate environment, or number of fatalities.

Tolerable-if-ALARP risks (i.e. tolerable life-safety risk, tolerable environmental risk and tolerable business risk for which structure is designed) shall be determined by the more onerous of the operator or regulator requirements.

NOTE 3 For risks where the structure cannot be designed to result in tolerable risks, mitigation by operational measures and emergency response planning can be applied.

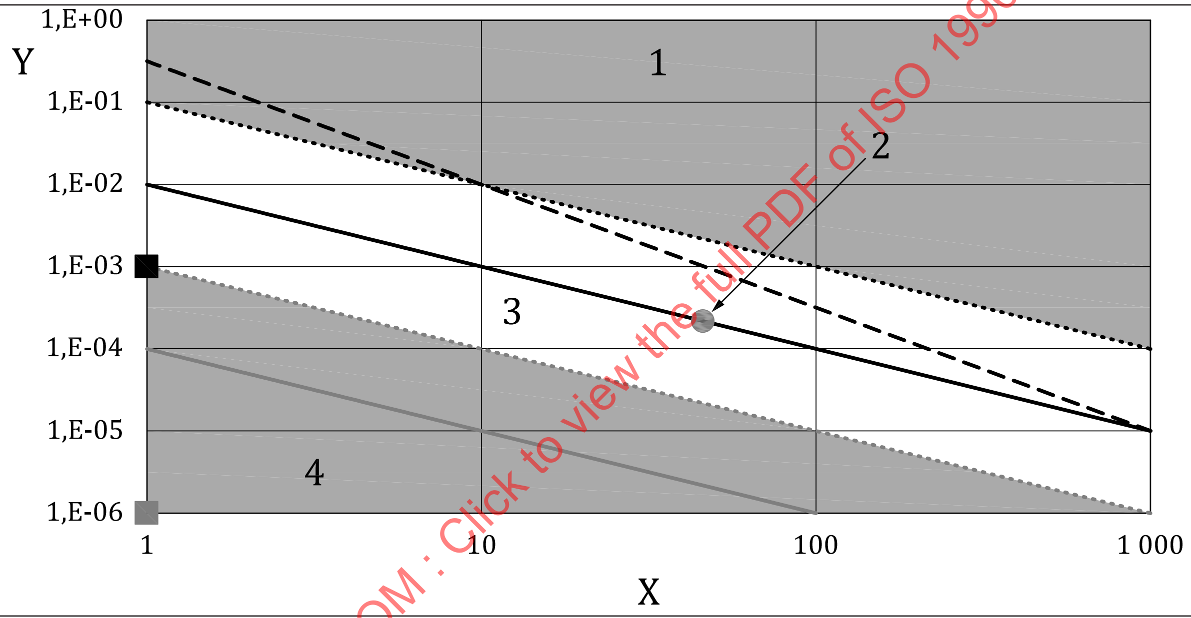
Targets for tolerable-if-ALARP risks shall be determined from the project risk matrix (or risk curve) using the probability of exceedance of the consequences, P(consequences), and the severity of the consequences, S(consequences), as inputs.

NOTE 4 Figure 5 shows an example (not a recommendation) of a project risk matrix with risk targets for a single event and for the summed risk from all hazards. Targets for the risk from single events are typically an order of magnitude less than targets for the summed risk from all hazards (accidental, metocean, seismic, ice and geohazards).

NOTE 5 SRPA (Societal Risk per Annum) also known as group risk, has a maximum tolerable value defined by the more onerous of the operator's or regulator's F-N curve for all life-safety risks on the platform.

NOTE 6 IRPA (Individual Risk Per Annum) is the sum of the life-safety risk probabilities from all hazards that an individual is exposed to while at work, including helicopter travel risk and occupational risk of the individual plus the fraction of the year the individual is on the facility times the risks from all hazards that the facility is exposed to.

	range of annual probability of exceeding the loss	degree of loss (life-safety) resulting from a hazardous event					
		l_1 recordable injury	l_2 lost time injury	l_3 >30 injury	l_4 >2 fatalities	l_5 >10 fatalities	l_6 >50 fatalities
frequent	between 0,25 and 1	tolerable if ALARP	unacceptable	unacceptable	unacceptable	unacceptable	unacceptable
occasional	between 10^{-1} and 0,25	tolerable if ALARP	tolerable if ALARP	unacceptable	unacceptable	unacceptable	unacceptable
infrequent	between 10^{-2} and 10^{-1}	broadly acceptable	tolerable if ALARP	tolerable if ALARP	tolerable if ALARP	unacceptable	unacceptable
rare	between 10^{-3} and 10^{-2}	broadly acceptable	broadly acceptable	tolerable if ALARP	tolerable if ALARP	tolerable if ALARP	unacceptable
very rare	between 10^{-4} and 10^{-3}	broadly acceptable	broadly acceptable	broadly acceptable	broadly acceptable	tolerable if ALARP	tolerable if ALARP
improbable	between 10^{-5} and 10^{-4}	broadly acceptable	broadly acceptable	broadly acceptable	broadly acceptable	broadly acceptable	tolerable if ALARP



Key

X degree of loss (N fatalities)

Y annual probability of exceeding a given degree of loss

1 unacceptable risk region

2 R2P2 point (reference point used by UK HSE, Reference [91] reducing risk protecting people])

3 tolerable if ALARP region (between shaded regions)

4 broadly acceptable risk region

- - - SRPA onsite unacceptable (inc risk aversion)

.... SRPA onsite unacceptable

.... SRPA onsite broadly acceptable

■ IRPA unacceptable

—— SRPA offsite unacceptable

—— SRPA offsite broadly acceptable

■ IRPA broadly acceptable

Figure 5 — Example risk matrix and example risk curve (based on UK HSE terminology and limits)

Tolerable-if-ALARP group risk from a single hazard type, as listed in 7.9.1.1, at original design should be an order of magnitude less than tolerable-if-ALARP group risk summed for all hazard types (i.e. the risk from all hazard types listed in 7.9.1.1 plus the risk from all environmental hazards).

If the risk is unacceptable, critical structure shall be redesigned. Alternatively, the hazard intensity, for a given probability of exceedance, shall be reduced.

Risk treatment should be applied by design iterations where the risk is "tolerable-if-ALARP" to further reduce the risk or avoid the risk where practicable.

When the risk is in the tolerable-if-ALARP region (see Figure 5), ALARP shall be achieved when the level of risk is reduced to the point where the economics of further reduction measures become grossly disproportionate to the additional risk reduction obtained.

NOTE 7 The UK HSE defines a mitigation as grossly disproportionate if the implementation cost is greater than 6 to 10 times the resulting cost benefit (see Reference [109]).

Critical structure design should be adopted if the risk is "broadly acceptable".

Robustness shall be incorporated in the design of critical structure to ensure the life-safety risk is broadly acceptable or tolerable-if-ALARP for situations where the structure, damaged by a controllable hazardous event, is subsequently exposed to environmental hazardous events in the period prior to repair.

9.5 Limit state verification for fire and explosion events by semi-probabilistic approach

9.5.1 DL limit state verification

The semi-probabilistic approach for verification of the DL limit state shall use Formula (10):

$$S_d \leq R_d \tag{10}$$

where

S_d is the total design action effect (typically the internal stresses) calculated from action combinations using representative values of the actions (NTE values of the actions G and Q, combined with A);

A is the representative value of the accidental action with a probability of exceedance or return period, see 9.5.3;

R_d is the representative resistance based on the yield stress for strength or buckling of the structural component (i.e., only localised regions can have stress above material yield).

Partial action factors, partial resistance factors, and correspondence factors shall be set to 1,0 in Formula (10).

NOTE 2 Verification of the DL limit state demonstrates that the business risk due to damage of critical structure is tolerable, that typically requires little or no repair costs when the structural component is exposed to SLB loading.

9.5.2 NC limit state verification

The semi-probabilistic approach for verification of the NC limit state shall use Formula (10) but with the resistance defined as follows:

R_d is the design value of the resistance at the specified deformation limit (strain limits as defined in Reference [19] with $\gamma_M = 1{,}0$).

Partial action factors, partial resistance factors, and correspondence factors shall be set to 1,0 in Formula (10).

NOTE 1 The strain increases rapidly as the NC limit state is approached, thus the action magnitude is not sensitive to uncertainty in fracture strain and γ_M =1,0 applies.

NOTE 2 A conservative strain limit of 5 % can be used in lieu of the strain limits in Reference [19].

NOTE 3 Verification of the NC limit state (similar to abnormal/accidental limit states) demonstrates that the life-safety risk due to collapse of critical structure is tolerable-if-ALARP, and typically requires no escalation of the blast or fire event when the structural component is exposed to DLB loading.

9.5.3 Representative values of accidental actions

The representative value of accidental actions for use in Formula (10) for the verification of the DL limit state (i.e. the SLB overpressure) shall have an annual probability of exceedance no greater than 10^{-2}.

The representative value for SLB action may be based on a less frequent event for the verification of the DL limit state, based on the business disruption risk. e.g. an overpressure of either a) overpressure with annual probability of exceedance of 10^{-3} or b) overpressure of DLB/3 (i.e. if reasonable practicable and cost effective, design for a less frequent event results in reduced business risk).

The representative value of accidental actions for use in Formula (10) for the verification of the NC limit state (i.e. the DLB overpressure) shall have an annual probability of exceedance no greater than 10^{-4}.

The representative value for DLB action may be based on a less frequent event for the verification of the NC limit state, based on the life-safety risk for catastrophic events where life-safety by evacuation cannot be demonstrated. e.g. overpressure with annual probability of 10^{-5} (i.e., if reasonable practicable and cost effective, design for a less frequent event results in reduced safety).

10 Structural systems

10.1 Topsides design

10.1.1 General

The topsides shall be investigated for all appropriate design/assessment situations and load cases. Permanent and operational 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 be based on criteria and assumptions which are consistent with those of the substructure to ensure equilibrium. Any consistent disposition of forces can be considered provided that both topsides and substructure have sufficient capacity for the internal forces. Various approaches are acceptable. For example, it is possible to model the sequence of construction stages in successive design/assessment situations to take account of locked-in stresses and changes in structural configuration. However, it is acceptable to use a linear analysis of the topsides including environmental forces by simulating fixed boundaries provided that the reaction forces at the substructure interface are consistently applied to the substructure.

10.1.2 Topsides on concrete substructures

In analysis, particular attention shall be paid to the interaction between steel topsides and concrete substructures (see 7.6.1.2). 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 substructure 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 substructure designers.

Attention should be given to the response of the integrated platform to wave actions, and ice actions if applicable. 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 substructure 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.

10.1.3 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 evaluated and taken into account in the design of supports. The use of sliding or elastomeric bearings at the topsides/hull interface can be required.

10.1.4 Equipment supports

Equipment supports that are subject to uplift shall be mounted directly on the supporting steelwork and not on deck plate unless the plate and the associated welds are designed accordingly.

Equipment should not be mounted directly on grating.

10.2 Topsides structure design models

10.2.1 General

Internal forces in structural components shall be derived using an indeterminate, three-dimensional structural analysis methodology.

Design/assessment situations shall be established for each phase of the design life and for each stage of construction (see ISO 19900).

Limit state verification of each design/assessment situation shall be performed by either of the following methods:

a) linear elastic analysis using manual calculation or analysis models (see ISO 19902:2020, cl. 12.3);

b) a method allowing redistribution using manual calculation or analysis models.

NOTE 1 Manual calculation can be an effective method to demonstrate equilibrium on simple systems.

NOTE 2 Method b) can result in different magnitude of member forces but typically results in a more efficient and lighter structure.

NOTE 3 Nonlinear analysis can be used for method b).

The structural analysis model for method b) may be configured with any or all of the following simplifications:

— neglect any plate panels axial stiffness (typically called stressed skin);

— assume an ideal hinge even in a stiff connection;

— use modified cross-section on a beam or plate panel properties (thickness of plate or stiffeners);

— neglect the increased stiffness in the joints;

— neglect effect of doors or windows on a panel when determining the internal forces;

— neglect web opening in a beam when determining the internal forces;

— neglect the sequence of prior stages of construction, including neglecting any effects of modules being installed at different times with different stiffness.

NOTE 4 In method b) the redistribution of forces and strains are determined for the design situations as opposed to simply adding the results of the forces and strains in the prior situations or in any prior sequential situations.

NOTE 5 These simplifications are based on there being adequate ductility of material and structural components, and on the substructure having sufficient capacity for internal forces to be redistributed. It is therefore not necessary to consider the effects of substructure stiffness on the distribution of internal forces within the topsides structure. For each design/assessment situation, total actions are applied to the actual structural configuration for that design/assessment situation disregarding the history of stress or strain from previous design situations. See A.10.2.1 for more guidance.

Verification of the design shall, however, take account of effects arising from the simplifications above. For example, reduction in component capacity due to web openings shall be evaluated even if internal forces are determined without modelling this opening.

Relevant assumptions made when determining the internal forces shall be applied when the design of the structural components is verified. For example, applying an ideal hinge between a beam and a column will influence the buckling length as no moment fixation can be included in the design verification.

The above simplifications are not applicable for design for fatigue that shall be in accordance with 6.7

Connections shall be designed to be ductile, avoiding brittle failure modes, regardless of any approaches, type of analyses and assumptions, see 6.8.

NOTE 6 Applying the simplifications listed above does not impact the ductility requirement in 8.4.1 or make them stricter, and not using the simplifications will not ease the ductility requirements.

The allowable simplifications are further discussed in A.10.2.1.

10.2.2 Substructure model for topsides design

The substructure shall be modelled for the topsides structure design in one of the following two options:

a) in same manner as used for modelling and analysis of the substructure itself;

b) simplified by applying boundary conditions to represent the substructure (for instance fixed boundaries).

If option b) is selected, reaction forces from the topsides analysis shall be used as the basis for verifying substructure capacity.

Regardless of which option is selected above, on a floating structure the effect of hogging and sagging shall be evaluated.

10.2.3 Topsides model for topsides design

The topsides structure may be modelled as more than one independent structure.

Topsides structure shall be modelled and analysed using one of the following two options:

a) the actual sequence of construction, for which the interaction between structures and differential deflections are assessed;

b) neglecting construction sequence, see 10.2.1.

Where the model is used to represent pre-service conditions (fabrication, loadout, transportation, and installation), appropriate boundary conditions and consistent forces across the interface shall be applied to represent the stiffness of the supports. For loadout, transportation, and installation conditions, appropriate accelerations and displacements shall be applied.

Eccentricities in primary steel joints shall be modelled to account for shear resisted by the chord web between the adjacent braces and/or columns.

10.2.4 Modelling for design of equipment and piping supports

Interconnecting pipework between equipment or between modules and its supports shall be checked for the effects of interaction.

NOTE 1 Relative displacements and deflections between pieces of equipment or between modules could affect the integrity of the interconnecting pipework.

Piping bridging between modules shall also be checked for deflections due to environmental, accidental and seismic actions.

NOTE 2 Local supports for piping and individual items of equipment can generally be analysed in isolation.

Equipment skids or packages shall be modelled such that the mass is lumped at the local vertical CoG for conditions subject to lateral accelerations.

NOTE 3 For explosion analysis of piping systems, integrated modelling approach can be used to reduce the conservatism due to decoupled analysis (see Reference [20]).

Dummy members shall not add stiffness or mass to the model and it shall be documented that they work as intended by verifying that the forces are neglectable.

10.3 Substructure interface

10.3.1 Responsibility

All assumptions at the interface between the topsides and substructures shall be documented to ensure that the designs are not compromised by any differences in assumptions. Forces (and moments) used for design each side of the interface shall be consistent. This can be achieved by modelling the boundary for the topsides at the interface and ensuring that the reaction forces are then applied for the substructure analysis. Alternatively, the substructure itself may be modelled for determining the internal forces in the topsides. If this alternative is selected, the forces (and moments) at the interface shall be consistent with the forces for which the substructure is designed.

10.3.2 Strength design

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

10.3.3 Fatigue design

A fatigue analysis should be undertaken where the internal forces in the connection between the topsides structure and the substructure vary. The structural models to be used for fatigue analyses shall reflect all stiffnesses at their expected values. Stiffnesses from non-structural components (e.g. piping, cladding) should be included in the model.

For floating and compliant substructures, the fatigue analysis shall include the effects of the distortions of the substructure, the stiffnesses of the supporting points and the changes in the direction of actions due to the motions and deformations of the substructure.

10.4 Flare towers, booms, vents and similar structure

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;

— effects of icing; and

— the indirect effects of metocean and ice actions on the substructure.

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 structure. Guidance on such structures is given in A.10.4. See also 6.5.2.4.

Snow and ice actions shall be combined with actions generated by wind.

NOTE 1 The static and dynamic behaviour of such structures can be substantially influenced by accumulations of snow and ice. Both wind actions and operational actions can be increased by such accumulations.

NOTE 2 Return period of wind actions is typically 10 years when combined with maximum snow and ice actions.

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 support of the access platform at the top of the flare tower/boom shall account for thermal expansion of the platform relative to the primary steelwork of the flare tower/boom.

Flare tower/boom design shall account for the effects of the flare header filling with liquid.

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

10.5 Helicopter landing facilities (helidecks)

10.5.1 General

Regarding the design, including helideck layout and plan dimensions, users shall be aware of the requirements of the regulatory 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 to accommodate the design helicopter. The design helicopter is that with the greatest size, weight 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.

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 evaluated (see A.10.5.1).

In this document the term MTOW (maximum take-off weight) is equivalent to the term MTOM (maximum take-off mass) as used in some aviation standards. The operator may require a weight reserve on the MTOW to allow for changes in the design helicopter weight.

10.5.2 Construction

The helideck and its supporting structure may be fabricated from steel, sea water resistant 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 (see 12.5.3).

10.5.3 Helideck design verification

10.5.3.1 Design/assessment situations

Design/assessment situations, see 6.2, shall be established for the following conditions:

— helicopter landings (emergency and heavy normal landings) (see 10.5.3.3);

— a helicopter at-rest (see 10.5.3.4);

— empty deck (see 10.5.3.5);

— post-damage (see 10.5.3.6).

Design/assessment situations shall include helicopter actions as appropriate, applied together with suitable combinations of permanent, operational and environmental actions.

The design of the structure shall be verified for the limit states relevant to each design/assessment situation.

10.5.3.2 Design requirements

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.

Helicopter skid or wheel locations shall be positioned on the deck to maximize the internal forces in the component being designed.

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

Slender structural elements forming part of the supporting structure shall be checked for wind induced vibrations (see 7.4).

10.5.3.3 Actions for helicopter landings

The action values and multipliers in this subclause are specified to obtain representative values of the actions.

The following actions shall be included in design/assessment situations for helicopter landing situations:

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

The dynamic helicopter landing action for accidental events (emergency landing) shall be taken as 2,5 x MTOW.

The dynamic helicopter landing action for an operational incident (heavy normal landing) shall be taken as 1,5 x MTOW.

The action shall be increased by a structural response factor to account for the dynamic 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) **Permanent and operational 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 for application as permanent actions.

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

Concentrated horizontal imposed actions equivalent in total to half the MTOW of the design 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 evaluated.

c) **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 evaluated.

The wind speed shall be taken as:

— accidental event (emergency landing): 1-year return period wind speed or the speed used for normal landing if higher than the 1-year return period wind;

— operational incident (heavy normal landing): wind speed restricting normal (non-emergency) helicopter operations at the platform (i.e. 31 m/s[21]).

Vertical (up or down) action on the helideck structure due to the passage of wind over and under the helideck shall be evaluated.

Only horizontal wind actions during landing shall be applied for topsides on floating structures.

2) **Snow and ice**

In the absence of more onerous site-specific data during landing, actions due to snow and ice weight may be assumed to be within the 0,5 kN/m^2 general area-distributed action specified in 10.5.3.3 b). Snow and ice weight shall be excluded in wind uplift situations.

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

The effect of accelerations and dynamic amplification arising from the predicted motions of the fixed or floating platform shall be evaluated and the actions arising shall be included for the following conditions:

— accidental event (emergency landing): a storm condition with a 1-year return period;

— operational incident (heavy normal landing): a storm condition with a 1-year return period. If restrictions on the motions of the helideck are implemented for helicopter landing, these may be used. Platform accelerations induce longitudinal (ax), transverse (ay), and vertical (az) inertial actions acting on the centre-of-gravity of the helideck based on the motions of the platform under the relevant environmental conditions. The accelerations for floating facilities shall in addition take into account the effect of roll, pitch, and yaw.

10.5.3.4 Actions for helicopter-at-rest

The action values and multipliers in this subclause are specified so as to obtain representative values of the actions.

The following actions shall be included in design/assessment situations with the design helicopter-at-rest.

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

All parts of the helideck accessible to helicopters, including any separate parking or run-off area, shall be designed to support an operational action equal to the MTOW of the design helicopter at any location. This shall be distributed at the undercarriage locations in proportion to the position of the centre of gravity of the design helicopter, taking account of possible different positions and orientations of the design 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 operational action of 2,0 kN/m^2 shall be included.

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

Self-weight of helideck structure and fixed appurtenances shall be included.

d) **Environmental actions**

Environmental actions shall be applied in accordance with the relevant design/assessment situation, including the actions from the helicopter.

1) **Wind actions on the helideck**

Vertical (up and down) actions on the helideck due to the passage of wind over and under the helideck shall be evaluated.

Only horizontal wind actions during landing shall be applied for topsides on floating structures.

2) **Snow and ice**

Snow and ice, if relevant, shall be taken into account for helideck primary and secondary structures. Snow and ice weight shall be excluded in wind uplift situations. Snow and ice actions may be excluded for the landing area of heated helidecks.

3) **Green water actions**

Green water actions, if relevant, shall be included in relevant design/assessment situations according to likelihood.

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

The effect of accelerations and dynamic amplification arising from the predicted motions of the fixed or floating platform shall be evaluated and the actions arising shall be included.

Platform accelerations induce longitudinal (ax), transverse (ay), and vertical (az) inertial actions acting on the centre-of-gravity of the helideck based on the motions of the platform under the relevant environmental conditions. The accelerations for floating facilities shall in addition take into account the effect of roll, pitch, and yaw.

If it is likely a helicopter can be present at rest during abnormal environmental conditions (e.g. helicopters are normally stored at the facility), an abnormal design/assessment situation shall be included. Otherwise, the abnormal design/assessment situation is addressed with empty deck.

10.5.3.5 Actions for empty helideck

The actions specified in 10.5.3.4 b), c), d) and e) shall be included in design/assessment situations with an empty helideck.

Empty deck situations shall include checking the helideck structure for maximum uplift forces.

10.5.3.6 Actions for post-damage situations

In the design phase, design/assessment situations shall be established for damage scenarios specified by the operator. The actions to be included shall be consistent with the damage scenarios and can be with or without a helicopter.

For assessment after an emergency landing, design/assessment situations shall be established to include the post-damage structural condition and configuration. See also 10.5.4 regarding minor local deformations.

Depending on the damage scenario and the operator's post-damage plans, design/assessment situations with and without a helicopter can be established. Damage scenarios which include a helicopter should include both at rest and heavy normal landing conditions for a helicopter which may be lighter or smaller than the design helicopter.

In addition to permanent and helicopter gravity actions, the following actions should be included in post-damage design/assessment situations:

— general area imposed operational action for helicopter landing or at rest;

— design environmental actions for the short duration;

— inertial actions due to platform motions for the short duration.

10.5.3.7 Combinations of actions

As a minimum, load cases with combinations of actions shall be applied for design/assessment situations in accordance with Table 7 for landings, at-rest, and empty deck conditions.

The representative actions of each type shall be factored and combined in accordance with the relevant standard used for the substructure (i.e. ISO 19902, ISO 19903, ISO 19904-1, ISO 19905-1, ISO 19905-3 and ISO 19906).

Table 7 — Combinations of actions

Actions	Heavy normal landing	Emergency landing	At rest			Empty deck	
Design/assessment situation category	Operational	Accidental	Operational	Extreme	Abnormal See 10.5.3.4	Extreme	Abnormal
Helicopter	Heavy normal	Emergency	At rest	At rest	At rest or none	None	None
Vertical helicopter actions	1,5x 1,3x MTOW	2,5x 1,3x MTOW	MTOW	MTOW	MTOW	NA	NA
Horizontal helicopter actions	0,5x MTOW	0,5x MTOW	NA	NA	NA	NA	NA
Inertial actions a_x, a_y, a_z	See 10.5.3.3 d)	See 10.5.3.3 d)	See 10.5.3.4 e)	See 10.5.3.4 e)	See 10.5.3.4 e)	See 10.5.3.5	See 10.5.3.5
Permanent actions	X	X	X	X	X	X	X
X = actions to be included							
NA = actions are not applicable.							

Table 7 *(continued)*

Actions	Heavy normal landing	Emergency landing	At rest			Empty deck	
Area imposed actions on landing platform	0,5 kN/m^2	0,5 kN/m^2	2,0 kN/m^2	2,0 kN/m^2	0,5 kN/m^2	2,0	2,0
Operational actions on walkways and staircases	NA	NA	X	NA	NA	NA	NA
Environmental actions	See 10.5.3.3 c)	See 10.5.3.3 c)	See 10.5.3.4 d)	See 10.5.3.4 d)	See 10.5.3.4 d)	See 10.5.3.5	See 10.5.3.5
X = actions to be included NA = actions are not applicable.							

Combinations of actions based on Table 7 shall be developed for the damage scenarios defined by the operator, see 10.5.3.6, and applied in short duration (post-damage) design/assessment situations.

Combinations of actions relevant to design verification for serviceability shall also be applied in serviceability design/assessment situations.

10.5.3.8 Strengths and resistance of structural components

Strength and resistance of structural components shall be in accordance with Clause 8 for steel helidecks and with 11.4 and A.11.4 for aluminium materials.

10.5.3.9 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 125 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.

10.5.3.10 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 design helicopter in the relevant design/assessment situations (see 10.5.3.4).

They shall take into account the inertial forces resulting from any movement of the platform (see 10.5.3.4).

Advice on recommended safe working load requirements for tie-down points for specific helicopter types should be obtained from the helicopter operator or manufacturer.

10.5.4 Reassessment of existing helidecks

Alternative requirements and criteria to those specified in 10.5.1 to 10.5.3 for the type of helicopter 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.

An updated range of potential landing conditions shall be identified and evaluated.

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 helicopter 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.

10.6 Crane support structure and crane boom rest

10.6.1 General

The crane support structure comprises the crane pedestal and its connections to the topsides structure. It does not include the slew ring or its equivalent, or the connections between the slew ring and the pedestal or the pedestal adaptor (transition between the slew ring and the pedestal).

Design requirements for the crane boom rest are given in 10.6.8.

10.6.2 Design requirements

Design of offshore crane support structure shall be in accordance with either the crane specifications API Spec 2C or EN 13852-1, except where specifically noted otherwise in 10.6.

Material requirements are given in Clause 11.

API Spec 2C or EN 13852-1 are mutually exclusive and shall not be interchanged at any stage with the other.

NOTE Where API Spec 2C or EN 13852-1 are referenced in the following, the requirement relates specifically to that document; otherwise, the requirement applies either to API Spec 2C or EN 13852-1.

The crane support structure reactions shall be included in the calculation of design actions, as defined in 7.2 for the verification of the topsides structure in each of the design/assessment situations described in 10.6.3.2 and using the appropriate action types as reported in 10.6.3.1.

The crane support structures shall be included in the analytical model of the primary structure as their stiffness can have a significant effect on load distribution.

Crane support structure shall:

a) either (preferred method) be attached at an intersection of topsides structure primary framing with minimal eccentricity, and be connected to a minimum of two main deck elevations, or

b) where this is not practicable, be adequately reinforced to have sufficient strength and stiffness to meet the provisions of the crane specification and of 10.6 (see also A.10.6.2).

The maximum deflection at the top of the crane support structure (typically the pedestal) should not exceed the values shown in Table 1 for the cantilever.

The designer should consult the crane manufacturer on designing the crane pedestal to minimize excessive crane motion during operations within the serviceability design limits (6.5.2.2) related to the operational limits and to the personnel comfort.

NOTE Additional stiffness can be required to prevent excessive motion of the crane and operator. Excessive motion can cause the operator discomfort even if the strength requirements are satisfied. See for further guidance A.10.6.2.

Specific requirements are given below, as follows:

— static design, 10.6.3;

— fatigue design, 10.6.4;

— seismic/earthquake design, 10.6.5;

— dynamic design, 10.6.6;

— fabrication, 10.6.7.

10.6.3 Static design

10.6.3.1 Design actions types

Actions on the crane support structure shall be categorized in accordance with 7.2.

NOTE Relevant operational actions are lifted loads and associated offlead and sidelead actions.

Design actions are then calculated using the equations in 7.3.

The designer should interface with the crane manufacturer to ensure the various crane actions are communicated in such a manner that they can be assigned to the correct action type.

10.6.3.2 Design/assessment situations and actions

The following design/assessment situations shall be assessed:

a) operational design/assessment situations: in-place operating conditions (10.6.3.3);

b) extreme design/assessment situations:

    1) in-place out-of-service conditions (10.6.3.4);

    2) seismic/earthquake ELE events (10.6.5);

c) abnormal design/assessment situations:

    1) seismic/earthquake ALE events (10.6.5);

    2) abnormal metocean events;

d) accidental design/assessment situations: crane collapse failure/gross-overload conditions (10.6.3.5);

e) serviceability design/assessment situations: see deflection limits in 10.6.2 and 6.5.3.

For each design/assessment situation the following conditions shall be evaluated:

a) maximum overturning moment with corresponding vertical action plus a horizontal action, as described below, applied simultaneously to the boom head sheave;

b) maximum vertical action with corresponding overturning moment plus a horizontal action, as described below, applied simultaneously to the boom head sheave.

A dynamic coefficient shall be applied to the vertical action referenced in a) and b) as follows:

1) for API Spec 2C, C_v, together with a pedestal factor;

2) for EN 13852-1, Φ_n.

The vertical dynamic coefficient, C_v or Φ_n (as applicable) shall be applied to SWL.

Both the aforementioned vertical and horizontal actions shall be computed as described in the crane specification, including any applicable supply boat and/or crane motions.

The representative (unfactored) actions to be imposed on the crane support structure should-be provided by manufacturer.

Further, the pedestal structural engineer shall ensure that the information provided by the crane manufacturer for crane support structure design includes the effects of the dynamic coefficient, C_v, as well as of the pedestal factor, PF, as described in API Spec 2C or the dynamic coefficient Φ_n for EN 13852-1.

In case of a large boom (as defined in 10.6.4), even in static design analysis the effects of the vertical dynamic coefficients (C_V or Φ_n) shall be applied to the boom weight.

In absence of information from the crane manufacturer, the preliminary crane support structure reactions should include:

f) the combined moment calculated based on the largest combination of target SWL times radius with a load factor in the range 3,9 – 5,2;

NOTE 1 The range covers a C_v or Φ_n of 1,5 to 2,0; higher factors are typically seen for offboard lifting in higher seastates and/or from floating structures. The minimum safety factor for any crane component over the safe working load times the radius is assumed to be 2,0.

g) the axial thrust calculated based on 10 % of the combined moment, disregarding the units.

The above reactions are only to be used for preliminary analysis. This will perhaps result in a rough order of magnitude to the final crane support structure design.

NOTE 2 For robust cranes with a higher factor of safety to collapse, the preliminary factors can be unconservative.

10.6.3.3 In-place operating conditions

Design/assessment situations for in-place operating conditions involve actions due to onboard and offboard lifts along with the associated operating condition parameters and the applicable environmental actions (wind, ice, and/or seismic/earthquake actions).

The design situations to be analysed shall include, in accordance with 7.3:

a) In-place situations with permanent and variable actions only (7.3.1);

b) In-place situations with operating environmental actions (7.3.2).

In-place situations shall include the full operating cycle from picking up the load, slewing and set-down including transient operating situations and the related actions due to accelerating or decelerating phases during starts and stops.

If the crane pedestal is used for storage, actions and internal corrosion protection shall be based on the nature of the liquid to be stored and the maximum levels and pressures under normal operating, test or failure of level control conditions.

NOTE Under certain circumstances, it could be advantageous to utilize crane pedestals for diesel or water storage.

The in-place actions shall be included in the in-place design action evaluation with the in-place partial action factors as described in 7.3.

10.6.3.4 In-place out-of-service conditions

Design/assessment situations of the pedestal shall include the crane in stowed condition and the applicable environmental actions (wind, ice and/or seismic/earthquake actions).

The out-of-service actions shall be included in the in-place design action evaluation with the proper in-place partial action factors for extreme and abnormal metocean actions as described in 7.3.3.

NOTE 1 The out-of-service actions affect the design of the crane boom rest structure.

The vertical reaction force shall be calculated based on the weight of the crane boom (including any added ice and vertical environmental effects) multiplied by a minimum 1,1 uncertainty factor.

The horizontal reaction force shall be calculated based on the maximum of:

a) the horizontal component of the crane boom weight (including any added ice and horizontal environmental effects) multiplied by a minimum 1,1 uncertainty factor.

b) the wind actions provided in ISO 19901-1 (including any horizontal environmental effects).

NOTE 2 The horizontal component of the crane boom weight (including any added ice load reactions) is based on the tilt of the crane support structure.

10.6.3.5 Accidental design/assessment situations – Gross overload condition/Crane collapse failure condition

Design/assessment situations for gross overload/crane collapse conditions are intended to cover unexpected events such as the crane hooking a supply boat, as described in the crane specification.

NOTE This situation is typically 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), the crane support structure is not damaged, and progressive collapse is prevented.

The crane manufacturer’s failure curves, for all crane conditions, shall be used to assess the overload conditions in accordance with the crane specification.

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 design action type shall be Q_2 as defined in 7.2.

The partial action and resistance factors shall be set to 1,0 as for accidental situations (see Clause 9).

10.6.4 Fatigue design

The crane support structure shall be checked for resistance to the crane foundation fatigue actions in compliance with ISO 19902 and the applicable S-N curves therein, as well as with the partial damage design factors provided in 6.7.

The fatigue design shall be in accordance with the crane specification.

Fatigue should be based on:

a) if available, a realistic load spectrum, including a full rotation of 360° around the pedestal flange, with a complete reversal of action occurring on upper and lower faces of the flange during each rotation.

   NOTE 1 The spectrum factor for actions is typically provided by the crane manufacturer and this is typically dependent on the specification for the design of the crane.

   NOTE 2 The number of expected crane rotations is typically determined from the expected operational frequency and intended life of the crane.

b) if not available, a minimum of 1,000,000 cycles under the following conditions (see A.10.6.4):

   1) a load of 2/3 SWL (not factored by C_v and PF or Φ_n) at the boom position and crane orientation producing maximum stress in each component of the crane support structure.

      NOTE 3 The boom weight or the effect of any accelerations is not typically added to the requirement of 0,66 of SWL, since this load level encompasses these considerations in the context of fatigue assessment.

      NOTE 4 The assumptions in point b) are typically unconservative for large boom cranes. Fatigue cracks have been observed in pedestal butt welds in some large boom cranes checked according to the assumptions in point b).

2) the bending stress due to large boom cranes combined with the associated stress produced by 2/3 of SWL.

NOTE 5 Typically, a boom crane is defined as large when the moment due to boom weight is greater than 30 % of the design moment.

The stress range used in the above point b) approach should be the difference between the stress caused by the above loading and the stress calculated with the boom in the same position but unloaded.

When a dynamic analysis is required the associated dynamic effects should be included in the fatigue life calculations for the pedestal structure.

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

Off-lead and side-lead actions, wind and other environmental actions are typically insignificant with respect to the fatigue analysis. Nevertheless, this shall be assessed on a case-by-case basis.

In case of floating structures special attention shall be given to any environmental action effects on the structure that can cause fatigue damage to the crane support structures.

NOTE 6 Fatigue damage from waves and operation are independent processes, and damage can therefore be added directly. In many cases, the topside is transported over long distances, which causes dynamic stresses from accelerations and consequently contributes to fatigue damage.

10.6.5 Seismic/Earthquake design

Design/assessment situations for seismic events and associated seismic design actions shall be in accordance with 7.7 and the additional guidelines of 7.7.2 and 7.7.3.

Owing to the very low probability of simultaneous occurrence of a design seismic event at the time of the crane being used for a maximum rated lift, design/assessment situations to be analysed may include a reduced crane load acting simultaneously with the design seismic event:

a) if a crane study to identify typical offloading loads that regularly occur during the life of the platform is performed:

   1) a load equal to 90 % non-exceedance, not less than 1/3 of the rated capacity.

b) in absence of such a study:

   1) a load producing 2/3 of the rated crane overturning moment capacity.

The design/assessment situations for seismic events shall include also:

c) the no-hook load case;

   NOTE Such analyses help identify components governed by uplift.

d) the boom in its stowed condition load case.

10.6.6 Dynamic design

Where the dynamic response of the crane-pedestal system is independent of the dynamic response of the deck supporting structure, the dynamic response of the crane-pedestal system may be determined by applying the stiffness of the deck at the pedestal connection points as a boundary condition.

The dynamic response of the crane-pedestal system shall be calculated by a coupled dynamic analysis when both of the following apply:

a) the mass of the crane-pedestal system is greater than 5 % of the total topsides mass;

b) the natural frequency of the crane-pedestal system is greater than 50 % of the main topsides lateral natural frequency.

10.6.7 Fabrication

Special attention should be given to the fabrication tolerances and surface finishing required for correct mechanical coupling of the interfaced structures as described in A.10.6.7.

10.6.8 Crane boom rest design

Design/assessment situations for the crane boom rest structure shall be established in accordance with 6.2.

Actions shall include vertical and horizontal reaction forces from the crane boom, calculated based on:

a) the weight of the crane boom:

   1) including a weight allowance reflecting the uncertainty in the weight of the crane boom if not yet determined, and including any added ice accretion weight and any inertial accelerations.

b) impact forces – the load factors used (for both preliminary and final design) in order to calculate representative values of the actions shall be*:*

   1) 2,5 factor for the vertical reaction force (comprising factors of 1,0 + 1,5).

   2) 1,5 factor for the horizontal reaction force (comprising factors of 1,0 + 0,5).

   NOTE 1 The 1,0 factor accounts for the reactions calculated at a) 1) above. The 1,5 and 0,5 factors account for the impact forces of the boom landing in the rest.

   NOTE 2 The horizontal component of the crane boom weight (including any added ice accretion weight reactions) is based on the tilt of the crane support structure.

c) the wind actions applicable for the design/assessment situation.

The vertical and horizontal reaction forces shall be taken as acting simultaneously on the boom rest structure.

In absence of information from the crane manufacturer, the preliminary crane boom weight may be assumed as 20 % of the total crane weight and used as the input to 10.6.8 a).

NOTE 3 The preliminary crane boom weight is only to be used for preliminary analysis. This normally provides a reasonable approximation to the weight to be used for the final boom rest design.

10.7 Derrick equipment set

The DES moves between well slots. Typically, the skid-base moves in only one direction while the derrick substructure and derrick move at 90 degrees to the skid-base. Design/assessment situations shall be established for all possible positions of the DES.

NOTE 1 The DES comprises the derrick, derrick substructure, skid base, and associated equipment that moves with the derrick from well to well. Associated equipment typically includes rotary table, setback, and catenaries (drag chains), and can also include items such as mobile mud and cuttings cleaning equipment.

NOTE 2 Provisions for drilling derricks can be found in API Spec 4F[22].

Details for the drilling operations at a specific platform (including the various configurations of moveable and stationary equipment and materials) should be developed in coordination with project/asset drilling engineers. Background information on drilling can be found in Reference [23].

The reactions and connections details at the base of the derrick should be passed to the designer of the derrick substructure and skid base. Subsequently the reactions and connections details at the skid base should be passed to the topsides designer.

NOTE 3 This will ensure that forces are consistent across interfaces.

The possibility of uplift or collapse of the derrick structure should be assessed by the derrick designer. Connections between components shall be designed for all conditions including abnormal seismic and metocean conditions.

The consequences of impulse actions such as jarring and vibration due to drilling operations on SECE, protective coatings, and similar sensitive materials and equipment shall be assessed.

10.8 Bridges

Bridges supports shall be fitted with longitudinal, lateral or rotational bearings, 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 (including static and fatigue).

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 events and actions shall be provided with suitable restraints for both lateral and vertical action effects. Both initial action effects and rebound action effects shall be taken into account.

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 taken into account.

Where bridge collapse is allowable during or after certain specified accidental events and actions, 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.

10.9 Bridge bearings

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

— fatigue and wear due to fluctuating actions and movements;

— maximum displacements in extreme, abnormal, and accidental design/assessment situations, including the effect of displacements on assessed actions;

— maximum actions in extreme, abnormal, and accidental design/assessment situations;

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

— installation inaccuracies and tolerances.

Design and fire protection of bearing systems shall take account of needs for inspection and change-out of bearing components in service, including the provision of jacking and lifting points, as necessary.

10.10 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 in extreme, abnormal, and accidental design/assessment 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.

Design and fire protection of AVMs and their support systems shall take account of needs for inspection and change-out of components in service.

10.11 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.

Interfaces with significant potential to compromise structural design assumptions, which can affect design, material and fabrication (welding), can include:

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.

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

10.12 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. Critical structure shall be identified and is 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 – PFP (intumescent coatings flexible jackets 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 taken into account.

Where the PFP can be wetted, the PFP shall be sealed to prevent water ingress and corrosion of the substrate. Where PFP is likely to be wetted frequently or for long periods, the top (weather) coat shall be designed to withstand such conditions.

The PFP 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.

Where protection from pool fires and jet fires is required, PFP products shall be tested in accordance with ISO 22899-1 or ISO 834-1 as appropriate.

PFP that produces toxic fumes in fires to an intolerable concentration shall not be used.

Coat-back lengths for secondary members attached to PFP protected primary members shall be determined to avoid potential system weaknesses (see 7.9.2).

Further information can be found in ISO 13702.

10.13 Penetrations

Penetrations and access openings may be included in structural components provided the component retains adequate capacity. Openings shall be included to allow access for inspection of the surrounding structure, including stiffeners and reinforcement, where necessary.

Where penetrations are required through safety critical barriers, penetrations shall be designed such that the performance objectives of the barrier are not impaired.

Detailing at penetrations shall minimise stress concentrations and corrosion development.

The effects of penetrations and cut-outs on both static and fatigue strength shall be assessed. 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.

10.14 Difficult-to-inspect areas

Provision shall be made at the design stage for accessibility to 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.

10.15 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.

10.16 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 taken into account.

10.17 Walkways, laydown areas and equipment maintenance

Walkways and access ways shall be designed to support an operational 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 operational 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.

If access is for inspection and repair, only walkways and access ways may be designed for 3 kN/m^2 (further guidance is provided in A.7.2).

In addition to the grating itself, the connections of sub-cellar deck grating to the supporting structure shall be designed to accommodate wave loads and ensure the grating remains fully functional and in place under the applicable design/assessment situations consistent with ISO 19900.

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 or swinging 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.

Heavy-duty handrails should be used in laydown areas.

10.18 Muster areas and lifeboat stations

In addition to the uniform allowance for operational 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 operational 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.

Dynamic impact factor applied to the loaded weight of the lifeboat can be subject to regulatory requirements.

11 Materials

11.1 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.

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) behaviour at low service temperatures, if applicable (see ISO 19906 and ISO/TS 35105[24]);

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

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

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

g) resistance to fatigue;

h) maintenance requirements;

i) weight;

j) whole life-cycle cost;

k) availability of material of consistent quality conforming to recognized standards supported by reliable certification.

11.2 Carbon steel

The selection of carbon steels for the topsides structures shall be in accordance with 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.

The lowest anticipated service temperature (LAST) shall be established taking into account all phases (including construction) of the life of the structure.

NOTE Topsides to be installed in warm geographical areas which are built in cold regions can have fabrication and welding problems.

For the material category method, the minimum strength group and toughness classes for components of the topsides structure shall be in accordance with Table 8.

For the design class method, the minimum steel quality level (SQL, described in ISO 19902:2020, 19.6.3.2) and inspection category shall be in accordance with Table 9. Figure 6 to Figure 8 show typical material selection for topsides according to the design class method.

Table 8 — Material category — Material selection for topsides

Component location in topsides		Strength group	Toughness class[a]		
			MC1	MC2	MC3
Deck columns	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 (away from the connections)	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

[a] Where two toughness classes are given, the higher class should be used 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 8 *(continued)*

Component location in topsides		Strength group	Toughness class[a]		
			MC1	MC2	MC3
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 should be used 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 9 — Design class approach — Typical minimum selection for topsides structures

Joint/component	Design class (DC)	SQL[a, c]	Inspection category[b]
Primary Structure			
Lift-nodes (complex)			
Butt welds in tension	1	I	A or B
Butt and fillet welds in shear or compression			C
Lift nodes (non-complex), support nodes and other nodes			
Butt welds in tension	2	I or II	A or B
Butt and fillet welds in shear or compression			C
Columns and diagonals to lift nodes			
Butt welds	2	II	A or B
Butt and fillet welds in shear or compression			C
Cans & conical transition	2	I	A
All other columns and diagonals			
Butt welds in tension	4	III	B or C
Butt and fillet welds in shear or compression			B
Cans & conical transition	2	I	A or B
Deck structures			
Plate Girder, Box Girder and Rolled beams in main gridlines	2	I or II	A or B
Plate Girder, Box Girder and Rolled beams in other gridlines	4	III	B or C
Crane pedestal and pedestal support	1	I	A or B
Flare chords and members	2	II	B
Stiffeners	4	III	B or C

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

[a] SQL I to be selected when joint strength through-thickness ductility is required.

[b] Inspection category allocation shall be based on ISO 19902:2020, Tables F.2 & F.3.

[c] Elements and connections sensitive to fatigue (e.g. rotating equipment supports) should have a DC2 and SQL I or II.

Table 9 *(continued)*

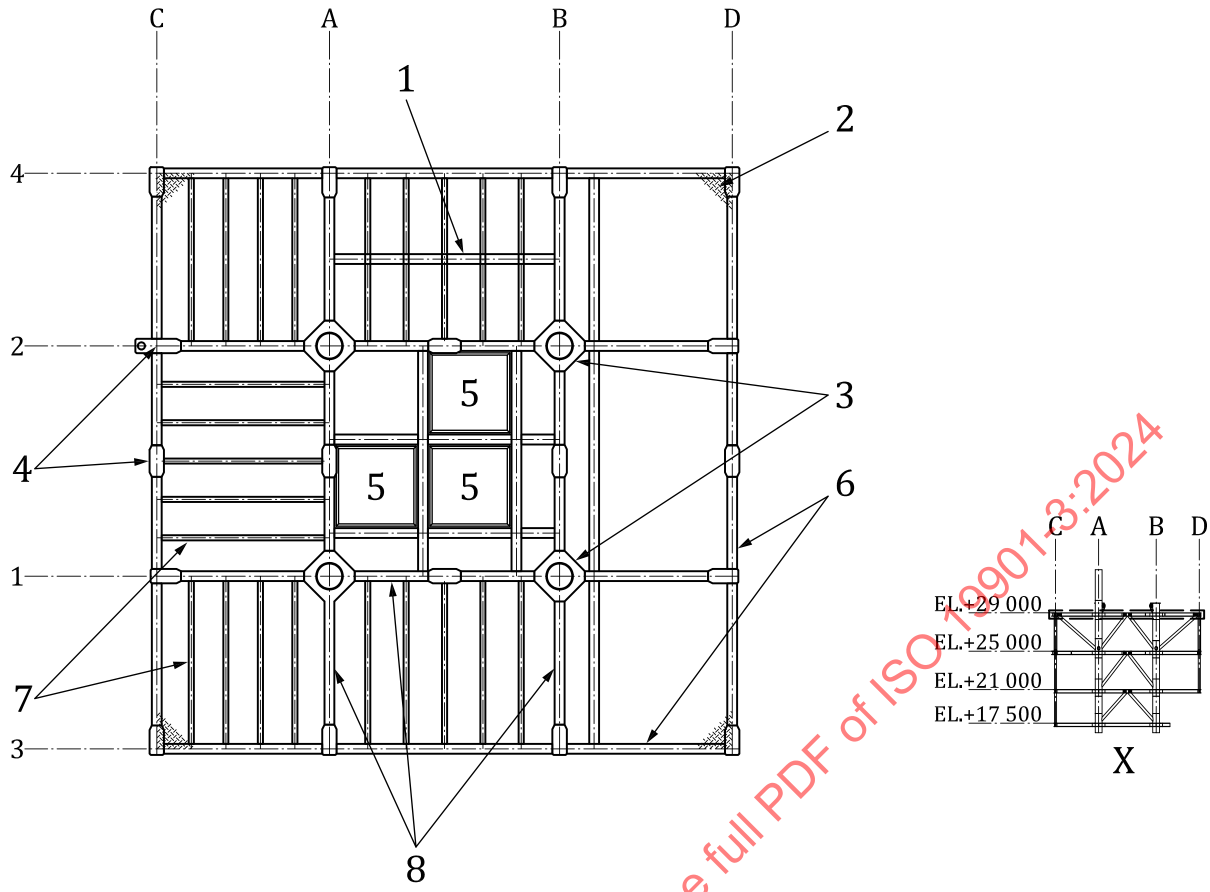
Joint/component	Design class (DC)	SQL [a, c]	Inspection category [b]
Bulkhead connection to deck girders			
Butt welds with high restraint and triaxial stresses	3	III	B or C
Secondary Structure			
Horizontal bracing/transverse			
Butt welds	4	III	B or C
Fillet and longitudinal welds			C
Deck structures			
Beams/Girders/Transverse	4	III	B or C
Blast/Explosion and Fire walls			
Equipment and piping major support structures (>10 t)			
Piperack and large pipe supports			
Deck plate and stringers			
Lay-down			
In-deck tank support beams			
Crane boom rest			
Walkways and Stair towers			
Mezzanine decks and large access platforms			
Equipment and piping minor support structures (<10 t)	5	III	C
Runway beams and lift lugs			
work limit load >10 t	2	I or II	A or B
work limit load <10 t	4	III	B or C
Tertiary Structure			
walkways & access platforms	5	IV	D-E
stairways, handrail, ladders			
Installation aid structure			
Seafastening	2	I or II	A or B
Spreader beam	2	I or II	A or B
Grillage and skidway	4	III	B or C
Tuggerline padeyes	4	III	C
Installation bumper and guide	5	IV	D

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

[a] SQL I to be selected when joint strength through-thickness ductility is required.

[b] Inspection category allocation shall be based on ISO 19902:2020, Tables F.2 & F.3.

[c] Elements and connections sensitive to fatigue (e.g. rotating equipment supports) should have a DC2 and SQL I or II.



Key

X key elevation

1 DC4 transverse

2 DC4 secondary structure deck plate/grating

3 DC1 lift nodes padeye (complex)

4 DC2 primary structure – other nodes

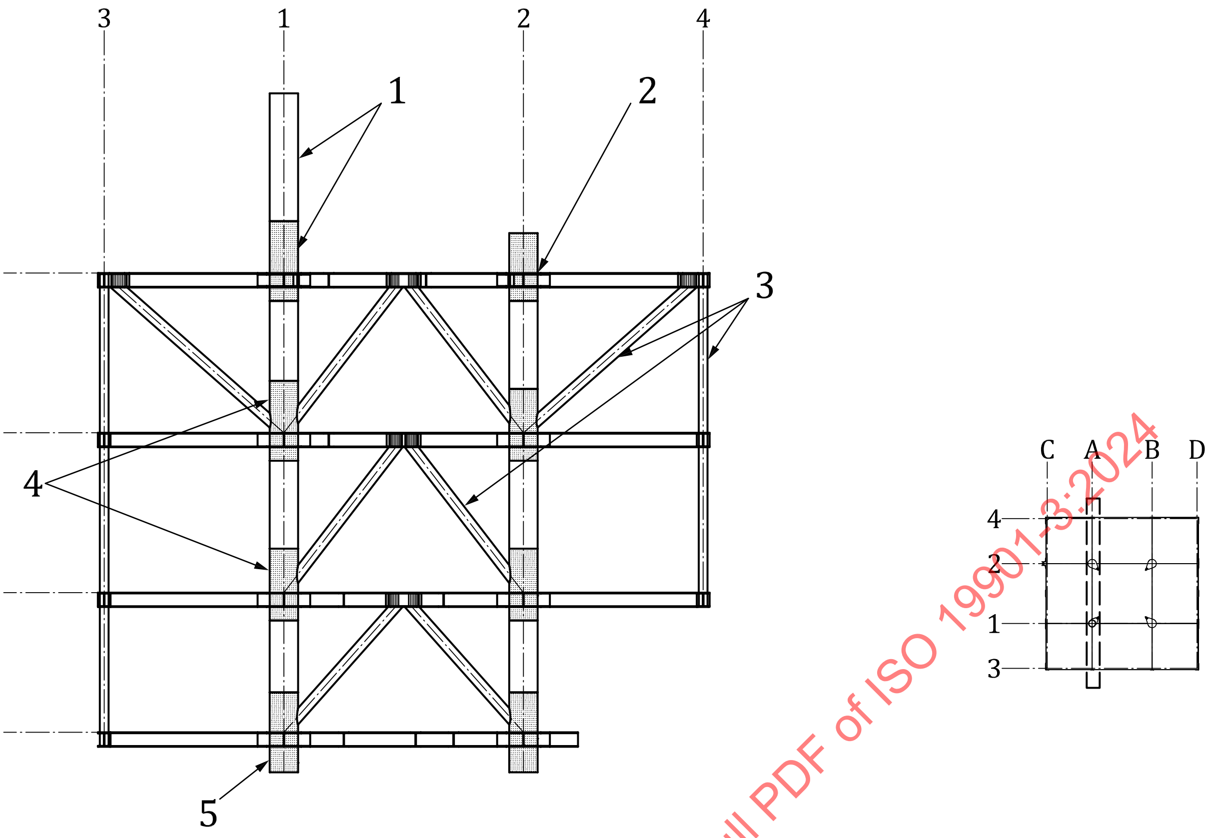
5 hatch for well access

6 DC4 primary structure – other main gridlines

7 DC4 secondary structure – deck beams

8 DC2 – primary structures – main gridlines

Figure 6 — Main deck – example



Key

1 DC1 primary structure – pedestal crane

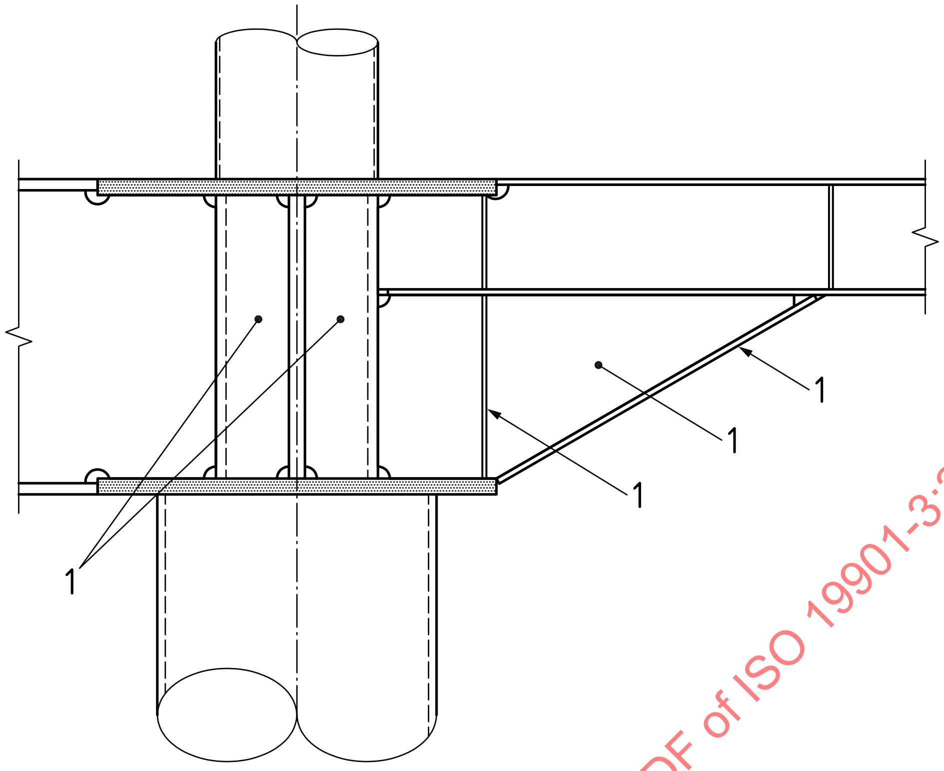
2 DC1 lift nodes padeye (complex)

3 DC2 primary structures – all other columns and diagonals

4 DC2 primary structures – all other columns and diagonals – cans and conical transitions

5 DC2 primary structures – support nodes and other nodes

Figure 7 — Main row – example



Key

1 DC4 stiffeners

Figure 8 — Node detail – example

11.3 Stainless steel

11.3.1 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. The risk of galvanic corrosion of connected materials, particularly with carbon steel, shall be taken into account.

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.

11.3.2 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 the ISO 3506 series[25].

11.3.3 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 accounted for 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.

11.4 Aluminium alloys

11.4.1 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.

11.4.2 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 should be selected from materials with a strength not exceeding 130 N/mm^2 at the specified 0,2 % strain.

Alloys with a strength exceeding 130 N/mm^2 may be used for non-welded construction.

11.4.3 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 (6 082 alloy): 250 N/mm^2

— yield strength (5 083 alloy): 110 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.

11.4.4 Thermite sparking

Thermite (aluminium-iron oxide) sparking can occur when iron oxide (rusty steel) comes into contact with aluminium. When thermite sparking occurs in combination with an explosive gas/air mixture it represents a hazard with appreciable energy.

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.

11.5 Fibre-reinforced polymers (FRP)

Fibre-reinforced polymers (FRP) can be produced with a wide range of properties, including high strength. 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 lack of design codes or standards for use of these materials and their suitability is usually determined by type-testing to meet performance criteria.

FRP 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.

FRP shall be assessed as an alternative to steel for floor grating, hand railing and ladders, lightweight fire and explosion-resistant panels, and miscellaneous access platform.

Examples of specific design and material aspects that can be used to assess and select FRP elements are listed in A.11.5.

In addition to the grating itself, the connections of sub-cellar deck grating to the supporting structure shall be designed to accommodate wave loads and ensure the grating remains fully functional and in place under the applicable design/assessment situations consistent with ISO 19900.

NOTE Grating at the sub-cellar deck or other lower levels of the platform can be subject to occasional wave loading.

Primary escape routes and access to safety critical active fire protection devices (e.g. water deluge or foam spray) shall not use FRP grating.

For locations where FRP does not satisfy the functional requirements, suitable steel components shall be specified.

FRP are often electrically non-conductive, and any conductive and metallic objects attached to FRP components should be independently earthed where necessary.

In fire conditions, FRP can give off toxic fumes and the risks from such fumes shall be assessed.

11.6 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.

12 Fabrication, quality control, quality assurance and documentation

12.1 Assembly

12.1.1 General

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

Fabrication for all materials, other than welding, shall be in accordance with a national building standard for fabrication that complements the design standard. 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.

12.1.2 Grating

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

12.1.3 Landing and stairways

Landing elevations and landings 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.

12.1.4 Temporary attachments

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

a) temporary attachments shall not be removed by hammering or arc air gouging. Attachments shall be cut to 3 mm above parent metal and mechanically ground to a smooth flush finish with the parent metal.

b) attachments on all areas that are to be painted shall be removed as above, prior to any painting.

c) attachments to aid in the splicing of legs, braces, sleeves, piling, conductors, etc. shall be removed and the surface ground to a smooth flush finish.

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

12.2 Welding

Welding shall conform to the requirements of ISO 19902 with the following additional provisions:

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 conform to the requirements of ISO 19902.

b) the design of highly constrained joints should be avoided in order to minimise net residual stresses.

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.

d) partial penetration and fillet welds shall be sized based on the tensile strength of the weakest element connected to the weld unless sufficient ductility is documented.

e) Fillet welds between webs and flanges may be designed equal to the shear capacity of the web unless loaded attachments are welded to the flange (e.g. web to web connections and plate girder web to flange connections).

f) in seam welds connecting different thicknesses, full penetration weld should be avoided by tapering at 1:4 or lower.

12.3 Fabrication inspection

The requirements for quality control and fabrication inspection shall be in accordance with the applicable clauses of ISO 19902.

12.4 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 structure 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.

12.5 Corrosion protection

12.5.1 Coatings

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

12.5.2 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.

12.5.3 Dissimilar materials

Where differing materials are used, the detailing of the connections shall be such as to avoid galvanic corrosion.

Fasteners for bolted aluminium support structures and clamps for the helideck decking shall have:

a) class A4-80 bolts;

b) type 316 SS washers under both the bolt head and nut;

c) washers coated with a 2-layer PTFE coating with a minimum thickness of 20 μm;

d) corrosion protective paste of type Molycote P-40 or equivalent applied on the bolt threads and shank.

Dissimilar materials (e.g. stainless steel cable trays and aluminium helidecks) shall be segregated from each other with non-metallic shims with a thickness of minimum 2 mm. The shims shall have sufficient creep resistance to maintain dimensional stability under the loads from the bolted connection. The non-metallic shims and washers shall have an excess protrusion onto the aluminium surface to ensure sufficient distance to prevent galvanic corrosion.

To prevent ingress of water, contact surfaces (including the bolt and rivet holes), shall prior to assembly be cleaned and receive a sealing compound extending beyond the contact area.

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

In-service inspection, monitoring and maintenance of corrosion control of topsides structure and structural components shall be performed in accordance with ISO 19901-9.

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 account any requirements of the operator.

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 document in conjunction with all factors from ISO 19901-6.

Installation aids that are planned to be removed shall be designed taking into account methods and sequences for their safe and easy removal.

Requirements for removal of temporary attachments are provided in 12.1.4.

The quay soil, the barge grillage/structure and its supports as well as the structure itself shall be verified accounting appropriately for all structural stiffnesses and relative displacements.

Deflections of the topsides structure during pre-service (fabrication, loadout, transportation and installation operations) shall be evaluated for each relevant phase to verify equipment and piping serviceability and structural strength are in accordance with 7.8, 10.2.3 and 10.2.4. Simplifications from 10.2.1 may be applied.

14 In-service inspection and structural integrity management

14.1 General

The requirements for in-service inspection and structural integrity management shall be in conformance with ISO 19901-9 for all topsides structure covered by this document, as well as the provisions applying to topsides structures given in 14.2.

14.2 Requirements applying to topsides structures

14.2.1 Corrosion protection systems

Corrosion protection systems shall be suitably maintained to retain their effectiveness.

NOTE 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 ISO 19902).

14.2.2 Critical structures

Inspection frequency, inspection methods and access for inspection shall be determined for critical structures as listed in 5.7 taking account of degradation mechanisms and other possible damage conditions.

14.2.3 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.

14.2.4 Accidental events and incidents

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 accident or incident.

14.2.5 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.

Annex A
(informative)

Additional information and guidance

A.1 Guidance on scope

No guidance is offered.

A.2 Guidance on normative references

No guidance is offered.

A.3 Guidance on terms and definitions

No guidance is offered.

A.4 Guidance on symbols and abbreviated terms

No guidance is offered.

A.5 Guidance on overall requirements

A.5.1 Conceptual design

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[26],which is only concerned with components formed from cylindrical tubular sections. Topsides, however, are constructed from a range of structural sections so API RP 2A-WSD[26] provides little design guidance relevant to topsides. As a result, different countries adopted a variety of national building standards to effect such designs, e.g. ANSI/AISC 360-22 in the USA, the so-called “Eurocodes” in Europe and CSA-S16:19[14] 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 360-22, which was calibrated as AISC LRFD[27] against API RP 2A-LRFD[28] with resistance factors derived for consistency with the reliability implicit in API. CSA-S16:19[14] was calibrated against land-based steel building practice and the resistance factors were determined for a given set of action factors.

A.5.3 Deck elevation

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 (or design iceberg) and the lowest point (structural component, equipment or fixing) of the lowest deck of the platform such that abnormal wave crests (or ice) 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 (or ice) 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 (or iceberg shape).

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 requirements

No guidance is offered.

A.5.6 Design physical environmental conditions

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

A.5.7 Critical structure

No guidance is offered.

A.5.8 Assessment of existing topsides structures

No guidance is offered.

A.5.9 Reuse of topsides structure

No guidance is offered.

A.5.10 Repairs, modifications and refurbishment

No guidance is offered.

A.6 Guidance on design requirements

A.6.1 General

The design of all systems of a topsides should be undertaken by competent engineers and drafters with appropriate training and qualifications that as a team has sufficient design and on-site experience.

A.6.2 Design/assessment situations

No guidance is offered.

A.6.3 Materials selection

Material selection and fabrication of equipment packages supplied by specialist suppliers can use agreed referenced standards and specifications (e.g. derrick system, crane system, rotating equipment packages, etc.).

A.6.4 Structural interfaces

No guidance is offered.

A.6.5 Design for serviceability

A.6.5.1 Serviceability limits

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 assessed.

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 higher than or lower than principal operating frequency with ±20 %. Where this cannot be achieved, the amplitudes should be assessed and shown to be within acceptable limits. A dynamic forced response analysis should be performed to satisfy requirements for accelerations in various areas.

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

A.6.5.2.2 Design limits on vibrations

ISO 2631-1, ISO 2631-2 and ISO 6897[29] present methods of determining, and guidelines on, the effects of vibrations on personnel. 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/s2 are not likely to be uncomfortable and those in the range 0,315 m/s2 to 0,63 m/s2 can be a little uncomfortable. Further guidance from ISO 2631-1, ISO 2631-2 and ISO 6897[29] should be followed where expected accelerations are greater than 0,63 m/s2.

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 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 EN 1993-1-1:2022 [13] but have been simplified. In EN 1993-1-1:2022,[13] 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 strength

No guidance is offered.

A.6.7 Design for fatigue

Based on experience, a transportation duration above 21 days can result in significant fatigue damage.

Even if flare nodes are accessible for inspection and repair, a minimum fatigue partial damage design factor of 3 should be applied to take into consideration that those nodes are usually not easy to access for inspection or repair.

The fatigue partial damage design factors given in Table 2 have been modified compared to those given in ISO 19902 for the substructure. The change allows smaller design fatigue factors (γ_{FD}) for certain topsides components, the table is better aligned with other industry standards such as Reference [30], DNV-OS-C101,[31] NORSOK N-004[32] and also ISO 19904-1. For components which can't be inspected, ISO 19902 γ_{FD} factors are maintained. For other components, a split has been introduced between readily accessible joints and joints with restricted access. The design fatigue factors for accessible components have been aligned with NORSOK N-004[32] and those not accessible for inspection or repair are similar to those in ISO 19904-1.

A suitable recommended practice for design for fatigue in cases where the substructure standard does not provide adequate guidance for some structural components of topsides is DNV-RP-C203[33].

A.6.8 Robustness

The robustness concept is closely related to resisting and mitigating the effects of hazardous events (see ISO 19900) arising from accidental, abnormal and seismic hazards, consequences of human error, and failure of equipment. Robustness is also important in the event of serious but unidentified fatigue damage.

Robustness is achieved by considering the abnormal and accidental design/assessment situations that evaluate 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 hazardous events to establish design/assessment situations. They will often be those from ship impact, dropped objects, explosions and fires.

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

— careful planning of all phases of development and operation;

— avoiding the structural effects of a hazardous event by either eliminating the hazard at source or by bypassing and overcoming the hazard;

— minimizing the consequences;

— designing for each hazardous event.

Hazardous events should be assessed independently, with only one occurring at a time.

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

Robustness requirements do not imply that all structures should be able to survive removal of any structural component if no hazardous events are likely to occur. The starting point is a hazardous event that is more unlikely to happen than the usual design situations, but not unlikely enough to be neglected. If there is no hazardous event, then there is no requirement in relation to robustness.

A.6.9 Confirmation of execution of design requirements

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 SECE, whether a structural 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 identification 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 for loadout, transportation and installation

No guidance is offered.

A.6.13 Design for structural integrity management

Further guidance can be found in NORSOK Z-001[34].

A.6.14 Design for decommissioning, removal and disposal

A.6.14.1 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.6.14.2 Structural releases

No guidance is offered.

A.6.14.3 Lifting appurtenances

Where lifting attachments are removed, attention 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.6.14.4 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 Guidance on actions and analysis methods

A.7.1 General

No guidance is offered.

A.7.2 In-place actions

During the design of a topsides, the structure is usually 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. Weight allowances and weight reserves are included in the weight database to ensure that the structural design is sufficient for the weights at the conclusion of the topsides design. These weight allowances and weight reserves are progressively reduced as the design matures. The values of G_1, G_2, Q_1, and Q_2 should include these weight allowances and weight reserves until the conclusion of the design. See ISO 19901-5 for further guidance on maintaining a weight database and on weight allowances and weight reserves.

Potential shifts of centre of gravity can be addressed by developing a centre-of-gravity envelope and applying an additional 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 by including weight allowances to ensure weights are not underestimated.

In the absence of specific requirements, operational local actions (Q) for local, primary and global design stated in Table A.1 (adapted from ISO 19904-1) may be used in the structural design of the topsides structure. Local action effects resulting from these actions should be combined with the corresponding global action effects for the structural components in question.

Table A.1 — Typical minimum local actions for topsides structure

Area	Local design[a]		Factor to be applied to distributed action for	
	Distributed action kN/m^2	Point action kN	Primary design[b]	Global design[c]
Storage area	q	1,5 q	1,0	1,0
Laydown area	q	1,5 q	f	f
Lifeboat platform	9,0	9,0	1,0	d
Area between equipment	5,0	5,0	f	[d]
Walkway, stairway and platform	5,0	5,0	f	[d]
Walkway and stairway for inspection and repair only	3,0	3,0	f	[d]
Roof accessible for inspection and repair only	1,0	2,0	1,0	[d]

q shall be evaluated for each case as follows:

— storage areas for cement or wet or dry mud should be 13 kN/m^2 or ρgH, whichever is the larger,

where,

ρ is the mass density (in kg/m^3),

g is the acceleration due to gravity (m/s^2), and

H is the storage height (m);

— laydown areas are not normally designed for less than 15 kN/m^2.

f is equal to either 1,0 or $(0{,}5 + 3/A^{0{,}5})$, whichever is the smaller, where A is the action area, expressed in square metres (m^2).

Wheel actions shall be added to distributed actions where relevant (wheel actions can normally be considered acting on an area of 300 mm × 300 mm).

Point actions shall be applied on an area 100 mm × 100 mm, and at the most severe position, but not added to wheel actions or distributed actions.

For actions on floors in accommodation and office sections, see ISO 2103.

Handrails should be designed for 1,5 kN/m, acting horizontally.

[a] Design of deck plates and stiffeners.

[b] Design of deck beams and deck columns.

[c] Design of deck main structure (and substructure). Global action cases should be established based upon "worst case", representative variable action combinations, conforming to the limiting global criteria to the structure. For buoyant structures, these criteria are established by requirements to the floating position in still water, and intact and damage stability requirements, as documented in the marine operational manual (MOM), considering variable actions on the deck and in tanks.

[d] Allowed to be ignored.

A.7.3 Action factors

A.7.3.1 Design actions for operational design/assessment situations in still water

No guidance is offered.

A.7.3.2 Design actions for operational design/assessment situations with operating environmental actions

Environmental actions can add to or subtract from still water actions. For the platform generally, operating environmental conditions representing normal conditions, which occur relatively frequently at the platform, can be defined by the operator.

Some platform operations can be performed only in environmental conditions which are less than the general operating conditions. The combination of the principal operational action with the lesser environmental conditions which limit the operation could govern the design of parts of the structure.

NOTE 1 Different operating environmental limits can be set for different operations.

NOTE 2 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.

A.7.3.3 Design actions for extreme design/assessment situations

No guidance is offered.

A.7.4 Vortex-induced vibrations

DNV-RP-C205,[36] References [37] and [38] provide information for lattice structures and exposed pipework.

For individual cylindrical tubular structural components in a wind-sensitive structure, the following guidelines can be used to avoid the necessity for a rigorous analysis:

a) member length-to-diameter ratios not exceeding 40;

b) member diameter-to-thickness ratios should be less than 33, i.e.

$$D/\delta < 33$$

where δ is the member thickness;

c) stress concentration factors at the end connections of members should not be greater than 5.

A.7.5 Indirect actions and resulting forces (action effects)

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 Metocean and ice actions

A.7.6.1 Wave, current and ice actions

A.7.6.1.1 General

No guidance is offered.

A.7.6.1.2 Fixed platform

No guidance is offered.

A.7.6.1.3 Floating facilities

No guidance is offered.

A.7.6.2 Wind actions

National building standards for wind actions that are typically used with ISO 19900 include EN 1991-1-4[2] and ASCE/SEI 7-22.[3] Other standards may also be applicable.

When using ASCE/SEI 7-22,[3] the additional guidance on wind actions provided in Reference [39] should be used.

When gust durations are not specified in the national building standard selected for wind actions, guidance in accordance with the type of structure and the nature of the structure's response can be found in ISO 19901-1.

For the most onerous wave loading condition, the wind actions are the companion values, which can lead to a reduction in gust duration or wind speed.

A.7.6.3 Cold region effects

No guidance is offered.

A.7.7 Seismic actions

A.7.7.1 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 does not exceed 0,10 g, the design of fixed platforms for storm conditions generally produces substructures 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 when 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 can 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 7-22[3] and EN 1998-1[40].

A.7.7.2 Minimum lateral acceleration

A minimum lateral deck acceleration of 0,05 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.7.7.3 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, the increased response towards the corners of the deck caused by torsional response of the platform should be taken into account.

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 – additional forces can be 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[28] and ASCE/SEI 4-16[41] 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. Higher partial action factors are 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, adjustment to partial action factor can be appropriate. In addition, for certain equipment, piping, appurtenances or supporting structures, the use of different (i.e. higher) partial action factors for the ELE or a full ductility analysis can be appropriate, depending on the component's anticipated performance under abnormal earthquake ground motions and the degree of redundancy, consequences of failure and/or characteristics of the metallurgy.

A.7.8 Actions during fabrication, loadout, transportation and installation

Damage to intumescent PFP can be caused due to deflection and deformation during loadout, transportation and installation. PFP vendors can provide deflection limits that, if exceeded, can result in damage to the PFP.

A.7.9 Actions due to accidental hazards

A.7.9.1 General

The capacity of the topsides structure is designed, as a minimum, to limit the risk of harm (life-safety, environmental and business) to a tolerable level, provided the risk is also ALARP and in addition, to a risk level as low as reasonably practicable. Robustness and inherent safety are important design considerations. Provision of adequate structural strength and a ductile structural response provide an inherently safer design of critical structure than where critical structure fails in a brittle manner.

Structural engineers designing for accidental events typically have knowledge of the following design approaches (EN 1990:2020[43] Clause C.3 and ISO 2394[1]):

— semi-probabilistic design (such as partial factor approach);

— performance-based design (i.e. simplified reliability-based design);

— risk-informed design (design situations where both the uncertainties and the consequences are outside common ranges).

ISD of topside facilities uses the following strategies:

a) eliminate or avoid – eliminates risk of harm by eliminating the hazards or removing the exposure to MA hazards by design;

b) substitute – reduces severity of harm by replacing hazardous materials with safer materials (but recognize other trade-offs);

c) moderate – reduces severity of harm by using less hazardous conditions, or facilities that minimize the impact of a release of hazardous material or energy;

d) minimize – reduces probability of harm by reducing the hazardous inventories or the frequency or duration of exposure;

e) simplify – reduces probability of harm by reducing complexity and make operating errors less likely.

The residual risk of harm following ISD is further reduced by barriers as shown in Figure A.1.

Prevention barriers prevent occurrence of MAs or reduce their likelihood of occurrence.

Mitigation barriers limit the extent and duration of any MAs that occur and/ or limit the severity of harm from MAs that occur and allow effective emergency response.

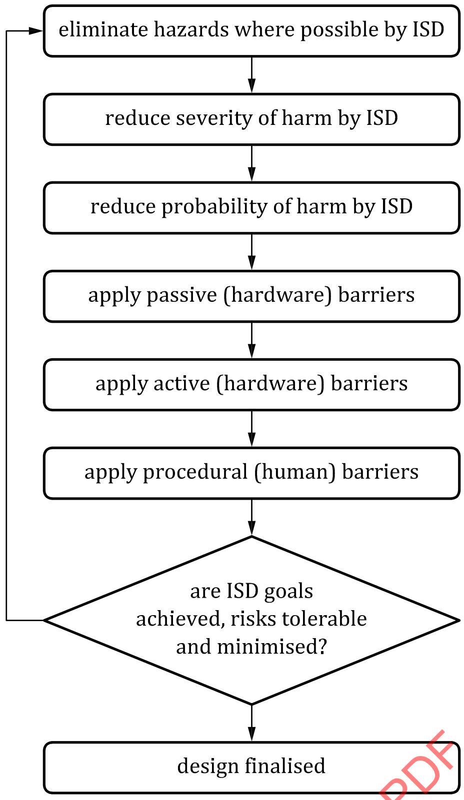


Figure A.1 — ISD and barrier steps for risk-based design

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 scrutinised so as to minimize potential damage from these causes.

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

A range of hazardous events for each accidental hazard should be evaluated. For example, gas explosions have 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. The explosion hazard curve defines the annual probability of exceedance of overpressure.

A.7.9.1.1 Accidental hazards

No guidance is provided.

A.7.9.1.2 Design for accidental events

No guidance is provided.

A.7.9.1.3 Intensity of hazardous event

If a risk-based approach to the assessment of major hazards is adopted, the risk can be expressed as a frequency of exceeding a certain level of consequence per annum. The risk calculation requires:

a) a hazard curve (i.e. the probability of exceeding a given intensity measure of the hazard). Examples include the frequency of exceeding a certain blast overpressure at a certain location per year (e.g. at the TR or blast wall or other SECE); or the frequency of exceeding a certain fire size per year at a location of a SECE after a certain time of exposure; or the frequency of exceeding a certain level of damage per year. The location is typically chosen to be the key SECEs of interest that are required to meet a certain survivability criteria. The results are project and layout specific and cannot be copied to other cases.

b) A vulnerability curve (i.e. the level of consequence conditional on a hazardous event of given intensity having occurred).

A.7.9.2 Structural design for fire hazards

A.7.9.2.1 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 or contraction can be significant for highly restrained components and should be taken into account. Contraction of members can result in connection rupture during the cooldown phase. Also, gravity induced deformations can continue after the fire event thus increasing the P-Δ moment which can lead to collapse. Continuing the analysis to simulate the post-fire response until the structure becomes stable or collapses can be used to obtain the total endurance time

A.7.9.2.2 Fire actions

Calculation methods for the temperature of topsides structure in a fire include:

— statistical models based on previous studies for early stage design [5];

— empirical models;

— zone (phenomenological) models;

— computational fluid dynamics (CFD), or

— field models[44].

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 be computationally challenging due to the thermal properties of the structural material and PFP differing by an order of magnitude, and limitations of available PFP data to simulate thermal properties. An equivalent heat transfer coefficient can be used to simplify the process using fire test results and or certification data.

Examples of the effects on the stress/strain characteristics of ASTM A36/A36M[45] and ASTM A633/A633M[46] Grade C and D steels at elevated temperatures are presented in Table A.2 (see Reference [102]) for temperatures in the range 20 °C to 1 200 °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 taken into account[6].

This document does not include any thermal data for other structural materials (see Clauses 10 and A.10).

Table A.2 — Yield strength reduction factors for steel at elevated temperatures (ASTM A36/A36M[45] and ASTM A633/A633M[46] 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
200	0,847	0,946	1,000	1,000
300	0,653	0,854	1,000	1,000
400	0,600	0,798	0,956	0,971
500	0,467	0,622	0,756	0,776
600	0,265	0,378	0,460	0,474

A.7.9.2.3 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 EN 1993-1-2[48]), 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 EN 1993-1-2[48]). Internal radiation from warm to cold surfaces should be evaluated 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 is designed 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.7.9.2.4 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. If fire duration is relatively short, 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 explicitly evaluated.

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 EN 1993-1-2.[48] Assessment of ultimate strength of carbon steel is not needed if the steel maximum temperature does not exceed 400 °C.

A.7.9.2.5 Nonlinear finite element analysis for response to fire actions

A.7.9.2.5.1 General

Structural analysis methods for nonlinear ultimate strength assessment can be classified as:

— distributed plasticity formulation, or

— lumped plasticity formulation.

Distributed plasticity represents nonlinear material behaviour at a fibre level. Lumped plasticity represents nonlinear material behaviour by closed-form solutions for interaction equations for cross-sectional forces and moments.

A.7.9.2.5.2 Material modelling of carbon steels

In stress–strain-based analysis of carbon steel structures, the temperature-dependent stress–strain relationships given in Reference [6] or EN 1993-1-2[48] may be used.

For stress-resultant-based design, the temperature reduction of the elastic modulus may be taken from EN 1993-1-2.[48] The yield strength temperature reduction can be taken as equal to the yield strength at 2 % strain (see Table A.2).

A.7.9.2.5.3 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. Alternatively, a small lateral load (e.g. a fraction of wind load) can be applied to vertical members to introduce imperfections.

A.7.9.2.5.4 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.7.9.2.5.3.

If beam modelling is used, local cross-sectional buckling should be explicitly evaluated.

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 evaluated. 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.7.9.2.5.5 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 5 % of member length for a reasonably uniform temperature may be assumed unless shown to be higher through tests. Local temperature peaks can localize plastic strains. Using critical strains for steel under normal temperatures is generally considered conservative. Triaxiality factor adjusted membrane strain limits can be applied for a refined assessment[49].

b) **Connections**

In place of more accurate calculations, the strength of the connection at a temperature θ may be taken as in Formula (A.1):

$$R_\theta = k_{y,\theta} \cdot R_0 \quad (A.1)$$

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;

$k_{y,\theta}$ is the yield strength reduction factor for the maximum temperature, θ, in the connection (see A.7.9.2.2).

A.7.9.2.5.6 Accuracy of fire thermal and response analyses

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, it can be considered that the structure has 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 U 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.7.9.2.6 Fire mitigation

In the event of a fire, the consequences can be mitigated 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.3. These ratings are applicable to PFP materials subject to pool fires.

Particular attention should be given to the application of PFP materials for jet fire service (see ISO 22899-1).

It has been a common practice to apply PFP also on the secondary stiffeners from the joint to a certain length. Coat-back analysis should be performed to determine the minimum coat-back length (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). The extensive application of coat-back can be very costly and tedious process. Particular attention should be given to ensuring adequate protection to beams supporting gratings and to the supports to SECE[50][51][52][53].

Table A.3 — 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[c]	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; and "A" and "B" for cellulosic fires. The second element describes the required minimum protection time, in minutes. The intensity of the "H", "A" and "B" fires are described in Reference [4] and ISO 834-1.

[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.

[c] Some class societies and PFP vendors use different notation for pool fire rated PFP to indicate less than 120 minutes fire rating and a maximum steel core temperature of 400 °C.

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. However, the effect of active fire protection systems on jet fires is known to be limited and no credit is typically taken for those cases.

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. Higher steel core temperatures can be justified through analysis for assessment of existing structures. 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.7.9.3 Structural design for explosion hazard

A.7.9.3.1 General

For the initial and conceptual design stage of a project, overpressures, and impulses in certain areas on a platform can be estimated using the simplified computational fluid dynamics (CFD) analysis or phenomenological models by competent engineers. The effects of likely interaction between explosion actions and the response of the structure should be evaluated. 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.

A.7.9.3.2 Explosion actions

Drag actions are caused by explosion-generated wind from the flow of air gases and combustion products past an object, produced either from unburned fuel mixture being pushed in front of the blast wave, or burned gas following behind. 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.

References [54], [20] and [55] provide further background on structural design of piping and equipment subjected to vapor cloud explosions.

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. 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[20][56].

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. 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.

A.7.9.3.3 Overpressure time-history

Explosion actions result from increases in pressure due to expanding combustion products. These actions are characterized by a simplified pressure–time curve; an example is given in Figure 2. Explosions actions can govern the design of many components such as explosion walls, floors, and roofs. When simplifying 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.

A.7.9.3.4 Structural response analysis

A.7.9.3.4.1 General

The structural response to explosion actions can be determined by:

— simple calculation models based on SDOF analogies and elasto-plastic methods of analysis (see A.7.9.3.5), or

— nonlinear dynamic FEA (see A.7.9.3.4.3).

Analysis of design situations based on an SLB 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 SECE, 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 analysis or quasi-static analysis are typically performed, using a linear FE model, 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 modelled 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.7.9.3.4 and A.7.9.3.5 should either be implicitly covered by the modelling adopted or be subjected to particular attention, 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.7.9.3.4.2 Dynamic analysis

The pressure–time curve generated by a CFD analysis as part of the assessment process in 7.9.3 and A.7.9.3 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 elasto-plastic 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[57][59] 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 conform with ductility and stability requirements for the structural component. For charts for rebound response, see Reference [10].

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, see Formula (A.2):

$$I = \int_0^{t_d} p(t)dt \tag{A.2}$$

— 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 [10] and [60]. Guidance on design of equipment for explosion actions is given in References [61] and [62].

A.7.9.3.4.3 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 included 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.7.9.3.4.4 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.3).

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

$$S_{\mathrm{d}} \le R_{\mathrm{d}} = K_{\mathrm{c}} \times \frac{R_{\mathrm{k,\,code}}}{1{,}0} \qquad \text{(A.3)}$$

where

F_{d}	is the design value of the action;
G	is the permanent action;
Q	is the operational 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 correspondence factor (see 8.1);

$R_{k,\,code}$ is the representative resistance of the structure or component.

A.7.9.3.4.5 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 the strain at fracture as discussed in A.7.9.3.4.6.

A maximum displacement can be dictated by the ductile bending and rotation capability of the 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 SECE 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 [10], [60] and [61] for more information on deformation limits.

A.7.9.3.4.6 Strain limit

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.

For typical structural steel grades, the strain from a nonlinear finite element analysis (FEA) should be limited in conformance with DNV-RP-C208[19].

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

A.7.9.3.5 Simplified design methods

A.7.9.3.5.1 Design of structural components

Simplified design methods can be used for some aspects of design of structural components, as follows:

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 taken into account 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 DNV-RP-C204.[63] 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 is, however, to 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. The possibility of rupture due to excessive straining should be taken into account.

b) **Beam or girder**

1) Resistance relationships of beams for the elastic, elasto-plastic, and rigid-plastic domain, based on SDOF models can be found in DNV-RP-C204.[63] Check beams and girders with slender cross-sections 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 including these additional actions, lateral instability modes can be missed, with consequent non conservatism in the analysis results.

5) Reference [10] gives guidance and worked examples on deck girders designed by manual methods. Reference [44] gives some limited guidance on the analysis of deck girders by nonlinear FEA.

A.7.9.3.5.2 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 [10] and [60]. Ductility limits for beams proportioned to class 1 are limited by the onset of local buckling (for further guidance, see Reference [64]). Simplified formulae extracted from Reference [64] are contained in Reference [10].

The effective flange of the stiffened deck plating should be evaluated to allow calculation by SDOF methods. DNV-RP-C204[63] 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 considered where this information is available. Alternatively, for beams and plates with full or partial restraint, the limiting deformation at tensile fracture is given in DNV-RP-C204[63].

A.7.9.3.5.3 Supports reactions, beam releases and non-fixed connections

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.7.9.3.5.4 Material properties for design

Strain rate affects yield strength and flow stress. Reference [20] 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 [20] 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.7.9.3.6 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 blast walls.

Walls and floors can be designed as explosion barriers 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. 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 under pressure phases. Where explosion waves are reflected, pressures can be augmented increasing applied actions and mortality and injury rates for persons in such zones[4][44].

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 Reference [4] and Reference [47].

Further guidance on design of explosion mitigation systems, including explosion relief panels, can be found in Reference [57] and Reference [58].

A.7.9.4 Explosion and fire interaction

A.7.9.4.1 General

The design strategy for fire can affect the design for explosion and vice versa.[4] For example, the design strategy for fire could be segregation of the topsides into small zones using fire walls to contain the fire. However, this segregation will likely 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 design strategy. These conflicts should be identified and resolved when designing the topsides.

Fire and explosion assessments should be performed together and the effects of one on the other should be carefully evaluated. 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 following subclauses describe practical considerations for designing a structure to resist fire and explosion actions.

A.7.9.4.2 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.7.9.4.3 Blast walls and fire walls

Designs should allow as large a displacement as possible at mid-span; however,

a) member shortening under large lateral displacements can impose severe actions on top and bottom connections, and

b) 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.7.9.4.4 Beams

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 because of the axial action and lateral deflection should be included 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 considered.

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 evaluated when designing for explosion actions. For 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.7.9.4.5 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.7.9.4.6 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 [61] and [62].

A.7.9.5 Cryogenic spill

No guidance is offered.

A.7.9.6 Actions due to vessel collision

The vessel collision analysis evaluates both the frequency of the risk and the consequence of the collision event. The overall approach for conducting the vessel collision frequency analysis should involve the following steps:

1) overview of historical vessel collision information;

2) field related traffic vessels review;

3) damage level of vessel collision;

4) collision frequency;

5) probability of vessel collision

6) individual risk calculation, and;

7) potential loss of life calculation.

The structural consequence of an accidental vessel collision to an offshore asset is predicted using either a simplified approach (if applicable) or non-linear static or dynamic FE analysis. Damage should be allowed to absorb the impact energy when designing for life-safety risk. If the offshore structure can absorb the impact energy and damage caused by the vessel impact is acceptable, no action is required.

The parameters influencing the collision analysis include:

— mass and stiffness of vessel and platform structures;

— added mass depending on the vessel impact direction;

— impact speed and direction;

— location of impact on the vessel and platform;

— location of centre of mass and rotation of the vessel;

— bow shape (bulb/no bulb) in bow collision or side/stern geometry for broadside/ stern impacts;

— coefficient of friction;

— damping (structural and hydrodynamic);

— platform deck mass and location of CoG;

— steel stress-strain curve and rupture strain and fracture properties;

— strain rate effects;

— analysis methodology and assumptions including; e.g. buckling, and post-buckling behavior;

— finite element model details and assumptions;

— platform foundation;

— platform fendering or collision protection design;

— damage acceptability criteria;

— mesh refinement and the selection of the time increment in the nonlinear analysis.

Reference [65] provides further background.

A.7.9.7 Actions due to 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 affected: in place of other information and further analysis, a 10-year return period environmental event should be used[63].

A.7.9.8 Actions due to loss of buoyancy

Since the industry has experienced several abnormal hull-heeling incidents involving semisubmersibles, it is recommended that a check be carried out to ensure that topsides on semisubmersibles (and possibly on other floating structures) can survive the maximum heeling angle for which the platform remains stable. This angle typically corresponds to the lesser of:

a) the angle for which the area under the righting moment curve is equal to or greater than 1,30 times to 1,40 times the area under the wind overturning arm curve, as defined by the governing classification society rules, or

b) 25°.

The purpose of this check is to ensure that the topsides can survive an accidental event where the hull heels to an abnormal angle up to a maximum value determined by the lesser of 1) or 2) above. In this context, survival means that the topsides will not be damaged to the extent that it is on the verge of collapse. Performing such a check is viewed as part of the operator’s ALARP process.

A.7.9.9 Actions due to topsides acceleration

Acceleration 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, living quarters and the platform's safety systems and components such as flanges on pipework and risers. Acceleration can cause damage due either the resulting inertial actions causing overload or the oscillatory response of the structure causing low cycle fatigue. Safety systems that can be vulnerable to damage from acceleration include emergency shutdown systems, emergency power supplies and communications, fire and gas detection systems, fire protection systems and evacuation, escape, and rescue equipment.

Control equipment, including computers and microprocessors, and telecommunications equipment, can be particularly susceptible to high-frequency 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, significant 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.7.10 Other actions

No guidance is offered.

A.8 Guidance on strength and stability of structural components

A.8.1 Correspondence factor K_c

A.8.1.1 K_c determined by equivalent reliability

A rigorous approach to determine K_c is by equating the probability of failure for a critical member in the primary truss of a topsides, based on U=1 for the required national building standard, with the probability of failure for a critical member in a jacket, based on U=1 for ISO 19902. Examples of the procedure are given in Reference [111] and in EN 1990:2020, Annex C.

The K_c values in Table 4 were calculated based on members subject to combined axial compression and bending with a typical reduction in axial capacity due to bending of 10 % to 20 %. (i.e. typical of a compression brace in a truss that is subject to reverse curvature bending from deformation-induced moments arising from the braces in the truss being welded to the chords rather than pinned).

The axial force in the topside brace was assumed to have contributions from permanent topside action, variable topside action and Metocean (wind) action acting on the topsides.

The axial force in the jacket member was assumed to have contributions from permanent topside action, variable topside action and Metocean (wave) action acting on the jacket.

The axial force in both the jacket member and the topsides brace was assumed to have a fixed ratio where axial force due to variable action was 1/3 of the axial force due to permanent action

(i.e. $P_Q = \frac{P_G}{3}$).

The ratio of axial force due to metocean action was varied from 0 % (i.e. entirely gravity actions) to 100 % (i.e. entirely Metocean actions).

The total axial force in the jacket member and the topsides brace was assumed to equal 1 unit of force

The above assumptions lead to Formula (A.4):

$$P_{ax} = P_G + P_Q + P_E = 1 \tag{A.4}$$

where

$$P_Q = \frac{P_G}{3}$$

$$P_E = r \quad 0 \leq r \leq 1$$

r=0 represents axial force due to gravity action entirely and r=1 represents axial force due to Metocean action entirely.

So, Formula (A.4) results in Formula (A.5):

$$P_{ax} = P_G + P_Q + P_E = \frac{3}{4}\cdot(1-r)+\frac{1}{4}\cdot(1-r)+r \tag{A.5}$$

where $0 \leq r \leq 1$

The limit-state equation (see 9.1, Formula (8)), for the case where U=1 becomes Formula (A.6):

$$S_d = R_d \tag{A.6}$$

or Formula (A.7):

$$\gamma_{load} \cdot S_d = \frac{R_{rep}}{\gamma_R} \tag{A.7}$$

or Formula (A.8):

$$\gamma_G \cdot P_{repG}(r) + \gamma_Q \cdot P_{repQ}(r) + \gamma_E \cdot P_{repE}(r) = \frac{R_{rep}}{\gamma_R} \tag{A.8}$$

or Formula (A.9)

$$R_{rep}(r) = \gamma_R \cdot \left[\gamma_G \cdot \frac{3}{4}\cdot(1-r) + \gamma_Q \cdot \frac{1}{4}\cdot(1-r) + \gamma_E \cdot r\right] \tag{A.9}$$

Limit-state equations (see 9.1, Formula (8)) for three situations were used, each with the U set to 1,0:

a) still water situation;

b) 1-year operating situation;

c) 100-year extreme situation.

The probability of failure is calculated from the following Formula (A.10):

$$P_{annum}(D \geq C) = P_{annum}(C - D \leq 0) \tag{A.10}$$

where

C is a capacity random variable;

D is a demand random variable, and

C-D is the safety margin.

The random variable for demand is Formula (A.11):

$$D(r) = P_{\mathrm{repG}}(r) \cdot X_{\mathrm{G}} + P_{\mathrm{repQ}}(r) \cdot X_{\mathrm{Q}} + P_{\mathrm{repE}}(r) \cdot X_{\mathrm{E}} \cdot X_{\mathrm{W}} \quad \text{(A.11)}$$

where

$P_{\mathrm{repG}}(r)$	is the axial force in the brace due to the representative (mean) permanent action;
X_{G}	is a normal distribution with mean =1, and CoV=8 %;[66]
$P_{\mathrm{repQ}}(r)$	is the axial force in the brace due to the representative (mean) permanent action;
X_{Q}	is a normal distribution with mean =1 and CoV=14 %[66];
$P_{\mathrm{repE}}(r)$	is the axial force in the brace due to the representative (100-year) Metocean action;
X_{E}	is a lognormal distribution fitted to the Generalised Pareto Distribution hazard curves for the wave or wind actions such that: $X_{\mathrm{E}} = 1$ when $P(X_{\mathrm{E}} > 1) = 10^{-2}$ and $X_{\mathrm{E}} = \frac{E_{10\mathrm{k}}}{E_{100}}$ when $P\left(X_{\mathrm{E}} > \frac{E_{10\mathrm{k}}}{E_{100}}\right) = 10^{-4}$
X_{W}	is a normal distribution with bias of 1,0 and 8 % CoV to represent the epistemic uncertainty in wave action (see Reference [67]).

A.8.1.2 K_c determined by utilisation ratio

K_c for national building standards not covered in Table 4 can be determined approximately by utilisation ratio in lieu of an equivalent reliability calculation.

The procedure is:

a) select a typical size of cylindrical tubular 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) the following types of member forces that result in approximately a set of member forces for the element that give approximately 90 % utilizations for the relevant design situation:

   1) pure tension;

   2) pure bending;

   3) pure compression;

   4) combined bending and compression.

d) calculate the utilization of the member for each type of member force following the requirements of ISO 19902 (U_{19902}) for the relevant design situation;

e) calculate the utilization of the element for each type of member force following the requirements of the selected national building standard (U_{code}) for the relevant design situation;

f) determine K_c as the minimum ratio of $U_{\mathrm{code}} / U_{19902}$.

An example calculation is given in Annex B.

A.8.2 Design of cylindrical tubular sections

No guidance is offered.

A.8.3 Design of non-cylindrical sections

A.8.3.1 Rolled and welded non-circular prismatic members

Suitable codes for the design of rolled and welded non-circular prismatic members for offshore topsides structures include ANSI/AISC 360-22,[12] CSA-S16:19,[14] EN 1993-1-1:2022 [13].

A.8.3.2 Plate girder

Suitable codes for the design of plate girders for offshore topsides structures include ANSI/AISC 360-22,[12] CSA-S16:19,[14] EN 1993-1-5,[68] and API Bulletin 2V,[69] DNV-RP-C201[90].

A.8.3.3 Box girder

Suitable codes for the design of fabricated box girders of the size and type normally associated with offshore topsides structures include ANSI/AISC 360-22,[12] CSA-S16:19,[14] EN 1993-1-5[68] and API Bulletin 2V,[69] DNV-RP-C201[90].

A.8.3.4 Stiffened plate components and stressed skin structures

Suitable codes for the design of stiffened plating for offshore topsides structures include CSA-S16:19,[14] EN 1993-1-5,[68] API Bulletin 2V,[69] DNV-RP-C201[90].

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[69].

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 AWS D1.1[70].

A.8.4.3 Bolted connections

A.8.4.3.1 Design and installation requirements

Suitable codes for design of bolted connections are AISC 325[96] and EN 1993-1-8.

Preloading to develop slip-critical connections is an effective way of precluding the effects of reversal of shear forces on the bolt, which can arise from vessel movement. For bolted connections on a floating structure where the dynamic actions due to the inertia of the vessel are insignificant when compared to varying static operational actions, a non-preloaded solution can be used. An example is bolting of handrail supports where bolts are not influenced by the ship inertia but dominated by the live load on the handrail. It is emphasized that the fatigue life on a non-preloaded connection is less than for a preloaded connection.

A.8.4.3.2 Connections classes

No guidance is offered.

A.8.4.3.3 Corrosion protection

No guidance is offered.

A.8.5 Castings and forgings

No guidance is offered.

A.8.6 Design for structural stability

A.8.6.1 General

Offshore structures are designed for pre-service conditions, including for various stages of fabrication, transportation and installation. Some of these conditions, and the in-service/in-place condition include significant metocean actions, which typically for steel framed structures result in structures braced against sidesway that are notably stiffer than unbraced onshore structure. Such structures typically undergo relatively small second order displacements compared with their first order displacements.

The magnitude of B_2 or α_{cr} quantifies the stability of the structure and is used, as described in 8.6, to select the appropriate stability design method.

α_{cr} values for typical fixed lattice steel offshore structures have been calculated by eigen bifurcation buckling analysis and are listed in Figures A.3 to A.7. Figure A.3 shows an example of a portal bay between bottom of deck and top of jacket, which is common in the Gulf of Mexico. Figures A.5 and A.6 show unbraced or partially braced topsides, while Figure A.6 shows an unbraced portal to allow floatover installation of the topsides. Module support frames for topsides equipment are occasionally designed without braces, which reduces their stability.

Symbols α_{cr} and B_2 are used by EN 1993-1-1:2022 [13] and ANSI/AISC 360-22[12] respectively, however, the criteria in 8.6 a), b) and c) can be used to choose the stability design method from other national building standard.

Stability design to EN 1993-1-1:2022 [13],7.2.2 (2) b) can produce unconservative results, see ANSI/AISC 360-22[12], in part due to EN 1993-1-1:2022 [13] not requiring a reduction in stiffness to account for residual stresses.

ANSI/AISC 360-22: Appendix 7, 7.2,[12] i.e. ELM, as recommended in 8.6 c), is a similar stability design method to that in AISC 316-89[72] except that:

a) the interaction equations are bi-linear and linear respectively;

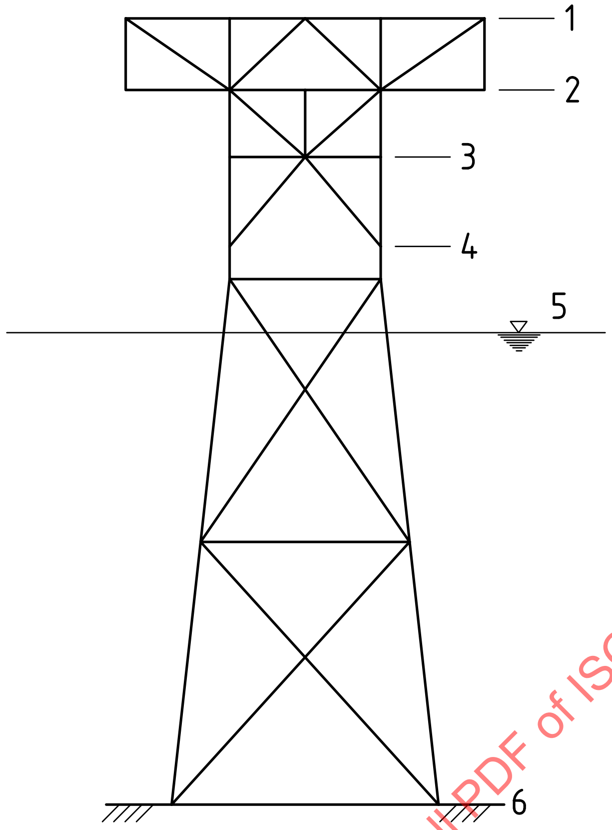
b) 2nd order amplification is applied in the code check equation in AISC 316-89[72] while ANSI/AISC 360-22[12] either explicitly includes amplification by a rigorous 2nd order structural analysis or by amplifying the forces and moments from a 1st order structural analysis.

The AISC 316-89[72] stability design method is also similar to the EN 1993-1-1:2022[13],7.2.2 (2) c) Equivalent Column Method.

A.8.6.2 Braced and unbraced frames

The concepts of sway (unbraced) and non-sway (braced) frames are fundamental to understanding stability design. Sway or unbraced frames are also known as portal or moment frames. Figures A.3 to A.7 show steel framed offshore structures with various combinations of braced versus unbraced bays or levels.

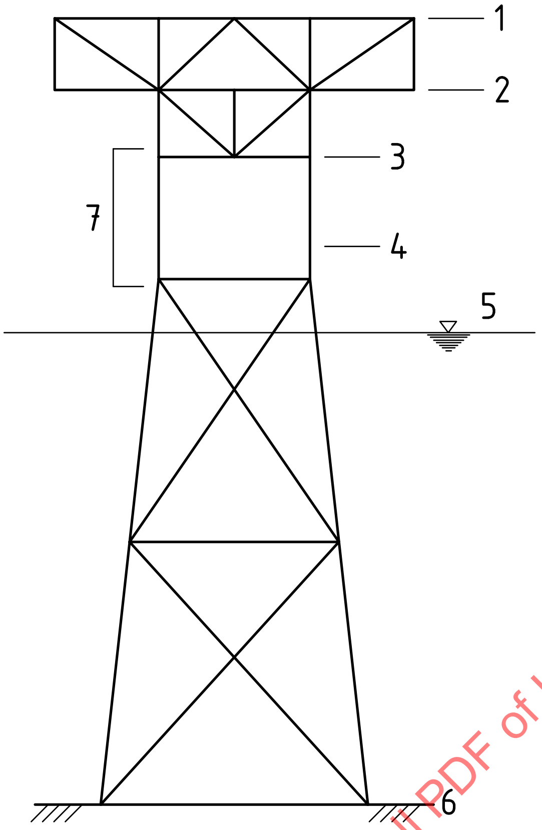
A braced frame is one where all framing levels are braced. Knee braces, which extend over only a portion of the height of a bay or between levels and not effectively over the full height of the bay, do not provide adequate bracing. Furthermore, the designer should minimize, to the extent reasonable, the distance between the bottom of bracing under the lowest deck level and the top braced level of the substructure to minimize any portal frame effect in this area - see Figure A.2. Unbraced major framing levels are likely to govern the structure’s stability.



Key

1 main deck
2 production deck
3 cellar deck
4 EL top of jacket
5 MWL
6 mudline

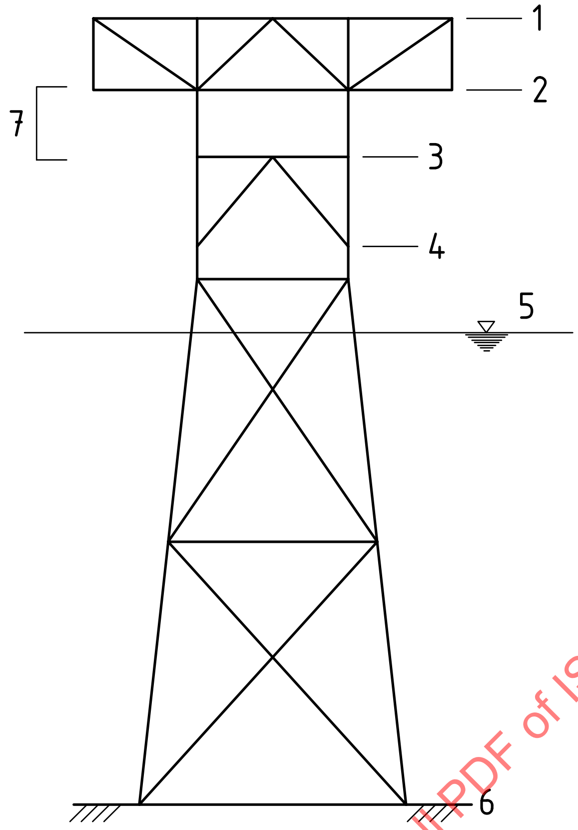
Figure A.2 — Braced topsides



Key

1 main deck
2 production deck
3 cellar deck
4 EL top of jacket
5 MWL
6 mudline
7 unbraced portal frame

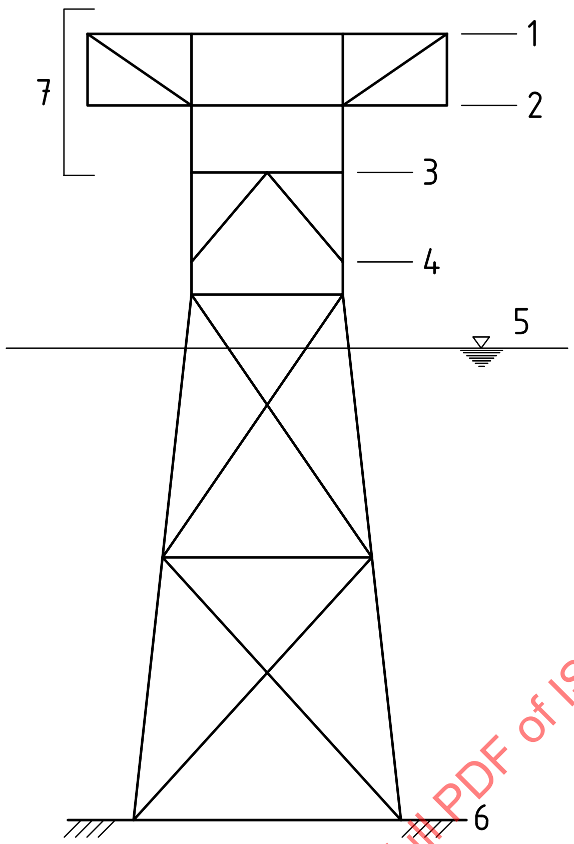
Figure A.3 — Unbraced portal frame between lowest deck and jacket



Key

1 main deck
2 production deck
3 cellar deck
4 EL top of jacket
5 MWL
6 mudline
7 unbraced portal frame

Figure A.4 — Unbraced portal frame at intermediate deck level

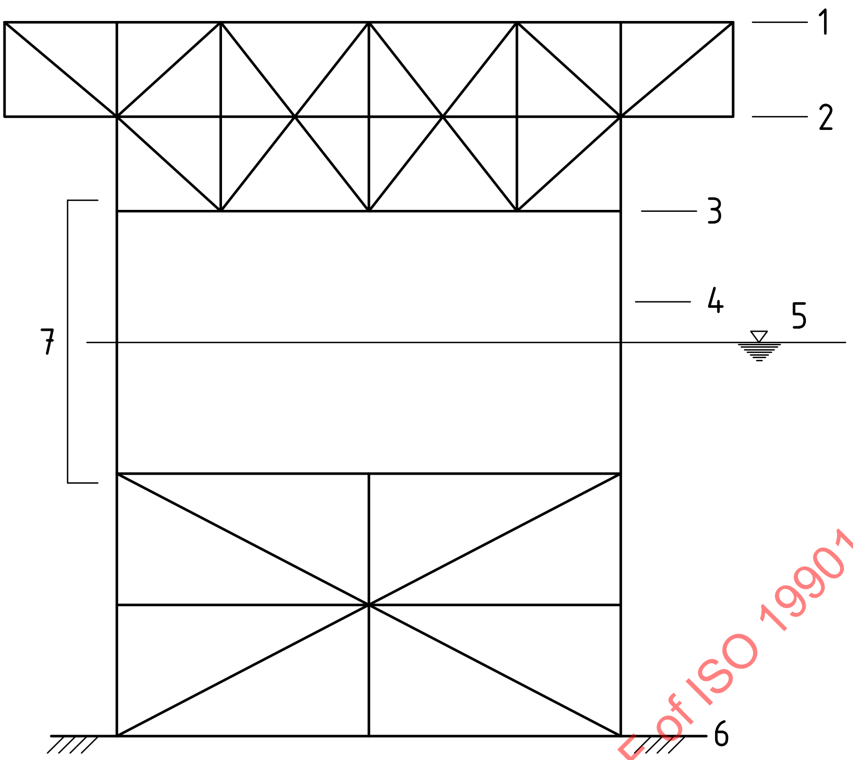


Key

1 main deck
2 production deck
3 cellar deck
4 EL top of jacket
5 MWL
6 mudline
7 unbraced portal frame between main legs

Bracings in cantilever is not necessarily sufficient to adequately minimize frame sidesway and second-order effects

Figure A.5 — Unbraced portal frame between main legs



Key

1 main deck
2 production deck
3 cellar deck
4 EL top of jacket
5 MWL
6 mudline
7 unbraced portal frame

Figure A.6 — Unbraced portal frame associated with float over deck configuration

A.8.6.3 *P*-Δ and *P*-δ second order effects frames

Figure A.7 illustrates the differences between *P*-Δ effects and *P*-δ effects. *P*-Δ effects, illustrated on the left, are the effects of actions on the displaced location of joints or nodes in a structure. *P*-δ effects, illustrated on the right, are the effects of actions on the deflected shape of a member between joints or nodes.

P-Δ effects and *P*-δ effects can be represented in a rigorous 2nd order structural analysis by use of non-linear geometry and by updating nodal displacements after every load increment. A physical member can be represented by a single beam element when based on stability shape functions or by a subdivided member comprised of say 3 or 4 beam elements when based on isoparametric shape functions. With respect to the latter, it is recommended that an even number of elements be used so that a joint/node is positioned at the midspan of the parent member where the software will display force-displacement and moment-rotation results useful in evaluating the member's 2nd order behaviour. In general, such modelling is only needed for members comprising unbraced portal bays where the 2nd order effects could be more noteworthy.

Beam elements based on engineering strain (i.e. small strain) are used in a first order structural analysis and moment amplification is used to account for the *P*-Δ effects and *P*-δ effects, e.g. amplifying the 1st order bending moment by using B_1 and B_2 as defined in ANSI/AISC 360-22,[12] Appendix 8 or using k factors as defined in EN 1993-1-1:2022,8.3.3.

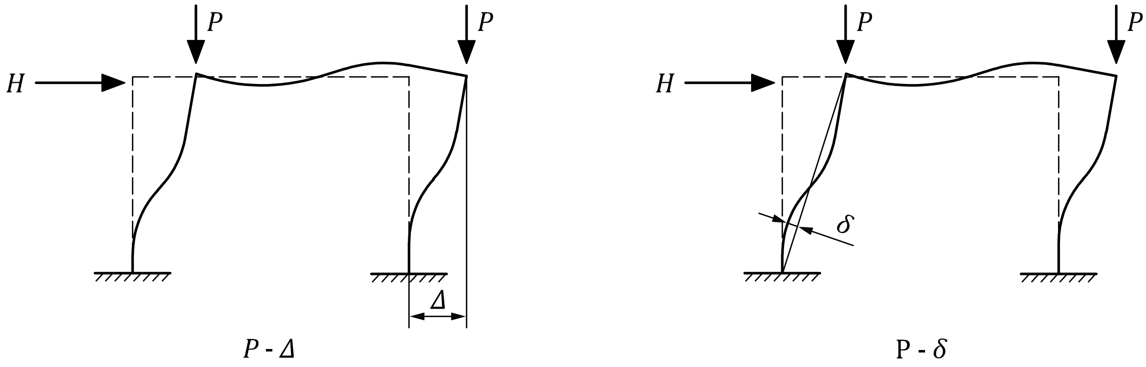


Figure A.7 — *P*-Δ and *P*-δ second order effects

Configurations other than the braced structure (Figure A.2) experience *P*-Δ effects of varying magnitude. The greater the instability the more the need to apply rigorous analysis because, the greater the instability, the more the effective length method suffers from limitations and approximations in determining effective length.

If a frame is braced against sidesway (Figure A.2), the *P*-Δ effects associated with sidesway are minimal and the *P*-δ effects are included using a 1st order analysis and the AISC ELM or EC3 ECM.

A.8.6.4 Specific requirements of ANSI/AISC 360-22[12] and EN 1993-1-1[13] stability analysis methods

The specific requirements of ANSI/AISC 360-22[12] stability analysis methods are described and compared in Table A.4.

The specific requirements of EN 1993-1-1[13] stability analysis methods are described and compared in Table A.5.

Table A.4 — Description and comparison of ANSI/AISC 360-22[12] stability analysis methods

Options included in the structural analysis for the stability design methods in clauses 8.6a), 8.6b) and 8.6c)	8.6b) ANSI/AISC 360-22, C2[12] DM–elastic	8.6a) elastic ANSI/AISC 360-22:Appendix 1.2[12] DMMI-elastic	8.6c) ANSI/AISC 360-22: Appendix 7.2[12] ELM (B_2<1,5)	8.6a) inelastic ANSI/AISC 360-22:Appendix 1.3[12] DMMI-inelastic
Axial, flexure, and shear member deformations plus connection deformation	yes	yes	yes	yes
Torsional member deformations	no	yes	no	yes
Second order *P*-Δ effects (system)	yes, or see (1) below	yes	yes, or see (1) below	yes
Second order *P*-δ effects (member)	yes, or see (1) below	yes	yes, or see (1) below	yes
Second order twisting effects (open sections only)	no	yes	no	yes
Effects of initial imperfections due to points of intersection of members displaced from their nominal locations (system imperfections) – only applies if *E* (wind + wave) < 0,2 % x (*G*+*Q*) i.e. still water condition and operating condition in benign regions.	Yes, explicitly, by eigenvector or by notional lateral load	yes, explicitly, by eigenvector scaled to max fabrication tolerance	yes, by notional lateral load for *G*+*Q* conditions only	yes, explicitly, by eigenvector scaled to max fabrication tolerance